Research Report
UKTRP-83-14

GEOTECHNICAL, HYDROLOGIC, AND HYDRAULIC INVESTIGATION OF MILL CREEK DAM -- PHASE II

by

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INTRODUCTION

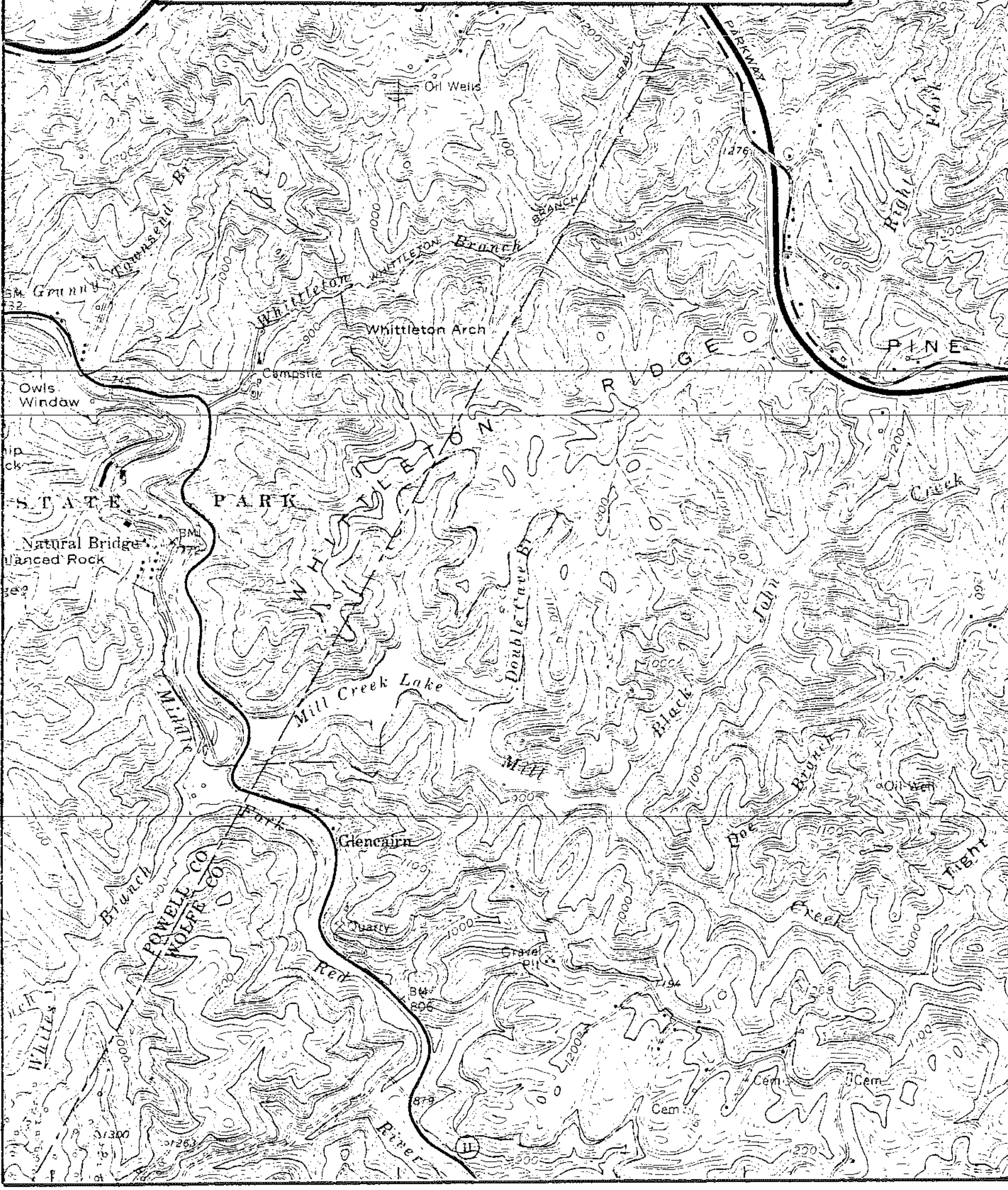
Mill Creek Dam is located in Natural Bridge State Park three miles southeast of Slade in Powell County, Kentucky. A portion of a 7-1/2-minute topographic quadrangle map is presented in Figure 1 showing the location of the structure. The dam is an earth and rockfill structure and impounds a 40-acre lake. The dam also serves as a highway embankment carrying highway route KY 11. Construction plans prepared by the Kentucky Department of Transportation, which owns the dam, were completed in 1962. The dam was built in 1964. A drawing in the design plans shows a drawdown for this dam, but the drawdown facility was never built. Filter zones serving as a transition between the impervious clay core and the random rockfill are not shown on the as built plans, although they were shown on preliminary and record plans (1). The dam (earth core and upstream embankment) is approximately 400 feet long. A downstream embankment, continuing as a sidehill fill for an additional 400 feet, also serves to impound water. A typical embankment section has a crest width of 55 feet and 2:1 upstream and downstream slopes. Total height of the dam measured from the crest to the estimated original stream channel is 71 feet. However, waste material from the roadway construction was placed as a wide, gently sloping berm that intersects the downstream slope at elevation 789 feet and results in an effective downstream slope height of 42 feet. The clay core has a cutoff trench into rock, but there is no grout curtain.

Mill Creek Lake has a drainage area of 5.39 square miles. At normal pool (elevation 821.1 feet), the lake has a surface area of 40.6 acres and a storage capacity of 1,049 acre-feet. At elevation 830.8 feet (top of dam), the surface area is 50.6 acres and the storage capacity is 1,341 acre-feet. Elevation of the top of the earth core is 820.0 feet. The overflow spillway inlet is at an elevation of 821.1 feet. Thus, normal pool is at least a foot higher than the crest of the earth core.

The outlet facility at Mill Creek is an ungated service spillway consisting of a rectangular concrete-lined flume 10 feet wide and 470 feet long with an inlet elevation of 821.1 feet, a 16-by-13.7-foot drop inlet box, and a 12-by-8-foot box culvert under KY 11, which has a free outlet down a rock cascade to the natural channel. The spillway and box culvert are significantly undersized, as shown by a Phase I study (2) and have the capacity to pass only a very small portion of the probable maximum flood (PMF).

On August 8, 1972, the Congress of the United States of America enacted Public Law 92-367 (3) authorizing the Secretary of the Army, through the Corps of Engineers, to undertake a national program for the inspection of dams. With some exceptions, all water-retaining structures having heights in excess of 25 feet and/or structures impounding more than 50 acre-feet of water were required to be inspected. Mill Creek Dam, having a height of 71 feet and a storage capacity of 1,341 acre-feet, was classified as an intermediate-sized dam with a high hazard potential. Hazard potential is not based on the condition of the dam itself but on the potential for loss of human life and/or property damage in the event of failure. There is a campground at the toe of the dam. Additionally, there is a second dam about 4,000 feet downstream and development in low-lying areas further downstream, all of which would probably be affected by a sudden failure of Mill Creek Dam. Due to that downstream development and to

Figure 1. Portion of a United States Geologic Survey 7-1/2-Minute Topographic Quadrangle Map Showing the Location of Mill Creek Dam.



the presence of KY 11 on the dam's crest, the dam was classified as a high-hazard potential. Although the high-hazard potential classification does not imply the dam is hazardous, it does make it appropriate to place a higher priority on assessing the safety of dams having that classification.

The performance of the dam has been only fairly satisfactory. Reservoir levels have been steady, allowing the lake to be used for recreation and water supply. However, there is evidence that pool elevation has been near the top of the dam. Elevation of water marks inside a boathouse at the site are near the elevation of the top of the dam. Based on eyewitness accounts, the pool level has been at the top of the dam. In terms of stability, there is no obvious settlement, cracking, or deviation of the roadway, or dam crest alignment. However, a poorly defined bulge was noted in the Phase 1 (2) inspection (June 14, 1978). The bulge is located near the top of the downstream slope close to the left abutment. Although the bulge appeared to be a slump, there were no signs of recent movement. The preliminary stability analysis of the downstream slope determined in the Phase 1 study indicated that the long-term factor of safety for steady seepage could be less than 1.5. Results obtained from the Phase I study indicated a need for additional in-depth study, exploration, and analyses.

SCOPE AND OBJECTIVES

The general scope of this study, Phase II, was to assess the safety of Mill Creek Dam. Findings obtained from detailed geotechnical, hydraulic, and hydrological investigations are presented. The structural stability, as well as the hydrological and hydraulic stability, were investigated. Specifically, objectives of the study were as follows:

1. To determine the engineering characteristics of the clay core, shells, and random fill.
2. To evaluate the potential for piping.
3. To evaluate seepage conditions at the site.
4. To evaluate the structural stability of the earth and rockfill dam.
5. To evaluate erodability.
6. To assess geologic conditions at the site.
7. To evaluate existing and required spillway hydraulics and hydrology of the site.
8. To analyze requirements for a drawdown facility.
9. To evaluate alternative remedial measures that could be used to correct deficiencies in the dam.

This study presents data relating to the degree of safety and alternative remedial schemes. Information presented herein will aid in the final selection of the remedial method and in implementing remedial construction. Development of detailed remedial plans, however, was not within the scope of this study.

GENERAL TOPOGRAPHY AND GEOLOGY

The dam is located on the boundary of the Mississippian Plateau and the Eastern Kentucky Physiographic Region of the Cumberland Plateau. The area is a maturely dissected plateau of varying altitude and relief. The area has a dendritic drainage pattern and contains irregularly winding narrow-crested ridges and deep narrow valleys. The site is shown on a portion of a United States Geologic Survey 7-1/2-minute topographic quadrangle map, Figure 1 (4). General relief of the terrain in the vicinity of the site ranges from about elevation 1,300 feet to 740 feet.

Geology of the site is shown on a portion of a United States Geologic Survey 7-1/2-minute geologic quadrangle map in Figure 2 (5). A generalized geologic columnar section of the site is shown in Figure 3. A cross-sectional view of the geology at the site along Section B-B' in Figure 2 is shown in Figure 4. The earth and rockfill dam rests on the Cowbell Member of the Borden Formation (Mississippian System). The Cowbell consists of siltstone (70 percent) and shale (30 percent). The siltstone is greenish and yellowish-gray and weathers to a yellowish-gray to yellowish-brown. It is locally stained dark brown by limonite. The shale is greenish and olive-gray and weathers to the same color and yellowish-gray.

Located above the Cowbell Member are the Nada and Renfro Members of the Borden Formation. Those members are predominantly dolomite and limestone, respectively. The Newman Limestone is located above the Nada and Renfro members. The Breathitt and Lee formations (Pennsylvanian System) are located in the upper reaches of the site. Those formations consist of sandstone, siltstone, and shale. Alluvium (Quaternary System) is located in the valleys of the site and consists of silt, clay, and gravel. The Borden Formation is generally covered with thin colluvium, which consists of locally derived siltstone and shale. As shown in Figure 2, the Glencairn Fault of the Irvine-Paint Creek Fault Zone is located approximately 2,500 feet south of the dam. Historically, the fault is inactive. The dam is sited in a Seismic Zone 1.

SITE GEOMETRY

A plan view of the site is shown in Figure 5. That figure shows the limits of the earth core and the arrangement of the spillway flume, drop inlet box, culvert, and rock cascade. No emergency spillway was constructed. Moreover, no emergency drawdown facilities were constructed. The earth core and upstream embankment are about 400 feet in length. The downstream embankment continues as a sidehill fill for an additional 400 feet.

As shown in a typical cross section, Figure 6, the earth and rockfill dam has a crest width of 55 feet. The upstream and downstream slopes of the dam are 2 horizontal to 1 vertical. Height of the dam measured from the crest to the estimated original stream channel is 71 feet. A berm constructed of waste material is situated at the toe of the downstream slope and, as a result, the effective height of the downstream slope is about 42 feet. The upstream and downstream slopes of the earth core as shown on the as-built plans are approximately 1.5 horizontal to 1 vertical and 1 horizontal to 1 vertical, respectively. From the as-built plans, the keyway trench is about 15 feet deep and 25 feet wide. The trench (slot) projects into the rock foundation about 3 feet. According to the as-built plans, no transitional zones between the clay core and rockfill shells were constructed, although the record plans show transitional filter zones. Also, neither the record plans nor as-built plans showed horizontal inverted filters and toe drains to control seepage.

A cross-sectional view along the roadway centerline of the site is shown in Figure 7. Between Stations 176+00 and about 180+50, the thickness of foundation soils ranges from approximately 2 feet to 35 feet. Slope of the rock abutment

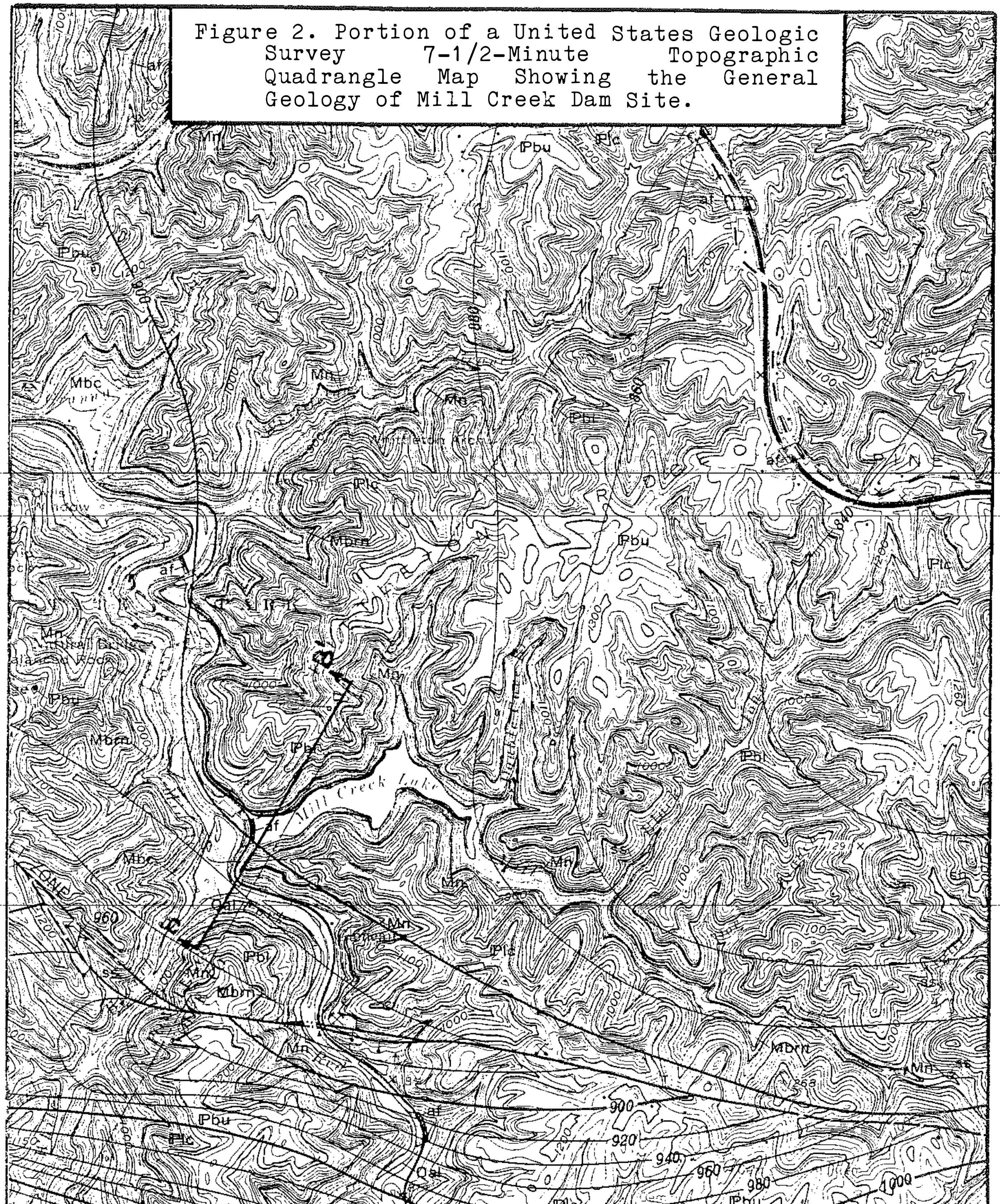


Figure 2. Portion of a United States Geologic Survey 7-1/2-Minute Topographic Quadrangle Map Showing the General Geology of Mill Creek Dam Site.

System	Series	Formation, Member, And Bed		Lithology	Description
			Alluvium		Silt, Clay, and Gravel
			Upper Member of the Breathitt Formation (Pbu)		Shale and Siltstone (90 Percent) and Sandstone (10 Percent), Inter-bedded
Pennsylvania	Lower and Middle Pennsylvania	Lee and Breathitt Formations	Corbin Sandstone Member of the Lee Formation (Pbc)		Sandstone (60 Percent) and Conglomeratic Sandstone (35 Percent), Conglomerate (About 5 Percent) and Minor Shale)
			Lower Tongue of the Breathitt Formation (Pbl)		Shale and Siltstone (90 Percent), Sandstone and Conglomeratic Sandstone (10 Percent), and Minor Coal.
Mississippian	Mississippian	Newman Limestone	Upper Member (Mn) St. Genevieve Limestone Member. St. Louis Limestone Member		Limestone (90 Percent) and Shale (10 Percent)
		Borden Formation	Renfro Member (Mbrn)		Dolomite, Silty
			Nada Member		Shale (80 Percent) and Shale (20 Percent).
			Cowbell Member (Mbc)		Siltstone (70 Percent) ond Shale (30 Percent)

Figure 3. Generalized Geologic Columar Section of the Mill Creek Dam Site.

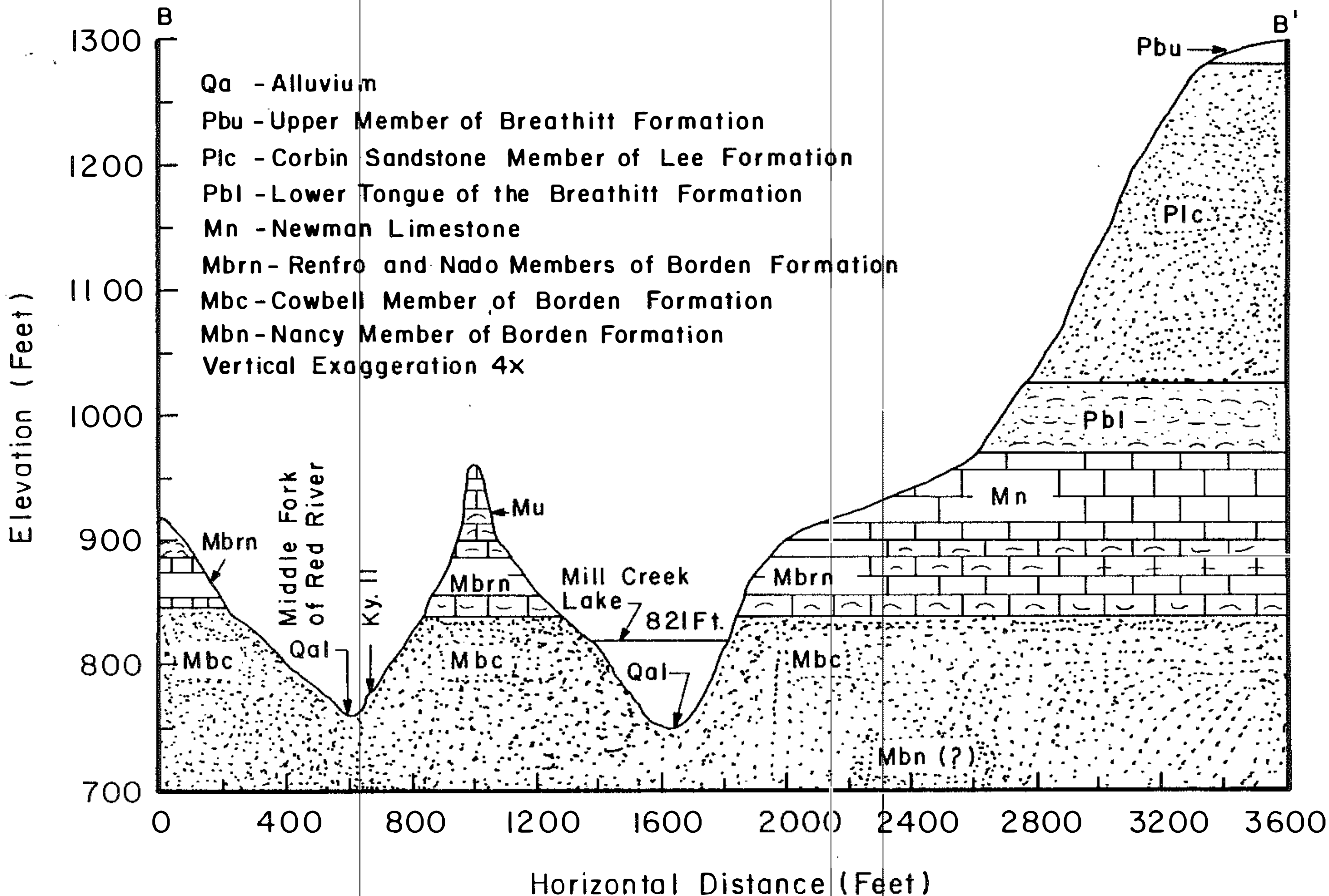


Figure 4. Cross-Sectional View of the Geology at the Mill Creek Dam Site along Section B-B' in Figure 2.

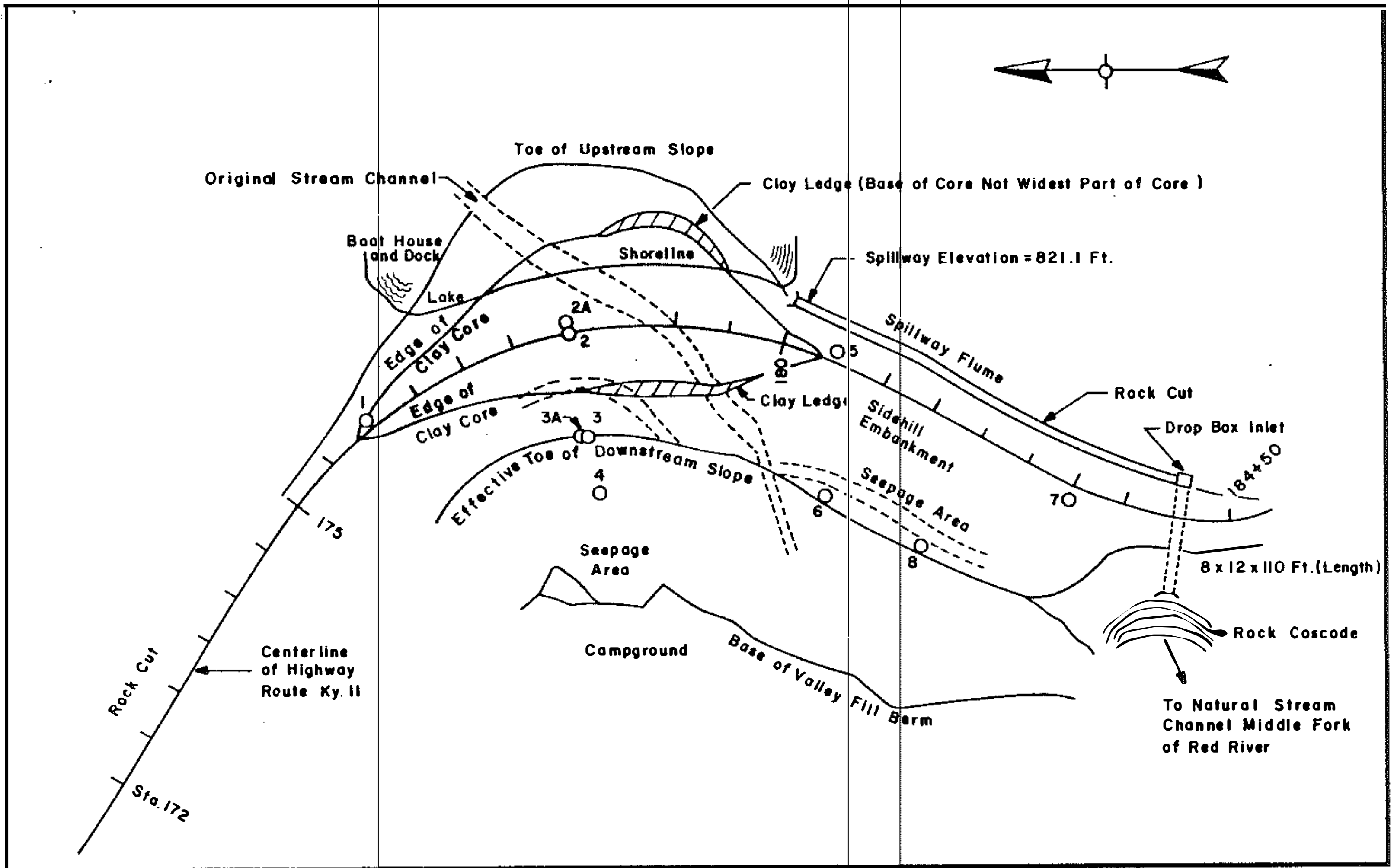


Figure 5. Plan View of the Mill Creek Dam Site.

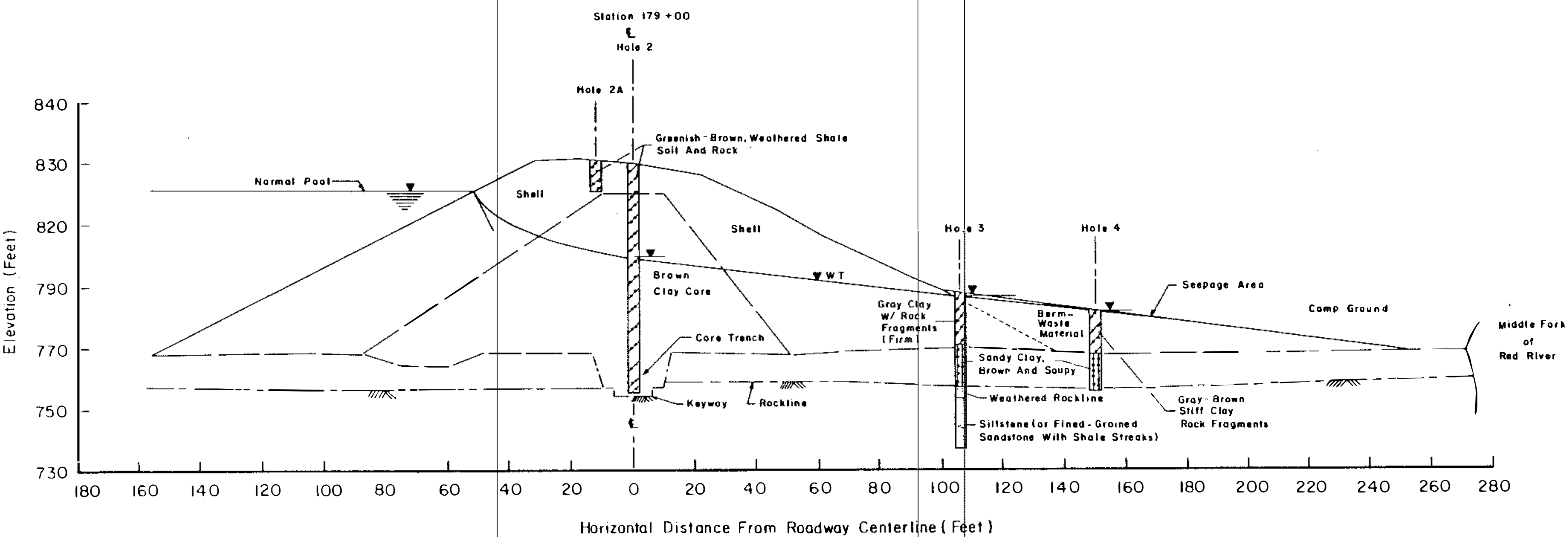
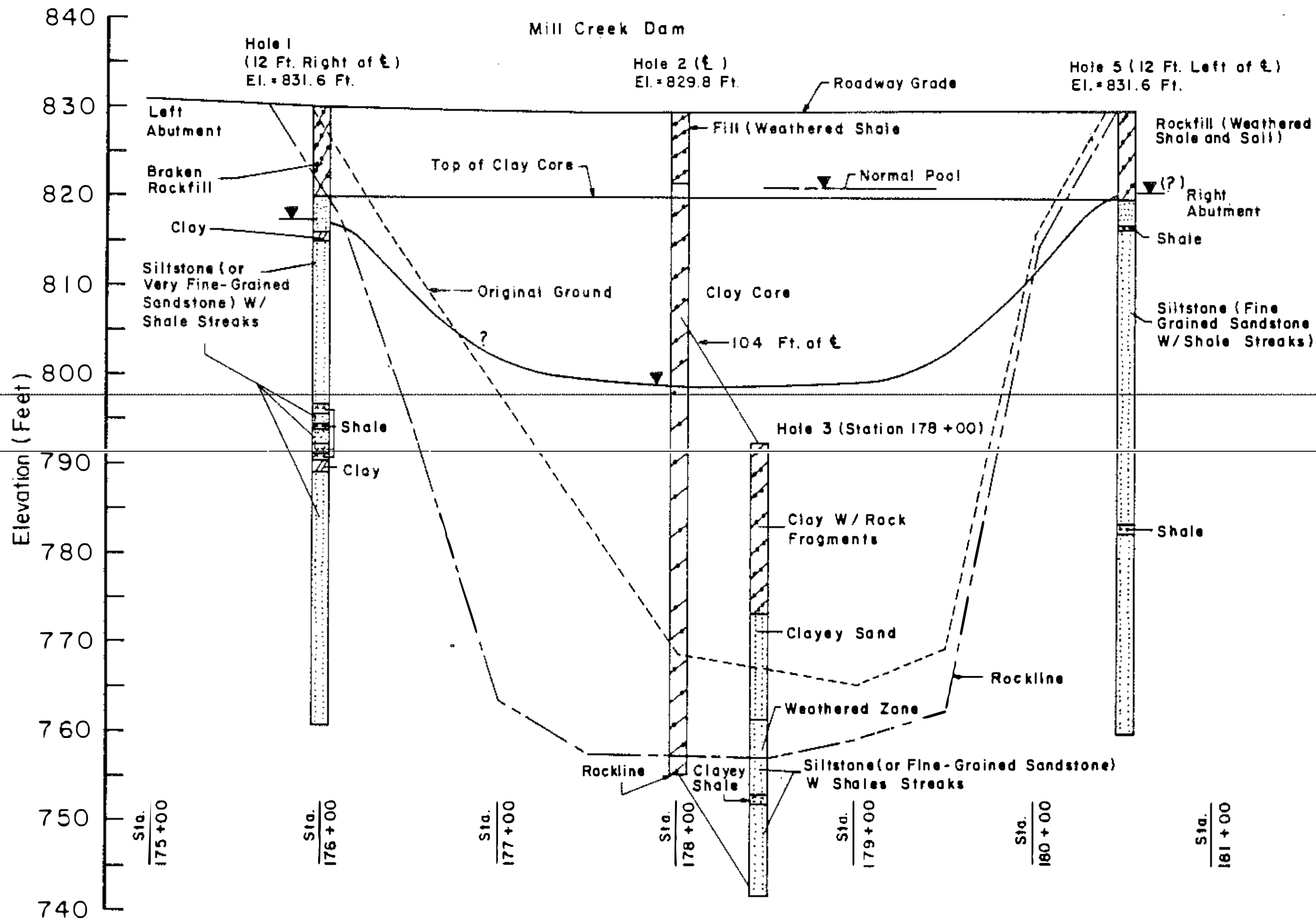


Figure 6. Typical Cross Section of the Mill Creek Dam at Station 178+00.



~~Figure 7. Cross-Sectional View of the Mill~~
Creek Dam along Centerline of Highway.

between Stations 176+00 and 177+00 is approximately 1 vertical to 1.7 horizontal. Slope of the rock abutment between Stations 179+50 and 180+00 is about 1 horizontal to 1 vertical.

A typical cross section of the spillway flume at Station 181+00 is shown in Figure 8. Depth of the flume is about 12 feet. Width of the spillway ranges from 32 feet at the top to 10 feet at the bottom. Approximate area of the spillway is 230 square feet. The area of the box culvert opening is 96 square feet (12 feet x 8 feet).

GEOTECHNICAL INVESTIGATION

The subsurface exploration program included ten borings. Nine of the borings were drilled using 6-inch diameter hollow stem augers. One hole was drilled using 4-1/2-inch diameter hard-stem augers. Locations of the borings are shown in Figure 5. Borings from the top of the dam include the following:

Hole 1, Station 176+00,
12 feet right of centerline
Hole 2, Station 178+00,
centerline
Hole 2A, Station 178+00,
12 feet left of centerline
Hole 5, Station 180+50,
12 feet left ofcenterline
Hole 7, Station 183+00,
12 feet right of centerline

Holes in the vicinity of the toe of the dam include the following:

Hole 3, Station 178+00,
104 feet right of centerline
Hole 3A, Station 178+00,
104 feet right of centerline
Hole 4, Station 178+00,
150 feet right of centerline
Hole 6, Station 181+00,
134 feet right of centerline
Hole 8, Station 182+00,
136 feet right of centerline

Rock core samples were obtained from Holes 1, 5, and 3 using a wire-line core system. Depths of the cores obtained from those holes were 70 feet, 70 feet, and 17 feet, respectively. Locations of the core holes are shown in Figures 5 and 7. In Hole 3, the boring was extended below the embankment and dam. Holes 1 and 5 were drilled in the left and right abutments of the dam, respectively. Thin-walled Shelby tube samples were obtained from Holes 2, 2A, 3, 4, and 8. Split-spoon samples were obtained from Holes 3A and 7. Because of the presence of rock fragments, some difficulties were encountered in attempting to obtain thin-walled tube samples. Several bag samples were obtained from Hole 2. All samples and bore-hole materials were described during drilling based on visual inspections.

LABORATORY TESTING PROGRAM

Index tests were performed on samples recovered from the dam to identify and classify materials from various zones of the dam. Those tests included natural water contents, Atterberg limits, specific gravities, and particle-size analyses. Natural water contents were performed according to ASTM D 2216-80. Atterberg limits were performed according to procedures of ASTM D 423-66(72) and D 424-59(71). Particle-size determinations were made according to procedures similar to ASTM D 421-58(78) and D 422-63(72). Specific gravity tests were performed according to ASTM D 854-58(79). Thin-walled Shelby tube samples (ASTM D 1587-74), split-spoon samples, and bag samples were described in the laboratory using the visual-manual procedure, ASTM D 2488-69(75). Rock-core specimens were logged in the laboratory. The soils were classified using the Unified Soil Classification System and ASTM D

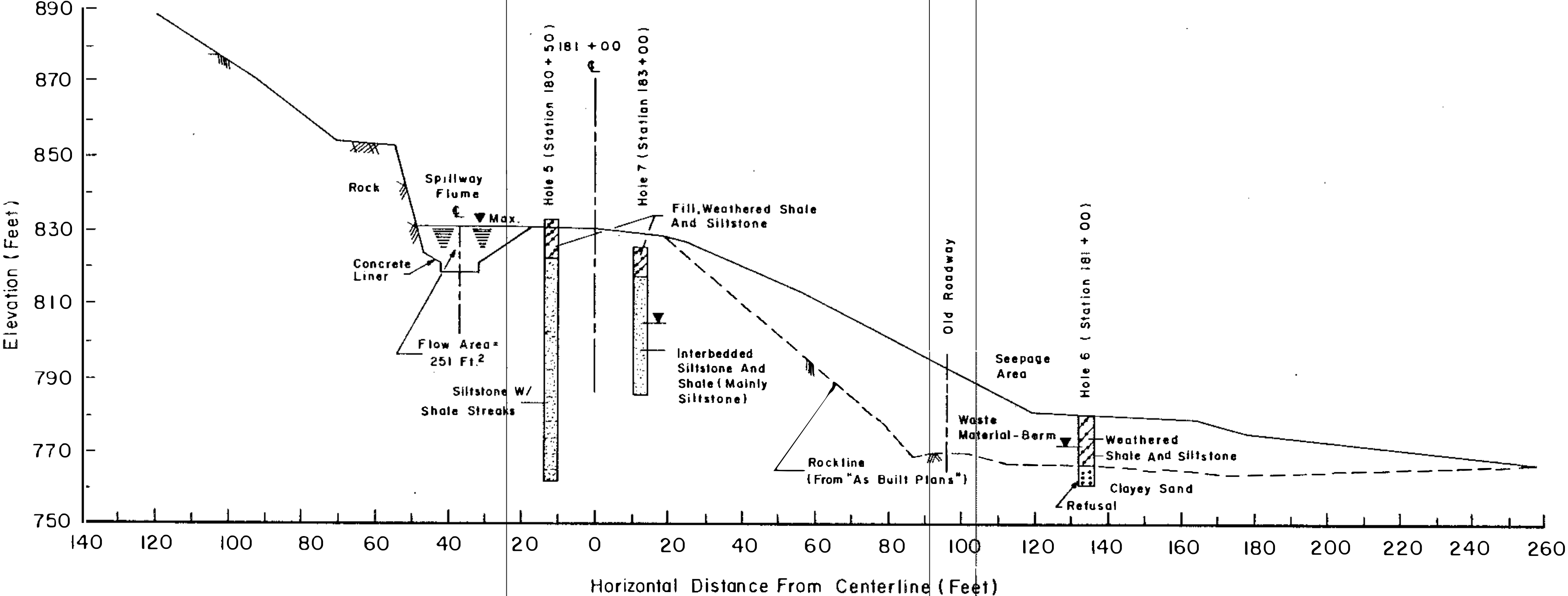


Figure 8. Typical Cross Section of the Service Spillway Flume at Station 181+00.

2487-69(75).

Engineering tests were performed to define the properties of the materials used to construct the dam. Those included isotropically consolidated undrained triaxial compression tests with pore-pressure measurements, constant-head permeability tests, and moisture-density compaction tests. The isotropically consolidated undrained triaxial compression tests with pore-pressure measurements were performd on thin-walled Shelby tube samples according to procedures described elsewhere (6, 7, and 8). Unconsolidated undrained triaxial tests were not performed because of a scarcity of samples. Constant-head permeability testing procedures described elsewhere (9, 10) were used to define permeability characteristics of the earth core material of the dam. Those tests were performed on thin-walled Shelby tube samples in a triaxial chamber. Compaction tests were performed on bag samples of the clay core following procedures described in ASTM D 698-78, Method A.

RESULTS AND DISCUSSION

BORING LOGS

Detailed descriptions of the soil and rock materials encountered at the site are shown in APPENDIX A. Boring logs for Holes 1 and 3, 2 and 7, 3A and 5, 4 and 8, and 2A and 6 are shown in Figures A.1, A.2, A.3, A.4, and A.5, respectively. Locations of those borings are shown in Figure 5.

INDEX CLASSIFICATION TESTS

Results obtained from laboratory index testing of specimens from Shelby-tube samples are summarized in Table 1. Locations from which the samples were taken are shown in APPENDIX A.

ENGINEERING TESTS

Results obtained from isotropically consolidated undrained triaxial compression tests with pore-pressure measurements performed on materials from the various zones of embankment and foundation are summarized in Table 2. Stress paths and K_f-failure envelopes for the clay core, shell, waste disposal berm, and foundation soils are shown on p-q diagrams in Figures 9, 10, 11, and 12, respectively. Only one triaxial test was performed on the foundation soils because only one Shelby-tube specimen could be retrieved from the sandy foundation materials.

Results obtained from moisture-density compaction tests (ASTM 698-78) on bag samples of the clay core are shown in Figure 13. To determine values of relative compaction of the clay core, dry unit weights and water contents obtained from three Shelby-tube specimens were compared to the maximum dry density and optimum water content of the clay core in Figure 13.

Permeability test results obtained from the triaxial chamber technique as discussed above are shown in Figures 14 and 15. The value of the vertical coefficient of permeability or hydraulic conductivity shown in Figure 14 was obtained from a Shelby-tube specimen from the shell material (Boring 2A, Specimen S-1B). The specimen was obtained from a point located above the clay core. In Figure 15, the vertical coefficient of permeability was obtained from a test on a Shelby-tube specimen of the clay core.

SEEPAGE OBSERVATIONS

Groundwater elevations observed in cased bore holes are presented in APPENDIX B. Groundwater elevation- -time curves for Bore holes 2, 3, and 4; 6 and 8; and 1, 5, and 7 are shown in Figures B.1, B.2, and B.3,

TABLE 1. SUMMARY OF LABORATORY TESTS

HOLE NUMBER AND STATION	SAMPLE NUMBER	DEPTH (feet)	ELEVATION (feet)	DESCRIPTION	NATURAL WATER CONTENT (percent)	LIQUID LIMIT	PLASTIC LIMIT	SPECIFIC GRAVITY	PERCENT PASSING NO. 4	PERCENT PASSING NO. 10	PERCENT PASSING NO. 200	USCS*
2A(178+00,12L) El=831.08ft	1A	1.7-4.2	829.3-826.8	Weathered shale	19.6							
	1B			Weathered shale	14.8							
	1C			Weathered shale	13.6	37.0	21.2	2.79	81.5	78.8	75.3	CL
	2A	5.0-7.4	826.1-823.2		10.6							
	2B				10.8	32.0	20.3	2.76	90.3	88.1	78.0	CL
	3A	7.4-7.9	823.7-	Green shale	10.2	32.5	20.5	2.77	77.3	65.9	58.4	CL
	3B			Green shale	8.7	32.0	20.7	2.76	92.5	92.1	83.1	CL
2 (178+00,CL) EL= 829.83	1A	8.3-10.3	821.5-819.5	Green shale	13.2							
	1C											
	2A	18.3-20.3	811.5-809.5	Brown clay	17.0							
	3A	28.3-30.3	801.5-799.5	Brown clay w/ rock fragments	14.5							
	4A	29.0-30.3	800.8-799.5	Brown clay w/ gravel	20.1							
	6A	38.3-40.3	791.5-789.5	Brown clay w/ small gravel	16.7	37.0	20.1	2.76	95.0	92.4	77.5	CL
	6B			Brown clay w/ few gravel	16.7							
	7A	43.3-45.3	786.5-784.5	Bro wn clayw/ small gravel	12.6	36.0	17.9	2 .76	79.8	76.4	63.0	CL
	8A	48.3-50.3	781.5-779.5	Brown clay w/ rock fragments	17.9	36.5	18.2	2.63	83.3	81.3	69.0	CL
	10A	68.3-70.0	761.5-759.8	Brown clay w/ rock fragments	17.7	36.0	18.3	2.76	91.5	90.3	78.3	CL
	11A	73.3-74.0	756.5-755.8	Brown clay	25.1							

TABLE 1 (continued)

HOLE NUMBER AND STATION	SAMPLE NUMBER	DEPTH (feet)	ELEVATION (feet)	DESCRIPTION	NATURAL WATER CONTENT (percent)	LIQUID LIMIT	PLASTIC LIMIT	SPECIFIC GRAVITY	PERCENT PASSING NO. 4	NO. 10	NO. 200	USCS*
3(178+ 100,104R) EL= 1787.51ft	1A			Weathered shale & soil	12.0	32.5	19.8	2.73	96.9	87.6	78.6	CL
	1B				16.6							
3A	1						NP	2.66	78.7	64.5	40.8	SM
	2						NP	2.67	93.6	90.5	16.2	SM
4(178+ 100,150R	1A	8.0-10.0		Weathered shale & soil	10.9	34.0	19.6	2.75	82.5	78.3	68.2	CL
	1B			Weathered shale & soil (hard green shale at 10')	6.2							
	2A	13.5-15.0		Weathered shale & soil	12.0	34.0	21.8	2.73	93.0	90.5	66.4	ML-CL
	2B			Weathered shale & soil	16.1							
	3A	18.2-20.2		Soft, muddy weathered shale	23.5							
	3B			Sandy weathered shale	22.4							
	3C			Weathered shale (sandn ear bottom)	20.4		NP	2.70	77.9	75.0	47.4	SM
7(182+ 100,136R)	1A	6-8.0		Shale and sandstone	16.6							
	1B			Green shale and brown soil	20.3	30.0	18.8	2.76	97.8	97.3	85.6	CL
	1C			Soft & muddy (greenish) and brown	24.9	29.3	17.9	2.73	97.3	95.5	79.4	CL

*Unified Soil Classification System

TABLE 2. SUMMARY OF TRIAXIAL TEST RESULTS

SPECIMEN NUMBER	DAM MATERIAL	INITIAL DRY UNIT WEIGHT (lbs/ft3)	INITIAL WATER CONTENT (percent)	INITIAL VOID RATIO	EFFECTIVE CONFINING PRESSURE (psi)	Qf	Pf	EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS PHI' (degs)	c' (psf)
H-2 S-6A	Clay core	110.6	21.1	.557	47.0	22.22	43.40	25.9	305.
H-2 S-10A	Clay core	109.1	20.4	.579	27.0	16.34	34.73		
H-2 S-8A	Clay core	107.5	21.9	.526	61.5	25.03	54.27		
H-4 S-2A	Berm	110.7	22.7	.539	30.9	18.92	33.63	28.0	573.
H-4 S-1A	Berm	115.2	17.0	.490	45.0	20.79	43.47		
H-7 S-1C	Berm				43.7				
H-7 S-1B	Berm				59.7				
H-2A S-3A	Shell	113.8	15.2	.519	31.0	22.74	41.46	30.6	271.1
H-2A S-1C	Shell	108.1	25.0	.593	88.5				
H-2A S-3B	Shell	123.2	13.4	.398	60.0	44.97	85.11		
H-3 S-1A	Shell	111.6	21.7		30.8				
H-4 S-3C	Foundation material	106.3	23.2		59.8	19.81	35.57	33.8	0.0

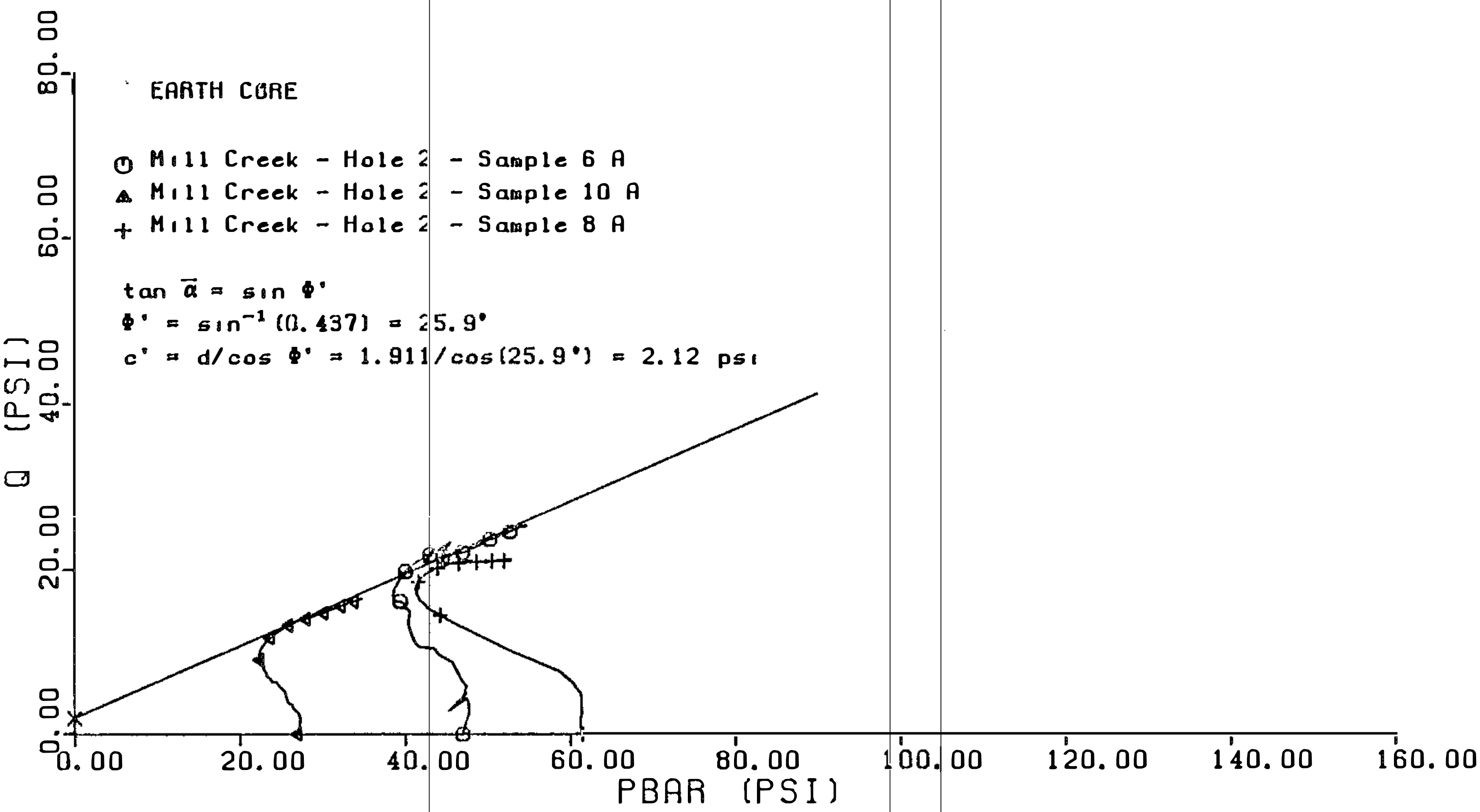


Figure 9. Stress Paths and K_f -Failure Envelope Obtained from Consolidated Undrained Triaxial Compression Tests (with Pore-Pressure Measurements) of Clay Core Specimens.

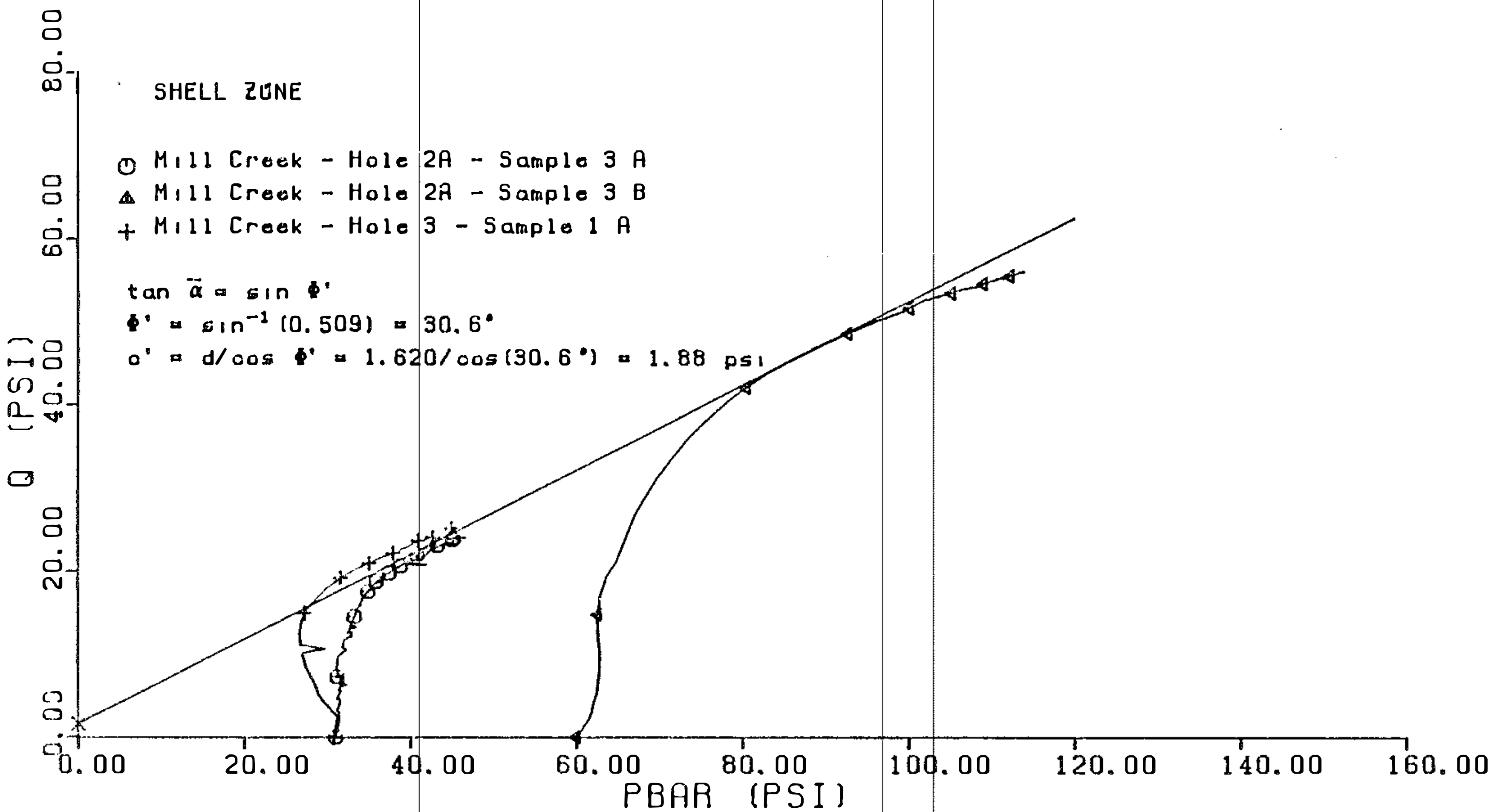


Figure 10. Stress Paths and K_f -Failure Envelope Obtained from Consolidated Undrained Triaxial Compression Tests (with Pore-Pressure Measurements) of Shell Specimens.

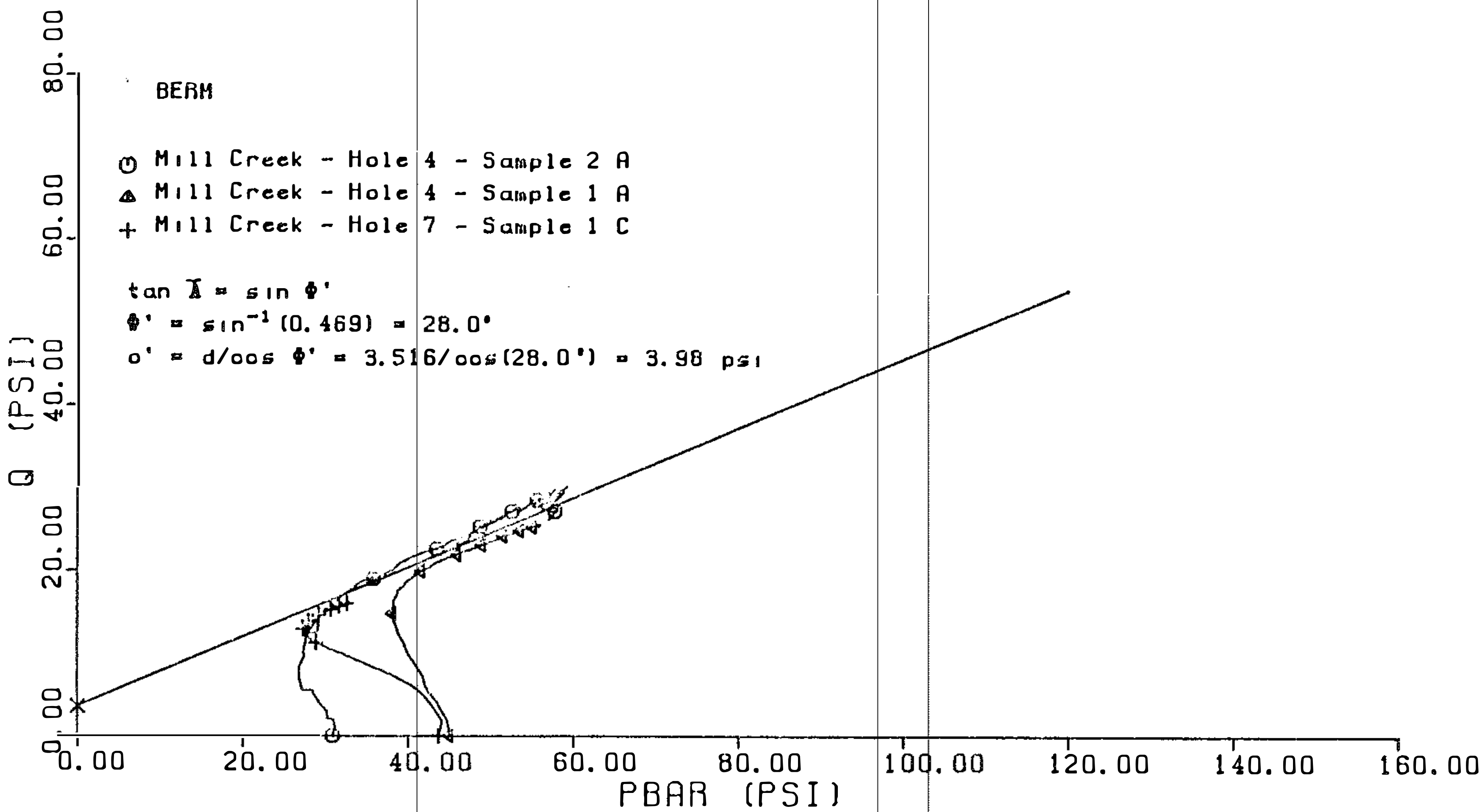


Figure 11. Stress Paths and K_f-Failure Envelope Obtained from Consolidated Undrained Triaxial Compression Tests (with Pore-Pressure Measurements) of Waste Disposal Berm.

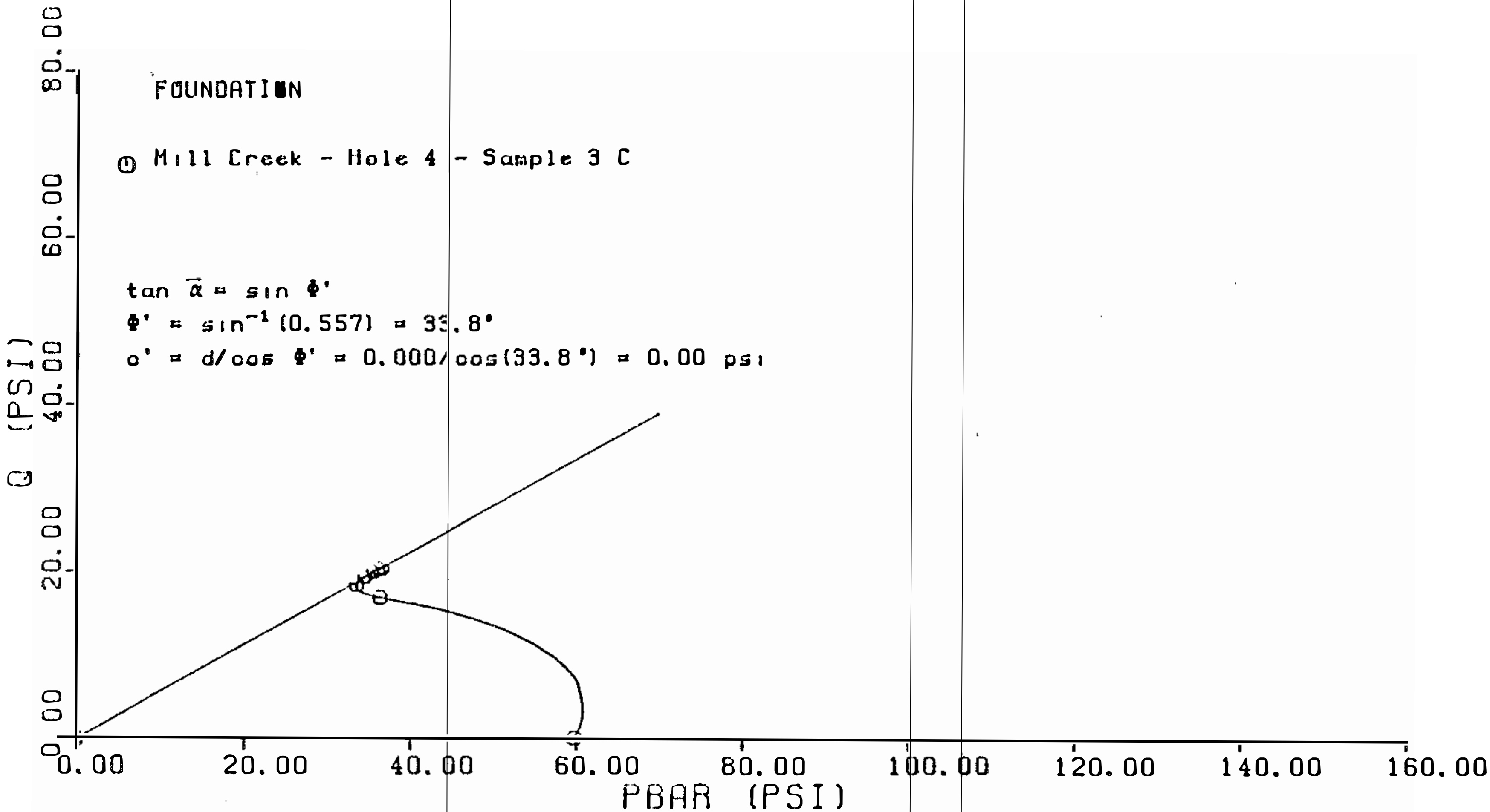


Figure 12. Stress Paths and K_f -Failure Envelope Obtained from Consolidated Undrained Triaxial Compression Tests (with Pore-Pressure Measurements) of a Specimen of the Foundation Soil.

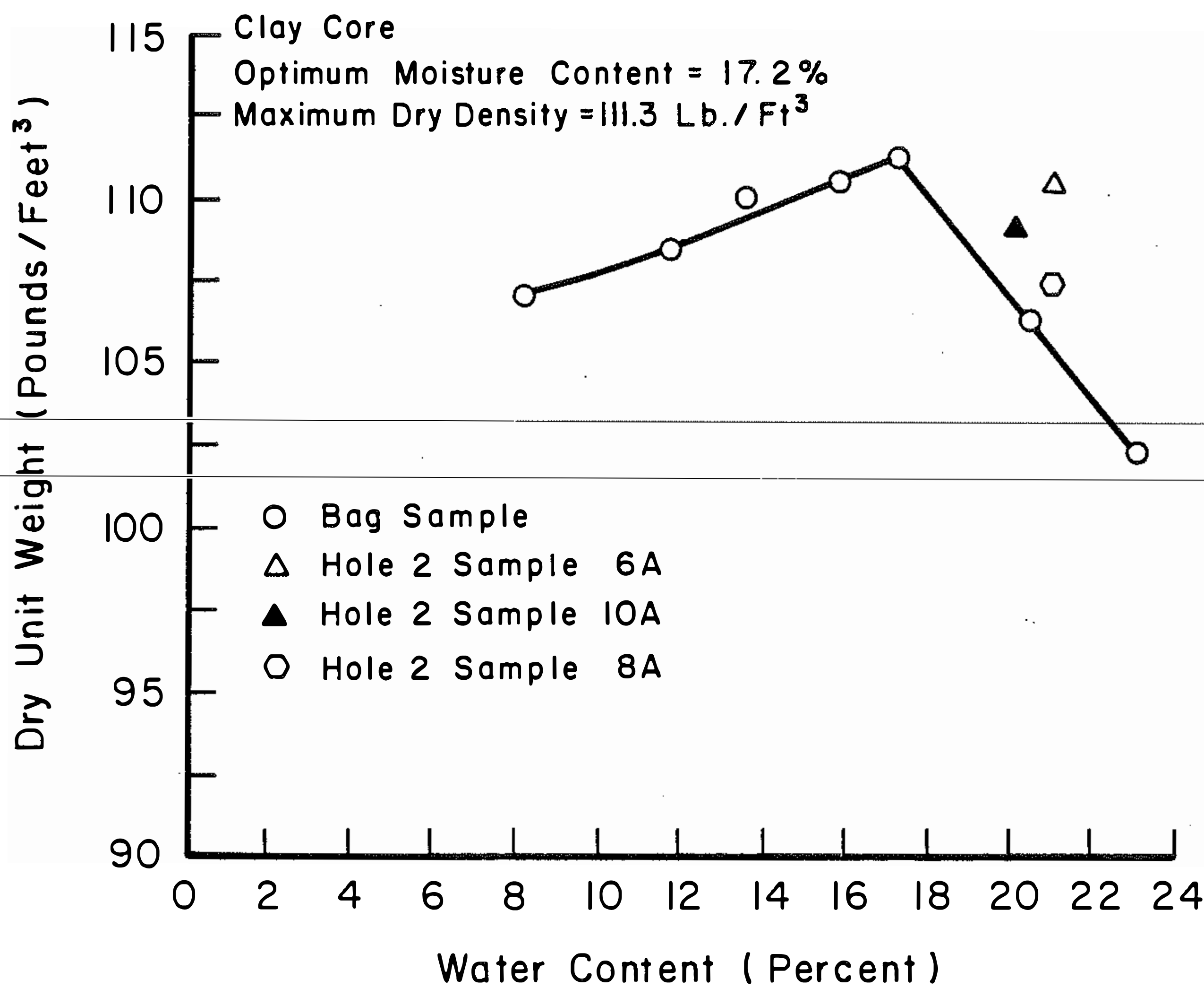


Figure 13. Comparison of Dry Unit Weight-Water Content Relationship of the Earth Core Obtained from Standard Compaction (ASTM D-698, Method A) and In Situ Dry Densities and Water Contents Obtained from Shelby-Tube Samples.

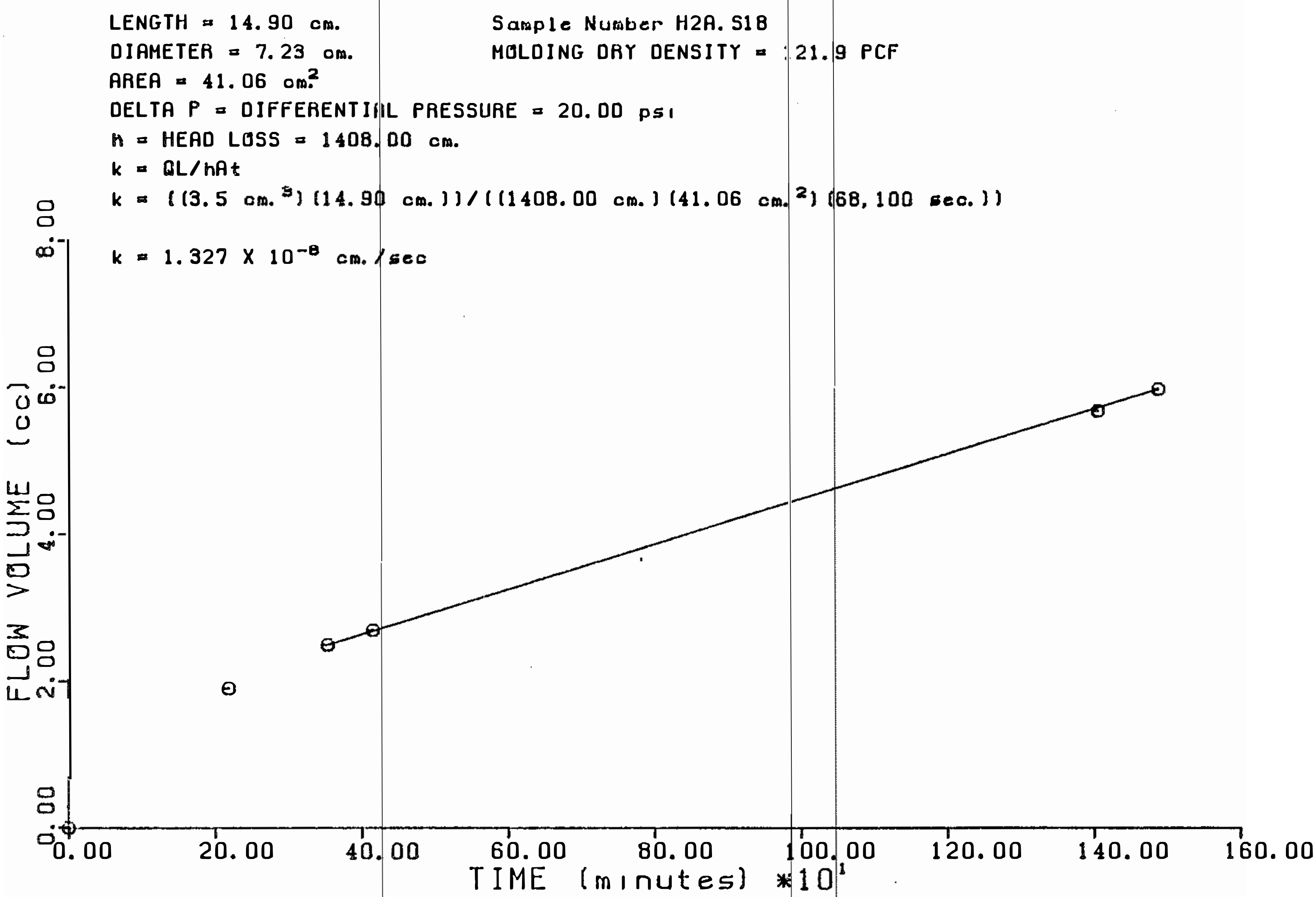


Figure 14. Volume Flow-Time Curve (Hole 2A, Specimen S-1B).

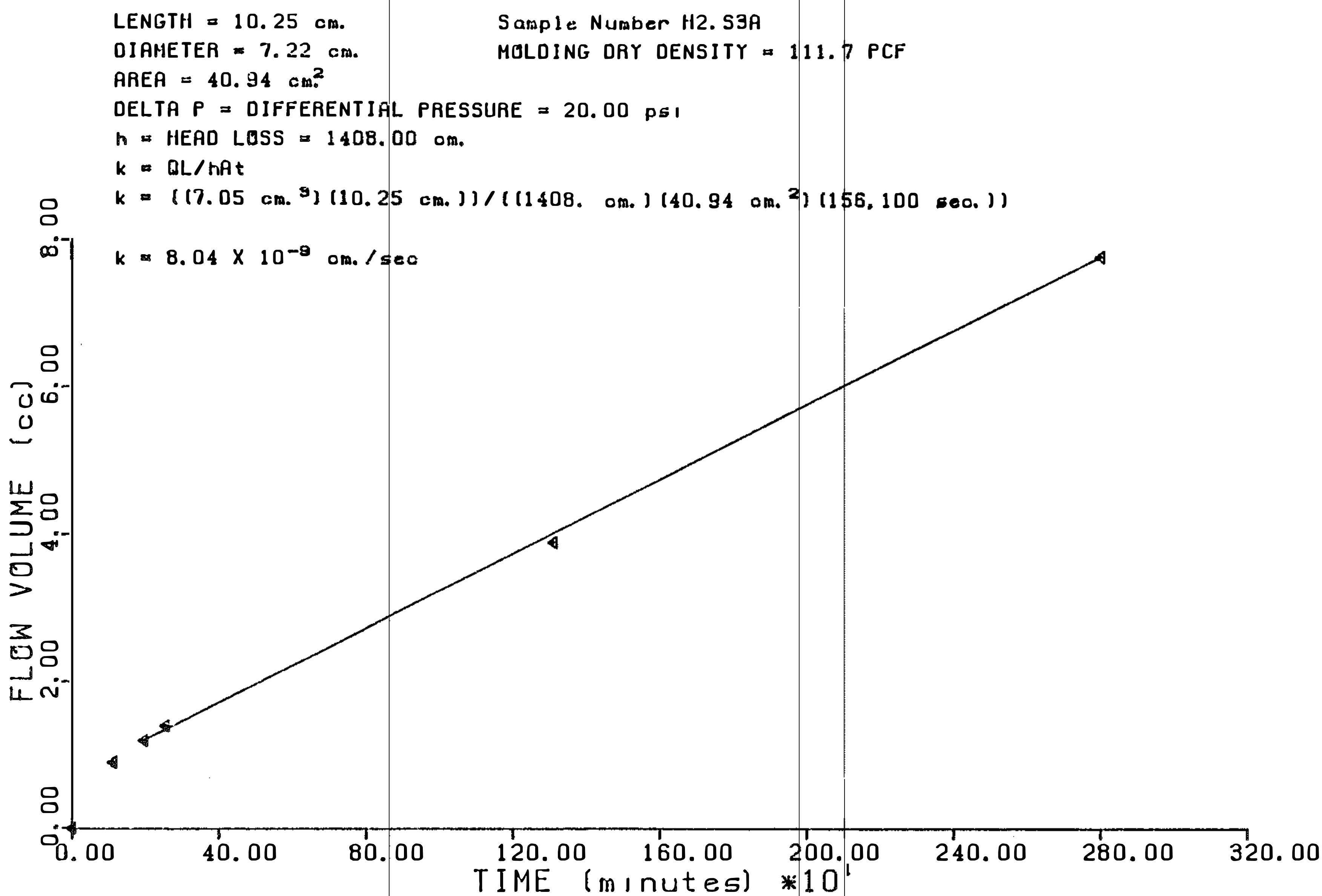


Figure 15. Volume Flow-Time Curve (Hole 2, Specimen S-3A).

respectively. The location of the phreatic line, based on groundwater elevations in Holes 2, 3, and 4, is shown in Figure 6, a typical cross section of the dam at Station 178+00. A sketch of the phreatic surface along centerline axis of the dam is shown in Figure 7. Groundwater elevations at Station 181+00 (sidehill portion of the embankment) are shown in Figure 8.

EMBANKMENT SOILS

Essentially, the dam consists of five zones of materials as shown in Figure 6. Those zones include the earth core, foundation soils, shell materials situated to each side of the earth core, waste disposal berm materials, and the rock abutments and foundation. Transitional filter zones are not shown on the as-built construction cross sections, although they are shown on original record plans.

As shown in Table 1 (Hole 2), soils in the earth core consist of brown clays with some small rock fragments. The core materials classified as CL according to the Unified Soil Classification System. Natural water contents of the soils in the clay core ranged from 12.6 percent to 25.1 percent and averaged about 17.6 percent. Liquid limits ranged from 36 to 37; plastic limits ranged from 18 to 20. As shown in Table 1 (data for Hole 2), the natural water contents of the clay core were slightly less than or equal to the plastic limits of the clay core, indicating that the core soils are overconsolidated. Liquidity indices of the core materials ranged from -0.02 to -0.29. Soils in the core contained a high percentage of clay. The percentage passing the No.-200 sieve ranged from 63 to 78 (based on particle-size analyses of Specimens 6A, 7A, 8A, and 10A from Hole 2). Material passing the No. 4 sieve ranged from about 83 percent to 95 percent. Hence, the soils in the core zone contained some 5 to 17 percent gravel or rock fragments. Values of activity (the ratio of the plasticity index to the percent by weight finer than 0.002 mm) for Specimens 6A, 7A, 8A, and 10A were 0.44, 0.70, 0.42, and 0.49, respectively. Soils having values of activity less than 0.75 are inactive.

Maximum dry density and optimum water content of the soils in the core zone were 111.3 pounds per cubic foot and 17.2 percent, respectively, as shown in Figure 13. Dry densities of specimens used in triaxial tests (6A, 10A, and 8A) were 110.6, 109.1, and 107.5 pounds per cubic foot, respectively. Consequently, values of relative compaction (ratio of in situ dry density to maximum dry density) were 99.5, 98.0, and 96.7 percent, respectively. In situ water contents of triaxial Specimens 6A, 10A, and 8A were 21.1, 20.4, and 21.9 percent, respectively. Water contents obtained during extrusion of the Shelby-tube samples were 16.7, 17.7, and 17.9 percent. Hence, water contents of the clay soils ranged from about 0.5 percent lower than optimum water content to 4.7 percent above. Generally, the in situ water content averaged some 2 percent above optimum water content.

The coefficient of permeability or hydraulic conductivity of the clay core, Figure 15, based on one test (Specimen S-3A from Hole 2) and measured in a vertical direction was 8.04×10^{-9} centimeter per second. Effective stress angle of internal friction and effective cohesion of the soils in the clay core zone were 25.9 degrees and 305 pounds per square foot.

Sampled materials from the rock shell zones of the embankment are represented in Table 1 by Specimens 1A through 3B of Hole 2A, 1A of Hole 2, and 1A and 1B of Hole 3.

The soils in the shell zones can be described as highly weathered green shales and siltstones. Based on the testing of Specimens 1C, 2B, 3A, and 3B from Hole 2A and Specimen 1A from Hole 3, liquid limits ranged from 32 to 37; plastic limits ranged from 19.8 to 21.2. In situ water contents ranged from about 8.7 percent to 19.6 percent (near the top of the dam), averaging 12.6 percent. Soils in the shell zone contained a high percentage of clay-size particles. The percentage of soil particles passing the NO.-200 sieve ranged from 58.4 to 78.6 percent. The percentage of soil particles passing the No.-4 sieve ranged from 77.3 to 96.9 percent. The rock fraction ranged from about 3 percent to 22 percent. In situ dry densities of the soils in the shell zones ranged from 108.1 to 123.2 pounds per cubic foot. Values of activity of the shell materials ranged from 0.47 to 0.59. Samples obtained from the shell zones classified as CL.

The coefficient of permeability, measured in a vertical direction, of soils in the shell zone based on one test (Specimen S-1B from Hole 2A) was 1.33×10^{-8} centimeter per second (Figure 14).

Effective stress angle of internal friction and effective cohesion of the soils in the shell zones were 30.6 degrees and 271.1 pounds per square foot, respectively.

Soils in the waste disposal berm consisted of a matrix of weathered green shale, siltstone, and soil. The waste materials are represented in Table 1 as Specimens 1A, 1B, 2A, 2B, 3A, and 3B from Hole 4 and Specimens 1A, 1B, and 1C from Hole 7. Those soils classified as CL and ML-CL and were similar to soils in the shell zone. Liquid limits of those soils ranged from 29.3 to 34.0. Plastic limits range from 17.9 to 21.8, and natural water contents ranged from 6.2 to 24.9 percent and averaged 16.3 percent. The natural water contents were equal to or somewhat less than the plastic limits of those materials. The materials are overconsolidated. The percentage of particles passing the No.-200 sieve ranged from 66.4 to 85.6. Soil particles larger than the No.-4 sieve ranged from about 2 percent to 18 percent.

Effective stress angle of internal friction and effective cohesion were 28.0 degrees and 573 pounds per square foot. Only two specimens of the berm material were suitable for triaxial testing.

FOUNDATION SOILS

Foundation soils consisted of brown, loose clayey sands and sandy materials. Those soils classified as SM and were nonplastic. They are represented in Table 1 as Specimens 1 and 2 from Hole 3A and Specimen 3C from Hole 3C. Other descriptions of the foundation soils are shown in APPENDIX A, Holes 3A, 3, and 4, Figures A.1, A.3, and A.4, respectively. Based on the few samples of the foundation that could be retrieved, soil particles passing the No.-200 sieve ranged from 16.2 to 47.4 percent. Approximately 7 to 22 percent of the material was gravelly (larger than a No.-4 sieve).

The material was composed mainly of sand- and silt-size particles. The material is apparently alluvium. Values obtained from the standard penetration tests were very low, on the order of 4, as shown in Figure A.3 of APPENDIX A. Based on those low values, the sandy material was in a loose state.

Effective stress internal angle of friction and effective cohesion are estimated to be 33.8 degrees and zero, respectively. Only one triaxial test was used to make that estimate.

ROCK ABUTMENTS AND FOUNDATION ROCK

Materials in the rock abutments and foundation rock strata consisted mainly of siltstone, or very fine-grained sandstone, as shown in Figures 5, 6, A.1, and A.2. The siltstones contained shale streaks interbedded throughout the rock abutments and foundation. Based on detailed coring logs in APPENDIX A, the rock abutments and foundation appeared to be tight with regard to seepage; that is, they were relatively free of joints, cracks, or crevices. Estimated coefficient of permeability of the siltstones is low, perhaps on the order of 1×10^{-6} centimeter per second.

ANALYSIS AND DISCUSSION

EMBANKMENT SOILS

Soils of the type used to construct the core zone of the dam rated relatively high as a core zone material of a zoned earth dam, as shown in an engineering use chart by Wagner (10). On a scale of 1 to 10 (lower number implies the least desirable soil for the intended purpose), soils in the core zone have a rating of 3. Since the core material has a small percentage of gravel, the rating could be slightly higher. Soils in the core have high percentages of clay-size particles and are inactive clays. Liquid limits and plastic limits of the soils were not excessively high. Workability of the core soils was good to fair.

Based on data in Figure 13, soils in the core were compacted near maximum dry density during construction. Relative compaction was 96.7 to 99.5 percent. Since the natural water contents and plastic limits of the soils in the core zone were nearly equal, and considering that the soils have liquidity indices less than 0.4 (11), then the soils are overconsolidated. The material appeared to have been compacted near or slightly wet of optimum moisture content. Clays compacted near or slightly wet of optimum moisture retain a plastic character and will deform without cracking more readily than soils compacted dry of optimum. Based on a visual inspection, no cracking of the dam was observed.

Soils used in the core appeared to have low permeabilities. The coefficient of permeability for a specimen from the core zone was 8.04×10^{-9} centimeter per second. That value is lower than the value of coefficient of permeability -- 1×10^{-7} centimeter per second -- generally accepted as "practically impermeable" (10). Considering the high percentages of clay-size particles of specimens from the clay core and high relative compactive values, the permeability of the clay core is probably very low. Generally, soils that classify as CL, when properly compacted, are impervious. However, the test value of the coefficient of permeability cited above was measured in the vertical direction. Since embankment cores are compacted in thin layers, the potential exists for the permeability in the horizontal direction to be much higher than in the vertical direction. Seepage may occur along the built-in bedding planes between layers, although the use of a sheepsfoot roller does help to some degree to minimize that effect. Assuming the horizontal value of coefficient of permeability is as high as 1×10^{-7} centimeter per second, then the ratio of horizontal coefficient of permeability to the vertical coefficient of permeability is 124. This is a rather large ratio. Normally in the design of an earth core, a minimum ratio of 9 or larger is used. Hence, the ratio as referenced above appears to be sufficiently large and if the horizontal permeability was 1 x 10^{-7} centimeter per second, then the core materials could still be considered to be practically impermeable.

Based on classification tests, the shearing strength of the compacted saturated soils in the earth core zone was fair. Effective stress parameters were 25.9 degrees and 305 pounds per square foot, respectively. Those were relatively low values; however, the values are not exceptionally low for clay cores of a dam.

According to the highway record plan and profile (1) of KY 11 (proposed state highway in 1962), the earth dam was to contain transitional zones (presumably the material would meet filter requirements (10)) located on the upstream and downstream faces of the clay core. Shell zones were located on the upstream and downstream faces of the transitional zones. The plans show the shell zones were to be constructed of rock. However, as-built cross sections do not show the transitional zones. Borings made during this study did not intercept any transitional zones. Apparently, those zones were not constructed.

The shell zones were to be constructed of rock, presumably durable rock. Also, the portion of the shell zone above the clay core was designated as rock. However, as shown in APPENDIX A, Borings 1, 3, 2, and 2A, and in Figure 6, the shell zones contained a matrix of highly weathered shale, siltstone, and soil. Sound durable rock was not found during exploration. Rather, the shell materials classified as CL; the materials contained fairly high percentages of soil particles passing the No.-200 sieve and do not qualify as rock for shell zones of an earth dam of the type constructed at the Mill Creek site. According to Wagner's engineering use chart (10), the relative desirability of the shell material existing at the site was unsuitable and should not have been used as shell material.

Rock materials in shell zones serve at least two purposes: 1) to prevent an excessive rise in the seepage line and the buildup of pore pressures in the downstream slope and 2) to provide stability of the upstream and downstream slopes. Considering the high percentage of clay-size particles in the material of the downstream slope, the downstream shell had very low permeability characteristics. The value of coefficient of permeability obtained from a test on the shell material was only 1.33 x 10^{-8} centimeter per second. To prevent the rise of the seepage line in the downstream slope, the ratio of the coefficient of permeability of the downstream slope to the the coefficient of permeability of the transitional filter material must be extremely large; that is, the shell material must be several hundred times more permeable than the core. However, the permeability ratio (1.33 x 10^{-8} cm/sec/8.04 x 10^{-9} cm/sec) was only 1.7. Consequently, the dam acts, in actuality, as a homogeneous earth dam. Consequently, for this situation, the proper design elements do not exist in the downstream shell to prevent a rise in the seepage line. Had the shell material been highly permeable, then, without the transitional filter, piping in the clay core could have occurred.

Based on results obtained from Boring 2, Figure A.1, and as shown in Figure 6, the clay core trench and keyway were apparently constructed. According to the as-built cross sections, the bottom of the clay keyway at Station 178+00 is at elevation 756.5 feet. Refusal or rock elevation in Boring 2 was at elevation 755.8 feet. Soils in the core trench (Shelby-tube No. 10A) classified as CL. About 78 percent of the soil passed the No.-200 sieve. Soils in the core trench were the same as soils used in the core.

FOUNDATION AND WASTE DISPOSAL BERM SOILS

Based on the classification, visual descriptions, and non-plastic nature of soils in the foundation, the permeability of the foundation soils is several times greater than the soils in the clay core and shells. The relative permeability of the foundation soils located under the downstream slope is estimated to be, perhaps, on the order of 1×10^{-4} to 1×10^{-6} centimeter per second. Because of the silty and sandy nature of the foundation soils, the material is pervious. The soils used to construct the downstream waste disposal berm were similar to the soils used in the shells. The berm materials classified as CL. Permeability of the berm soils was estimated to be several times smaller than the permeability of the foundation soils. According to the as-built plans, a horizontal filter drain was not constructed. To prevent the rise of the seepage line into the downstream slope, a horizontal filter drain should have been located between the foundation and downstream shell zone. Piping potential of the foundation soils is several times greater than the piping potential of the berm materials, since the foundation soils are nonplastic while the berm materials are plastic and possess cohesion. Since a horizontal drain is not present, there is nothing to prevent the rise of the seepage line into the berm.

SEEPAGE

As shown in Figure 6, and based on groundwater elevations obtained from Holes 2, 3, and 4 (see APPENDIX B), Station 178+00, the line of seepage intercepts the downstream slope of the dam in the vicinity of the berm. Seepage (clear) in the vicinity of the berm was observed during visual inspections. Interception of the seepage line with the downstream slope is an undesirable situation because of the potential for piping in the downstream slope. The seepage line shown in Figure 6 was for normal pool. When the lake rises, the seepage line in the dam may rise and intercept the lower ranges of the downstream shell. The factor of safety against heave, or a blowout, in the downstream berm area was estimated to be on the order of 1.4 to 1.8.

During visual inspections of the dam, seepage was observed flowing from several small springs in an area located approximately 80 to 100 feet right of centerline and between Stations 180+00 and 182+00. The water was clear. That area was a sidehill fill; the thickness of the material in that area was shallow, as shown in Figure 8. The source of seepage was probably through the right abutment, since the bedrock was close to the downstream slope in that area.

STABILITY ANALYSIS

Generally, the critical potential failure surface for an earth dam resting on solid rock and composed of rock shells and a central core is wedge shaped because the shear strengths of rock shells and rock foundations are generally much higher than the shear strengths of relatively soft clay cores. However, the shells of Mill Creek Dam were composed of clayey materials, and shear strengths of the shell materials were only slightly higher than the strengths of the core zone. Consequently, circular shear surfaces were also analyzed.

Stability of the dam was investigated for three assumed conditions of loading: 1) steady-state seepage from the service spillway elevation, or normal pool, 2) rapid-drawdown from the normal pool, and 3) earthquake or seismic loading. In the first case, stability of the downstream slope was investigated. In Case 2, stability of the upstream slope was investigated for a rapid, or sudden, drawdown condition. Stabilities of the upstream and downstream slopes were investigated for earthquake or seismic loading conditions (Case 3). Those analyses were performed using a commonly used psuedo-statical approach, although that approach has several shortcomings (15).

Stability of the dam was analyized using two different methods. The ICES-LEASE slope stability computer program (12), based on Bishop's simplified methods of slices (13), and the HOPK-I model (method of slices) developed by Hopkins (14). Two modes of failure were investigated.

The ICES-LEASE program is limited to circular shear surfaces. Moreover, that program does not solve psuedo-statical earthquake problems. The HOPK-I computer program solves problems involving any shear surface configuration and earthquake forces. That program was used in solving all three loading conditions listed above, as well as solving both circular and wedge-type failure modes. The ICES-LEASE program was used to investigate Cases 1 and 2 assuming a circular failure mode.

Stability analyses of the downstream slope assuming a circular failure mode are shown in Table 3. Those analyses were based on the effective stress shear strength parameters in Table 2. The minimum factor of safety obtained from the ICES-LEASE program for the downstream slope of Mill Creek Dam using the observed seepage line was 1.81. A factor of safety of 1.82 was obtained from the HOPK-I program. Critical circular shear surfaces obtained from both programs are compared in Figure 16. The X- and Y-coordinates of the centers of the critical circular shear surfaces obtained from both programs, as well as radii and locations of the critical arcs, were almost identical.

Various wedge-shaped failure configurations of the downstream slope were examined using the HOPK-I computer program. Those analyses are summarized in Table 4. Various two-block and three-block failures were examined, as shown in Figure 17. Factors of safety are shown as a function of horizontal distance in Figure 17. The upper curve in Figure 17 is for shear surfaces abc, abnJ1, abmJ2, etc. while the lower plot is for shear surfaces dec, denJ1, demJ2, etc. The lowest factor of safety obtained from the wedge analysis (shear surface dekJ4 in Figure 17) of the downstream slope was 1.96.

A summary of factors of safety obtained from stability analyses of the upstream slope assuming a sudden or rapid drawdown of the lake from normal pool elevation to some lower pool elevation is given in Table 5. Various positions of the pool elevation were assumed as shown in Figure 18. Both circular and wedge configurations were investigated. A comparison of the factor of safety obtained from the ICES LEASE computer program and the factor of safety obtained from the HOPK-I computer program is shown in Figure 18 and the lower portion of Table 5. Critical arcs obtained from the two programs for the rapid drawdown case assuming pool elevation at 780 feet are also compared in Figure 18. Both programs gave a factor of safety of 1.04 for that case. As shown in Figure 18, factors of safety obtained from those analyses are plotted as a function of pool elevations. In performing rapid drawdown analyses, no drainage was assumed to occur in the

TABLE 3. SUMMARY OF FACTORS OF SAFETY OBTAINED FROM STABILITY ANALYSES OF ASSUMED CIRCULAR CONFIGURATIONS OF DOWNSTREAM SLOPE, STATION 178+00

SHEAR SURFACE CONFIGURATION	FAILURE MODE	SEEPAGE CONDITION AND LOCATION	COMPUTER PROGRAM AND MODEL	COORDINATES OF CRITICAL ARC X (feet)	Y (feet)	R (feet)	FACTOR OF SAFETY*
ab	Circular	Steady State; Seepage Line from Spillway Crest Elevation	ICES-LEASE	100.0	897.7	134.1	1.81
cd	Circular	Same	HOPK-I	95.0	895.0	127.1	1.82

* Factors of Safety were obtained from a Grid-type Search Analysis

TABLE 4. SUMMARY OF FACTORS OF SAFETY OBTAINED FROM STABILITY ANALYSES OF ASSUMED FAILURE WEDGE CONFIGURATIONS OF DOWNSTREAM SLOPE, STATION 178+00

SHEAR SURFACE CONFIGURATION	FAILURE MODE	SEEPAGE CONDITION AND LOCATION	COMPUTER PROGRAM AND MODEL	FACTOR OF SAFETY*	COMMENTS
afgh	Three-Block Wedge	Steady State; Seepage Line from Spillway Crest Elevation	HOPK-I	2.17	Failure Surface through Upstream Shell and Core and Downstream Foundation
dec	Two-Block Wedge	Same	Same	2.34	Failure Surface through Core and Downstream Shell and Berm
denJ1	Three-Block Wedge	Same	Same	2.08	Same
demJ2	Same	Same	Same	2.02	Same
delJ3	Same	Same	Same	1.99	Same
dekJ4	Same	Same	Same	1.96	Same
dejJ5	Same	Same	Same	1.98	Same
delJ2	Same	Same	Same	2.05	Same
abc	Two-Block Wedge	Same	Same	2.59	Failure Surface through Upstream Shell, Core and Downstream Shell and Berm
abnJ1	Three-Block Wedge	Same	Same	2.51	Same
abnJ2	Same	Same	Same	2.46	Same
ablJ3	Same	Same	Same	2.46	Same
abkJ4	Same	Same	Same	2.44	Same
abjJ5	Same	Same	Same	2.42	Same
abiJ6	Same	Same	Same	2.42	Same

* Factors of Safety were obtained from a Grid-type Search Analysis

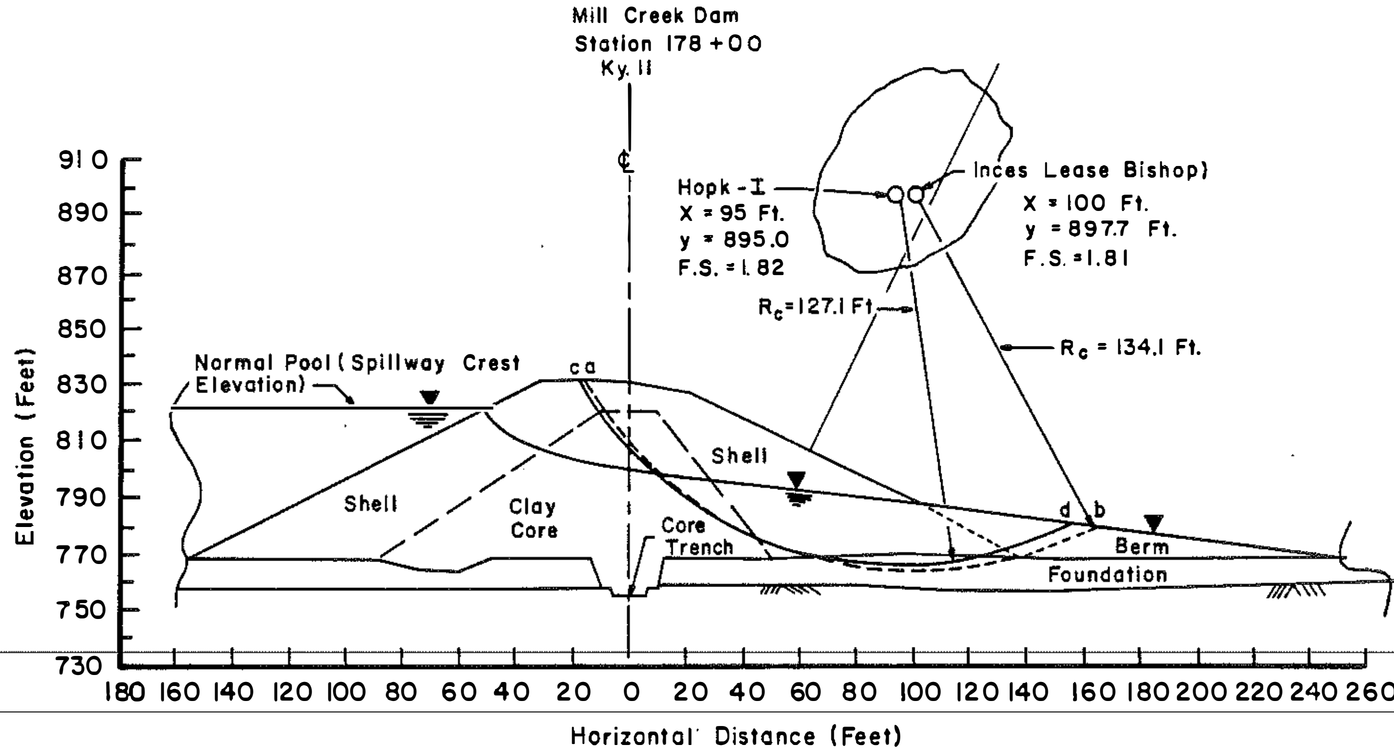


Figure 16. Results of Circular Stability Analyses of the Downstream Slope of Mill Creek Dam, Station 178+00.

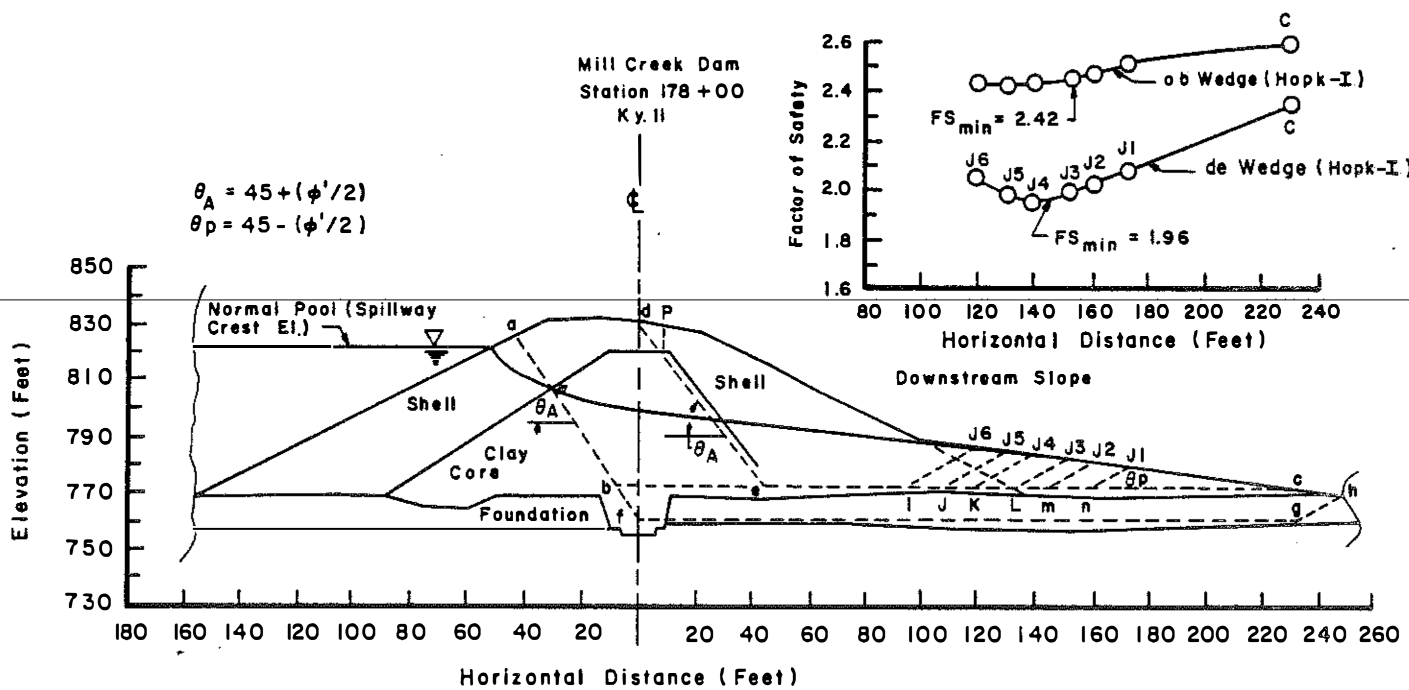


Figure 17. Results of Stability Analyses of the Downstream Slope of Mill Creek Dam, Station 178+00, Assuming Various Wedge-Shaped Configurations.

TABLE 5. SUMMARY OF FACTORS OF SAFETY OBTAINED FROM STABILITY ANALYSES OF UPSTREAM SLOPE ASSUMING WEDGE AND CIRCULAR FAILURE CONFIGURATIONS AND VARIOUS CONDITIONS OF RAPID DRAWDOWN, STATION 178+00

SHEAR SURFACE CONFIGURATION	FAILURE MODE	SEEPAGE CONDITION	COMPUTER PROGRAM AND MODEL	COORDINATES OF CRITICAL ARC X (feet)	Y (feet)	R (feet)	POOL ELEVATION	FACTOR OF SAFETY
PST	Wedge	Normal Pool	HOPK-I	-----	-----	-----	820.5	2.04
PST	Wedge	Rapid Drawdown from Normal Pool	Same	-----	-----	-----	810.0	1.73
PST	Wedge	Same	Same	-----	-----	-----	800.0	1.50
PST	Wedge	Same	Same	-----	-----	-----	780.0	1.26
PXYZ	Wedge	Normal Pool	Same	-----	-----	-----	820.5	2.14
PXYZ	Wedge	Rapid Drawdown from Normal Pool	Same	-----	-----	-----	810.0	1.81
PXYZ	Wedge	Same	Same	-----	-----	-----	800.0	1.55
PXYZ	Wedge	Same	Same	-----	-----	-----	780.0	1.22
----	Circular	Normal Pool	Same	150	975	217.1	820.5	1.84
----	Circular	Rapid Drawdown from Normal Pool	Same	150	970	212.1	810.0	1.58
----	Circular	Same	Same	125	900	142.3	800.0	1.35
gh	Circular	Same	Same	148	908	143.2	780.0	1.04
ef	Circular	Same	ICES-LEASE	140	920	157.1	780.0	1.04

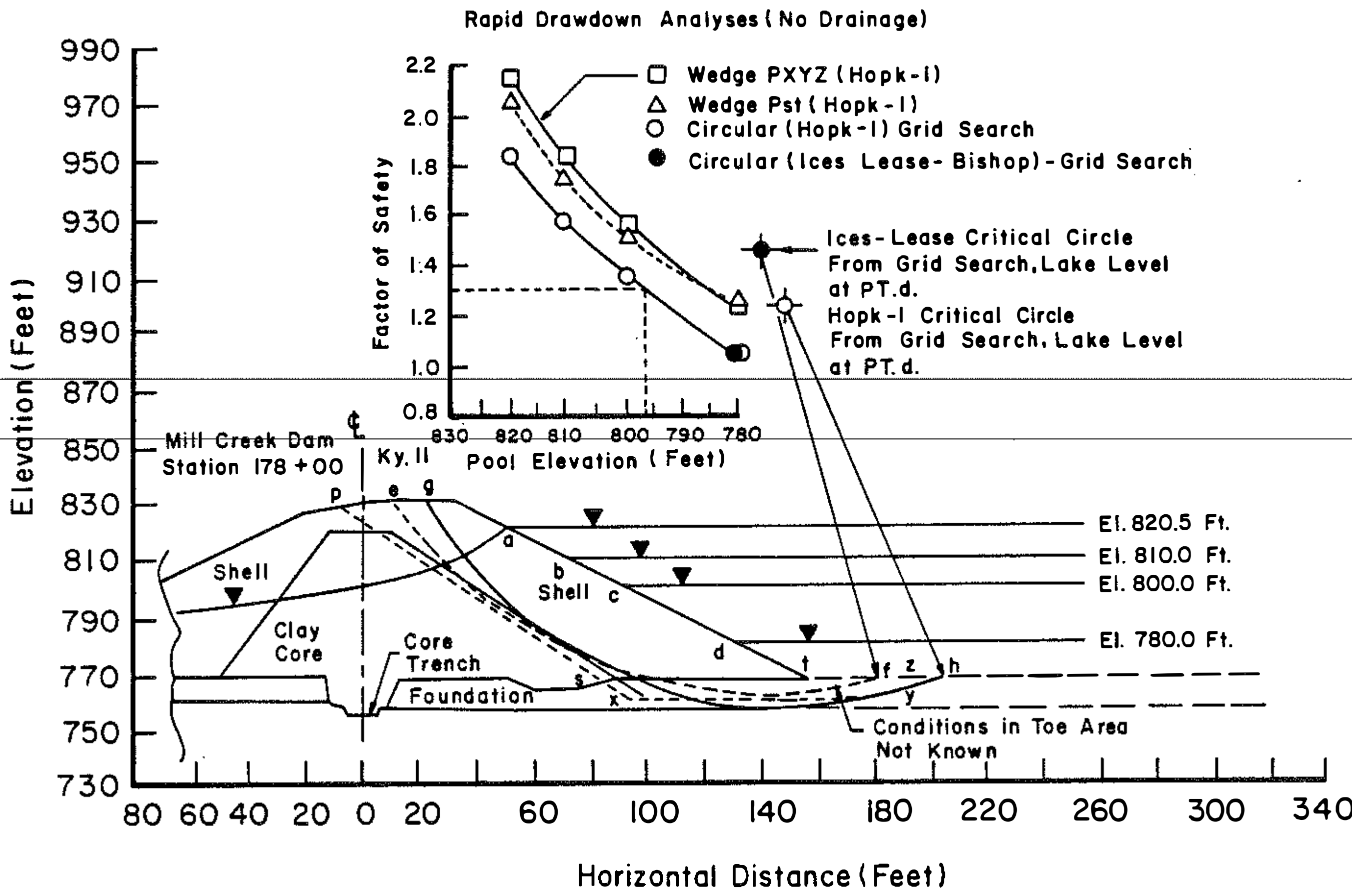


Figure 18. Results of a Rapid Drawdown Stability Analysis of the Upstream Slope of Mill Creek Dam, Station 178+00, Assuming Various Wedge-Shaped Failure Configurations, Circular Failure Configurations, and Various Pool Elevations.

upstream slope as the pool elevation of the lake was lowered. Such an assumption was based on the fact that test results strongly indicated the permeability of the upstream shell material was very low. Additionally, an estimation of the lowering of the seepage lines in the upstream shell, based on a method discussed elsewhere (16), showed the seepage line could not be lowered in a reasonable time period; that is, several months, even years, would be required. Analyses, as shown in APPENDIX C, were based on the coefficient of permeability of 1.88×10^{-8} centimeter per second. Consequently, to prevent a failure of the upstream slope in the event the lake must be drained, or lowered, extreme care must be exercised since a failure would affect the roadway as well as the safety of the dam. The factor of safety during drawdown should not be lower than 1.30. As shown in Figure 18, a factor of safety of 1.30 corresponds to a pool elevation of about 796 feet; that is, the pool elevation should not drop below this elevation during drawdown unless the seepage line in the upstream slope falls during drawdown. Since the permeability in the upstream slope may be larger than the permeability obtained from laboratory tests, and considering the uncertainty of permeability measurements, two observation wells should be installed in the upstream slope to observe the fall of the seepage line in the event the lake is lowered. If the seepage line falls much faster than indicated by the analysis in APPENDIX C, then the lake could be lowered to an elevation lower than 796 feet. To determine lower safe pool levels, stability analyses should be performed using the observed seepage line during rapid drawdown.

The ability of the upstream and downstream slopes to withstand earthquake, or seismic, forces was investigated using a psuedo-statical approach. In that method, which is a traditional approach, the seismic force is assumed to act horizontally at the centroid of each slice in the direction of failure. The force acting on each slice is computed from the equation

$$F = W\, a/g = \psi W,$$

in which

- F = horizontal seismic force,
- W = weight of sliding slice mass,
- g = acceleration of gravity,
- a = horizontal earthquake accerleration,
- ψ = seismic coefficient based on the degree of seismic activity in the region in which a dam is located.

Based on a seismic map published elsewhere (17), Mill Creek Dam is located in Seismic Zone 1 with a seismic coefficient of 0.05. Based on that map, Mill Creek Dam would not be subjected to a strong earthquake. However, an earthquake did occur on July 27, 1980, in the general region of the dam. The earthquake measured about 5.1 on the Richter Scale. No damage to the dam was reported.

Factors of safety obtained from earthquake analyses, assuming a shear-type failure occurs, for the upstream and downstream slopes are summarized in Tables 6 and 7. Factors of safety obtained from those analyses are plotted as a function of seismic coefficient in Figure 19. Both wedge and circular failure configurations were investigated. Critical arcs corresponding to a seismic coefficient of 0.10 are shown in Figure 19. Various seismic coefficients were assumed. As shown in the upper left-hand plot in Figure 19, a factor of safety of 1.0 corresponds to a seismic coefficient of about 0.095. At a safety factor of 1.0, the seismic coefficient for the

TABLE 6. SUMMARY OF FACTORS OF SAFETY OBTAINED FROM PSUEDO-STATICAL EARTHQUAKE STABILITY ANALYSES ASSUMING WEDGE AND CIRCULAR FAILURE CONFIGURATIONS AND VARIOUS SEISMIC COEFFICIENTS, UPSTREAM SLOPE, STATION 178+00

SHEAR SURFACE CONFIGURATION	FAILURE MODE	SEEPAGE CONDITION	COMPUTER PROGRAM AND MODEL	COORDINATES OF CRITICAL ARC			ASSUMED SEISMIC COEFFICIENT	FACTOR OF SAFETY
				X (feet)	Y (feet)	R (feet)		
PST	Wedge	Steady Seepage from Normal Pool	HOPK-I	-----	-----	-----	0.00	2.04
PST	Wedge	Same	Same	-----	-----	-----	0.05	1.55
PST	Wedge	Same	Same	-----	-----	-----	0.10	1.23
PST	Wedge	Same	Same	-----	-----	-----	0.15	1.02
PST	Wedge	Same	Same	-----	-----	-----	0.20	0.86
PXYZ	Wedge	Same	Same	-----	-----	-----	0.00	2.14
PXYZ	Wedge	Same	Same	-----	-----	-----	0.05	1.51
PXYZ	Wedge	Same	Same	-----	-----	-----	0.10	1.15
PXYZ	Wedge	Same	Same	-----	-----	-----	0.15	0.92
PXYZ	Wedge	Same	Same	-----	-----	-----	0.20	0.76
----	Circular	Same	Same	150	975	217.09	0.00	1.84
----	Circular	Same	Same	150	910	202.10	0.05	1.39
at	Circular	Same	Same	175	945	180.76	0.10	0.97

TABLE 7. SUMMARY OF FACTORS OF SAFETY OBTAINED FROM PSUEDO-STATICAL EARTHQUAKE STABILITY ANALYSES ASSUMING WEDGE AND CIRCULAR FAILURE CONFIGURATIONS AND VARIOUS SEISMIC COEFFICIENTS, DOWNSTREAM SLOPE, STATION 178+00

SHEAR SURFACE CONFIGURATION	FAILURE MODE	SEEPAGE CONDITION	COMPUTER PROGRAM AND MODEL	COORDINATES OF CRITICAL ARC			ASSUMED SEISMIC COEFFICIENT	FACTOR OF SAFETY
				X (feet)	Y (feet)	R (feet)		
deckJ4	Wedge	Steady Seepage from Normal Pool	HOPK-I	-----	-----	-----	0.00	1.96
deckJ4	Wedge	Same	Same	-----	-----	-----	0.05	1.73
~~deckJ4~~	~~Wedge~~	~~Same~~	~~Same~~	~~-----~~	~~-----~~	~~-----~~	~~0.10~~	~~1.46~~
~~deckJ4~~	~~Wedge~~	~~Same~~	~~Same~~	~~-----~~	~~-----~~	~~-----~~	~~0.15~~	~~1.31~~
deckJ4	Wedge	Same	Same	-----	-----	-----	0.20	1.18
cd	Circular	Same	ICES-LEASE	95	895	127.1	0.00	1.81
ab	Circular	Same	Same	100	897.7	134.1	0.00	1.82
--	Circular	Same	HOPK-I	110	930	163.0	0.05	1.53
uv	Circular	Same	Same	160	910	153.0	0.10	1.31

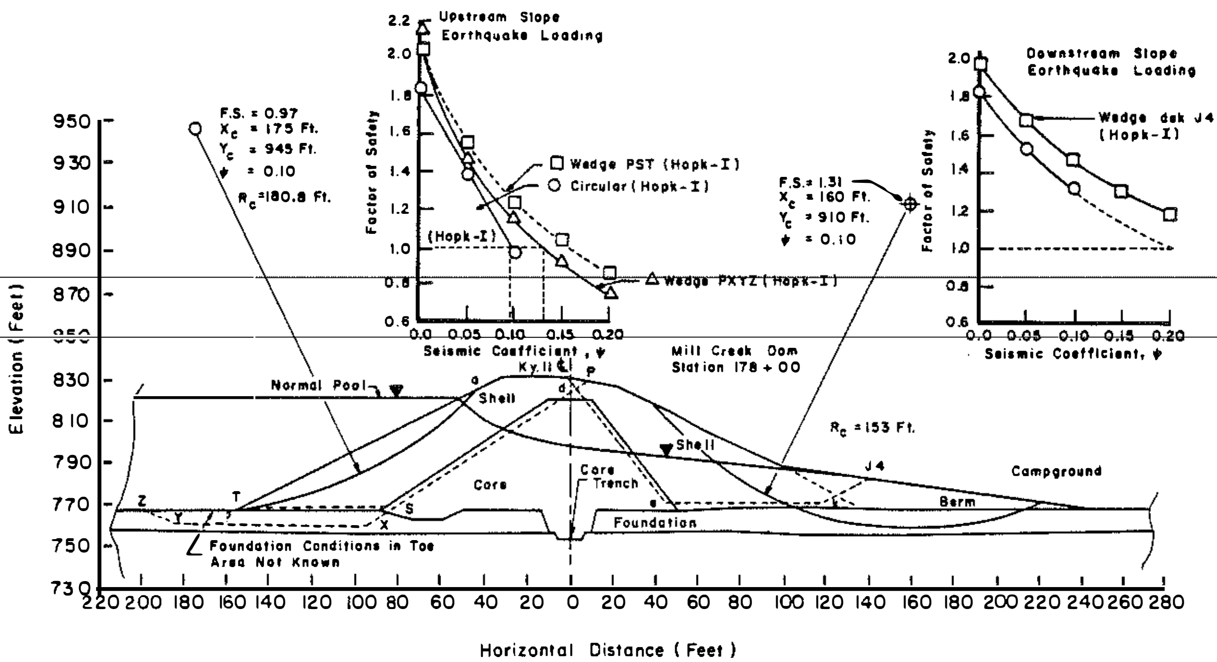


Figure 19. Stability Analyses of Upstream and Downstream Slopes of Mill Creek Dam, Assuming Various Earthquake Loadings.

downstream slope is greater than 0.15 and close (based on a projection of the curve) to about 0.20. Hence, for both slopes, the seismic coefficient is greater than 0.05 -- the seismic coefficient for Zone 1. Consequently, the dam could withstand fairly large earthquake forces with regard to a shear-type failure.

During an earthquake, two modes of failure may occur. The first takes the form of a shear-type movement. That particular failure mode was analyzed above. The second mode of failure may occur due to liquefaction. The liquefaction potential of an earth dam subjected to earthquake motion depends on the characteristics of the soil, relative density or void ratio, initial confining stress, intensity of ground shaking, and duration of ground shaking (18, 19). An investigation of liquefaction potential of the dam requires specialized testing and analysis. Such an analysis is beyond the scope of this study. However, a few comments concerning liquefaction potential of soils located at the site are given below.

Generally, cohesionless soils are more susceptible to liquefaction when subjected to earthquake motion than soils having cohesion. From that viewpoint, the foundation soils would be more likely to liquefy than the soils located in the core, shells, and berm. The later materials were clays and have cohesion. Based on standard penetration tests, the foundation materials are in a fairly loose state. In areas where the berm thickness is shallow, mainly at the toe of the berm, the foundation soils may liquefy under a moderately strong earthquake. Liquefaction of the foundation soils in the toe area of the berm could potentially lead to failure of the dam during a moderately strong earthquake.

HYDROLOGIC ANALYSIS

The Phase I inspection report (2) recommended an additional hydrological study to determine the feasibility of increasing the capacity of the emergency spillway to safely discharge the probable maximum flood (PMF). As reported in the Phase I study, the existing spillway has the capacity to pass 19 percent of the PMF. The report included a Dams 2 computer program printout for the structure using a Class A (low hazard) design. Input data for the program included probable maximum precipitation PMP = 27 inches and curve number CN = 80. Program output apparently indicated the maximum pool elevation (830.8 feet) would occur at rainfall P = 5.1 inches with the resultant spillway discharge being 1,573.6 cubic feet per second (cfs). The 5.1 inches of rainfall is approximately 19 percent of 27.0 inches.

In accordance with current guidelines (3), the dam is a Class C (high hazard) structure since failure could cause probable loss of human life. There is a campground immediately at the toe of the facility. Additionally, there is a second dam about 4,000 feet downstream and beyond that there are recreational areas and several small businesses and homes. Failure of the dam also could lead to loss of life of occupants of vehicles travelling KY 11.

The following project data were obtained from the Phase I inspection report and files at both the Kentucky Transportation Cabinet and the Kentucky Natural Resources and Environmental Protection Cabinet.

Basic data were input into the Dams 2 computer program (20) with Class C designation, PMP (probable maximum precipitation) (21) = 28.2 inches, and CN = 80. The output listed the resultant runoff as Q (number of inches of runoff from

PMP) = 25.4 inches and V (total volume resulting from PMP storm) = 7,231.5 acre-feet. A peak inflow of 39,083.9 cfs was listed at 3.07 hours. A spillway discharge of 20,434 cfs was noted at a head of 44.2 feet or pool elevation of 865.3 feet (fictitious values since top of dam is 830.8 feet). The spillway capacity of 1,573.6 cfs at maximum pool elevation was verified. Lake storage between normal pool and maximum pool would be 444 acre-feet.

The existing spillway capacity is on the order of 4 percent of the probable peak inflow and approximately 11 percent of the overall average 6-hour runoff. The spillway discharges to the drop structure, through the box culvert, and then into the natural channel. Full-flow outlet velocity of the box culvert at a discharge of 1,573.6 cfs would be 16.39 feet per second. At part-full flow for that discharge, the outlet velocity would probably exceed 20 feet per second. The culvert exits at a rock (or shale) cascade to the natural channel; therefore, an outlet velocity exceeding 6 to 7 feet per second could be potentially erodable. Naturally, the box culvert could not possibly transmit the peak discharge and overtopping of the dam would be eminent (4 to 6 feet deep for full dam length).

CONCLUSIONS

Based on laboratory and field data and the analyses presented herein, the following conclusions are made concerning Mill Creek Dam:

1. Factors of safety against a shear failure obtained from stability analyses of the upstream and downstream slopes, assuming steady-state seepage from normal pool, were 1.84 and 1.81, respectively. Those values exceeded the recommended value of 1.5. All safety factors were determined from effective stress analyses.

2. For the case of rapid drawdown and in the event the lake must be lowered, the pool elevation may be drawn down to about elevation 796 feet; that elevation corresponds to a recommended safety factor of 1.3. Drawdown below that level should be conducted at a rate that does not exceed a difference in elevation between the upstream seepage line and pool elevation of about 20 to 24 feet. A value smaller than 24 feet is preferred. That difference should be measured in the vicinity of the intersection of normal pool and the upstream slope. Two observation wells should be installed in the upstream slope to observe the seepage line during drawdown; actual seepage lines should be used in a stability analysis to ascertain the safety of the upstream slope.

3. Factors of safety against a shear failure when the earth dam is subjected to various degrees of earthquake motion indicated the dam could withstand a moderate to high earthquake. For a factor of safety of 1.0, the corresponding seismic coefficients for the upstream and downstream slopes were 0.095 and about 0.20, respectively. The earth dam is located in Seismic Zone 1, which has a design seismic coefficient of 0.05. Hence, the seismic coefficients of 0.095 and 0.20 exceed the design coefficient. For a seismic coefficient of 0.05, corresponding factors of safety of the upstream and downstream slopes were 1.39 and 1.53, respectively.

4. Although the dam appeared to be capable of withstanding a moderate to high earthquake with regard to a shear-type failure, the earth dam when subjected to large earthquake motions could fail due to liquefaction of the foundation soils. Foundation soils were essentially cohesionless and had a high liquefaction potential. Those soils

appeared to be in a loose state. Because of the small thickness of cover in the vicinity of the berm toe, foundation soils in that area would probably liquefy first. That could induce failure of the dam. A detailed laboratory study to determine the liquefaction potential of the soils (mainly the sandy soils of the foundation) was considered to be beyond the scope of this study.

5. Although rock was originally designated for the shells of the dam, materials that presently exist in the shells are soil-like and weathered. Transitional filter zones apparently were not built nor was a toe drain constructed at the site. Consequently, the earth dam essentially performs as a homogenous dam since the properties of the materials in the shells are similar to those of the soils of the core. The ratio of the coefficient of permeability of the downstream shell to the coefficient of permeability of the clay core was less than 2.0. For the downstream shell to function properly, the ratio should have been on the order of several hundred. Consequently, measures to prevent the rise of the seepage line into the downstream shell do not exist at the site. The high seepage line in the downstream shell and berm represents an undesirable situation, and the potential for piping exists. Moreover, sandy materials of the foundation also are susceptible to piping. Fortunately, the downstream shell and berm, which are composed of clayey soils, have low susceptibilities to piping.

6. Had the shells been constructed of sound durable rock containing a small percentage of fines, there could have been a continous flow of water across the top of the clay core since normal pool elevation is slightly higher than the elevation of the top of the clay core. The low permeability of the upstream shell prevented, or at least minimized, that situation.

7. Based on results of Boring 2 and as shown on the as-built record plans, the core trench and keyway -- a seepage-reducing measure -- were apparently constructed at the site.

8. The core of the dam was constructed of clay materials that have very low permeabilities. Those materials were compacted very well. The soils were suitable for the clay core zone of the dam.

9. Based on visual inspections of rock core specimens recovered from the site, the abutments are composed of low to practically-impervious rocks. The abutments contain very few joints, or cracks, and consist of cemented siltstones, or very fine-grained sandstones, and shales. Flow through the abutments is most likely nominal.

10. Seepage observed in the vicinity of the right flank of the dam -- the sidehill portion of the embankment -- is probably a result of some flow through or at the right abutment, since the rock line in that area was very close to the slope of the downstream embankment slope. Considering that water was lost in some of the borings in that area, there are probably some zones of materials in the sidehill embankments that serve as conduits for runoff from the roadway. Hence, part of the seepage may be the result of accumulated runoff in the sidehill fill material.

11. There was evidence that pool elevation was near or at the top of the dam. Elevation of water marks inside a boathouse at the site and eyewitness accounts indicate that to be the case.

12. Based on a field inspection, an emergency drain, as shown on record plans, was not constructed. The highway design plans specified that a 12-inch pipe was to be located at the toe of the embankment (waste berm was not

shown on plans) and was to contain a gate valve. That valve was to be housed in a manhole.

13. Based on analyses of the hydrology of the site using the DAMS 2 (20) computer program, the dam and the service spillway at the site are inadequate. Data show that for a PMP of 28.2 inches of rainfall, overtopping of the dam would occur; the dam would be overtopped by 4 to 6 feet of the full length of the dam.

14. Existing spillway capacity is on the order of only 4 percent of the probable peak inflow and approximately 11 percent of the overall average 6-hour runoff.

15. Based on present design criteria, and considering the potential for loss of life and property, the Mill Creek facility is unsafe.

RECOMMENDATIONS AND DISCUSSION

No practical means of increasing discharge capacities of the existing facilities to accommodate a maximum expected discharge appear evident. Ratios of the lake areas at normal pool and maximum pool elevations to total drainage area are 85 and 68, respectively. Lake storage and discharge capacities are only nominal in comparison to the maximum probable runoff. In a sense, the structure is unique since it serves as an impoundment facility as well as the embankment for KY 11. Maintenance of traffic through the area during corrective or remedial operations may be a requirement. Transportation Cabinet officials should be consulted in regard to that possibility. Other factors to be considered include the campground at the downstream toe and the Department of Parks' domestic water supply. Costs of abandonment, relocation, or preservation of those facilities should be considered in development of an overall corrective plan.

Possible solutions for increasing the safety of the dam to acceptable standards are summarized in Table 8. Plan 1 includes depletion of the lake and breachment of the dam. In the event of breachment, construction of either a bridge and approach embankments or culvert and embankment would be necessary in lieu of relocation of KY 11. In the absence of impoundment, the bridge or culvert would be designed hydraulically to pass runoff resulting from a 50-year or 25-year storm, respectively, and then checked for the 100-year storm. Construction of a culvert would probably be the least expensive alternative. The campground might be preserved but the present source of domestic water supply would be lost. The small lake downstream might serve as a potential new source for domestic water supply.

If the lake is to be preserved, use of a multibarrel culvert installation might be investigated (Plan 2). The structure(s) could be installed along the top of the dam with a new roadway placed above the conduit(s). It would be necessary to widen the embankment on the downstream side if the existing spillway is to be maintained. To control downstream seepage, a drainage blanket and a small berm would be required. In addition, paving, rip rap, or flumes would be required on the embankment slope. Abandonment of the campground probably would be necessary; however, the domestic water supply could be preserved.

If the required area of the multibarrel culverts is impractical or too large, consideration could be given to changing the drainage characteristics of the basin. A series of check rock dams could be constructed in the basin that would change the effective area of the drainage basin. Those dams would

TABLE 8. ALTERNATIVE SOLUTIONS FOR INCREASING THE SAFETY OF MILL CREEK DAM TO ACCEPTABLE STANDARDS

PLAN	GENERAL DESCRIPTION OF PLAN	TASK REQUIRED TO IMPLEMENT
1	Drain Lake and Breach Dam	1. Drain lake; install two water observation wells in upstream slope; observe drop in top seepage line; perform stability analyses during drawdown to insure safety of upstream slope and highway. 2. Excavate embankment approximately between Stations 176+00 to 180+00. 3. Construct culvert or bridge; requires design. ~~embankment between abov~~ stations. 5. Traffic control required; detour must be constructed.
2	Construct Multibarrel Culverts on Top of Existing Dam and Constuct Spillway on the Face of Dam	1. Construct multibarrel culvert or bridge. Requires design and stability analysis. May have to partially drain lake. 2. Construct drainage blanket and berm on the face of dam to control downstream seepage. 3. Construct spillway on face of dam. 4. Design and construct new approaches. 5. Traffic control required, detour may be required. 6. If the required area of the multi-barrel culvert or spillway is impractical, then consider changing the drainage characteristics of the drainage basin. Consider constructing a series of rock check dams or small retention dams in the basin. Requires detailed hydrological and hydraulic studies of the basin.

TABLE 8. Continued

PLAN	GENERAL DESCRIPTION OF PLAN	TASK REQUIRED TO IMPLEMENT
3	Construct New Emergency Spillway Parallel to Existing Service Spillway	1. Rock cut required; approximate length of 800 feet. 2. Spillway must cross KY 11. Excavate embankment and construct spillway. 3. Construct multibarrel culvert or bridge. 4. Traffic control required; detour must be constructed. 5. Construct drainage blanket on face of dam and berm to control seepage.
4	Consturct Additional Dam Upstream	1. Locate borrow pit; field exploration; laboratory testing. 2. Field exploration of new site. 3. Design and construct new dam and spillway.

retain water during large precipitation, but would drain and essentially remain empty most of the time. Rock for those dams could be obtained from the Newman Limestone located in the basin. Alternately, small check dams could be constructed with a drainage pipe that would slowly drain the pools behind the check dams. To design the check dams and determine the number of dams required, a detailed hydrologic and hydraulic study would be required.

The third alternative consists of constructing a new emergency spillway parallel to the existing spillway; however, the spillway must cross KY 11. Excavation of the roadway and embankment and construction of multibarrel culverts or a bridge would be required. Plan 3 would require a detour for traffic control. Additionally, to control downstream seepage, a new drainage blanket and berm would be required.

In the event current guidelines or existing policy do not preclude it, construction of a second dam (Plan 4) immediately upstream of the reach of the existing lake might prove a viable alternative. That dam could be designed and constructed to impound water only at times when the existing lake rises to some designated elevation. Thereby, the drainage area for the existing structure would be approximately 500 acres and required modifications to the existing spillway and other discharge facilities may be more realistic. The second lake would then gradually be drained upon passage of flooding rainfalls.

Although Plan 1 may be the least expensive of the solutions shown in Table 8, Plan 2 may be more desirable because the lake could continue to be used for recreational and water-supply purposes. From a cost viewpoint, Plan 2 would not be as expensive as Plans 3 and 4. To avoid large approach grade changes in KY 11, the bottoms of the multibarrel culverts could be located approximately at normal pool or perhaps 2 or 3 feet above normal pool. Approximately 8 to 10 feet of the top of the present embankment would be excavated, using the material as an approach.

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APPENDIX A

DETAILED DESCRIPTIONS OF SOIL AND ROCK CORE BORINGS

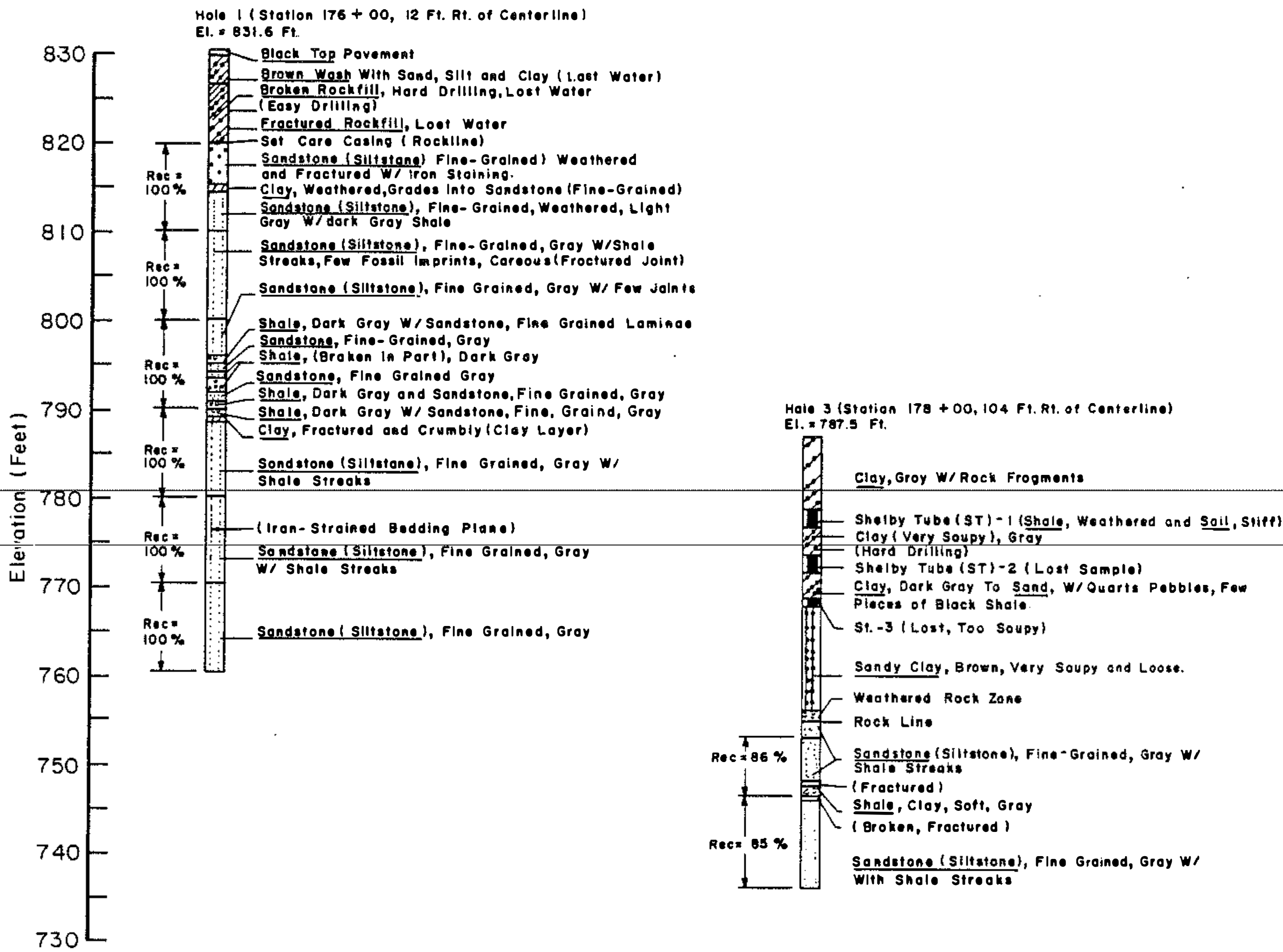
Hole 1 (Station 176 + 00, 12 Ft. Rt. of Centerline)
El. = 831.6 Ft.
Black Top Pavement
Brown Wash With Sand, Silt and Clay (Lost Water)
Broken Rockfill, Hard Drilling, Lost Water
(Easy Drilling)
Fractured Rockfill, Lost Water
Set Core Casing (Rockline)
Sandstone (Siltstone) Fine-Grained) Weathered and Fractured W/ Iron Staining.
Clay, Weathered, Grades Into Sandstone (Fine-Grained)
Sandstone (Siltstone), Fine-Grained, Weathered, Light Gray W/dark Gray Shale
Sandstone (Siltstone), Fine-Grained, Gray W/Shale Streaks, Few Fossil Imprints, Careous (Fractured Joint)
Sandstone (Siltstone), Fine Grained, Gray W/ Few Joints
Shale, Dark Gray W/Sandstone, Fine Grained Laminae
Sandstone, Fine-Grained, Gray
Shale, (Broken In Part), Dark Gray
Sandstone, Fine Grained Gray
Shale, Dark Gray and Sandstone, Fine Grained, Gray
Shale, Dark Gray W/ Sandstone, Fine, Graind, Gray
Clay, Fractured and Crumbly (Clay Layer)
Sandstone (Siltstone), Fine Grained, Gray W/ Shale Streaks
(Iron-Strained Bedding Plane)
Sandstone (Siltstone), Fine Grained, Gray W/ Shale Streaks
Sandstone (Siltstone), Fine Grained, Gray
Rec = 100 %
Elevation (Feet)
830
820
810
800
790
780
770
760
750
740
730
Hole 3 (Station 178 + 00, 104 Ft. Rt. of Centerline)
El. = 787.5 Ft.
Clay, Gray W/ Rock Fragments
Shelby Tube (ST)-1 (Shale, Weathered and Soil, Stiff)
Clay (Very Soupy), Gray
(Hard Drilling)
Shelby Tube (ST)-2 (Lost Sample)
Clay, Dark Gray To Sand, W/ Quarts Pebbles, Few Pieces of Black Shale
St.-3 (Lost, Too Soupy)
Sandy Clay, Brown, Very Soupy and Loose.
Weathered Rock Zone
Rock Line
Rec = 86 %
Sandstone (Siltstone), Fine-Grained, Gray W/ Shale Streaks
(Fractured)
Shale, Clay, Soft, Gray
(Broken, Fractured)
Rec = 85 %
Sandstone (Siltstone), Fine Grained, Gray W/ With Shale Streaks

Figure A.1

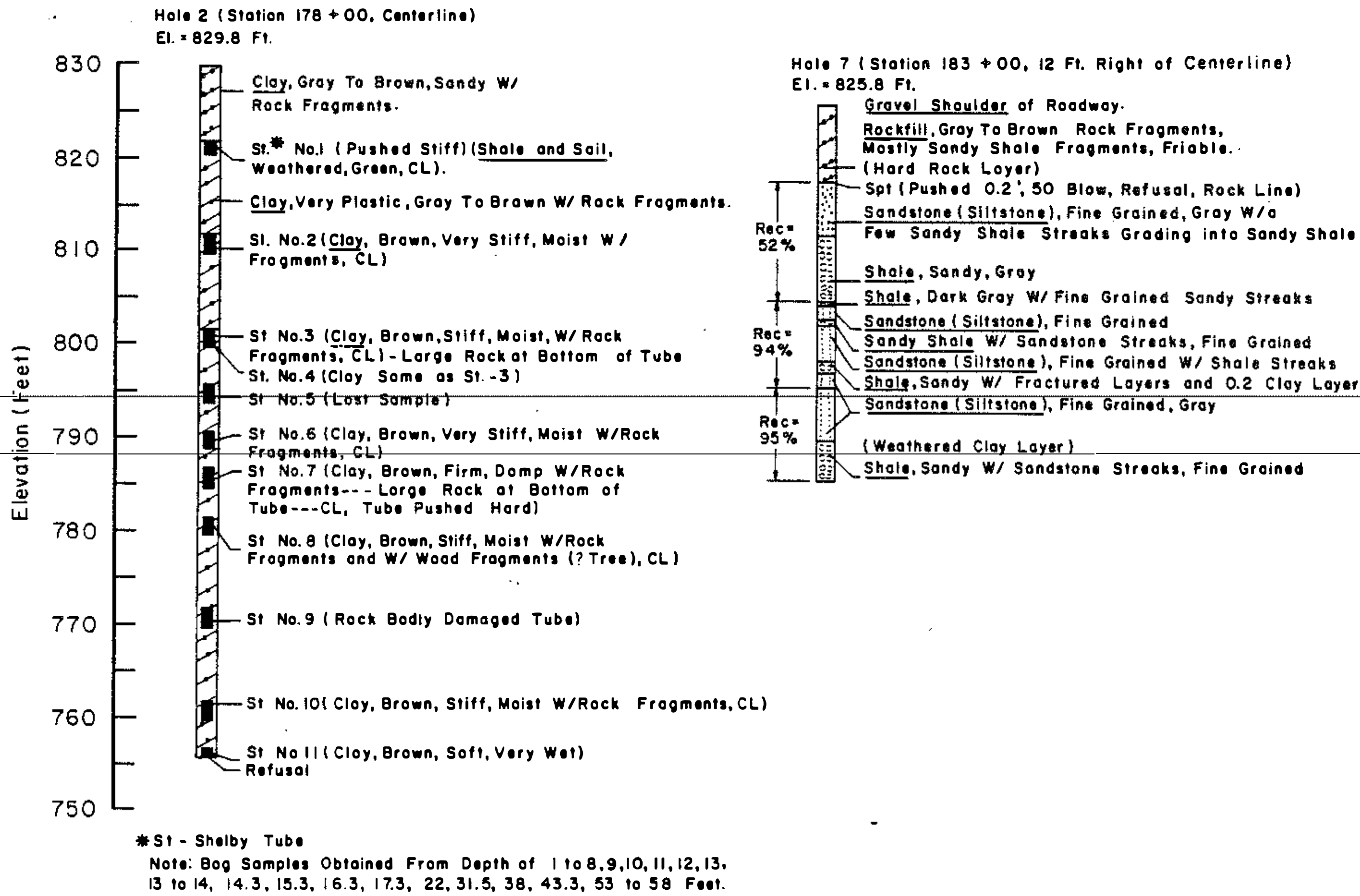
Hole 2 (Station 178 + 00, Centerline)
El. = 829.8 Ft.
830
820
810
800
790
780
770
760
750
Elevation (Feet)
Clay, Gray To Brown, Sandy W/ Rock Fragments.
St.* No.1 (Pushed Stiff) (Shale and Soil, Weathered, Green, CL).
Clay, Very Plastic, Gray To Brown W/ Rock Fragments.
St. No.2 (Clay, Brown, Very Stiff, Moist W/ Fragments, CL)
St No.3 (Clay, Brown, Stiff, Moist, W/ Rock Fragments, CL) - Large Rock at Bottom of Tube
St. No.4 (Clay Same as St.-3)
St No.5 (Lost Sample)
St No.6 (Clay, Brown, Very Stiff, Moist W/Rock Fragments, CL)
St No.7 (Clay, Brown, Firm, Damp W/Rock Fragments--- Large Rock at Bottom of Tube---CL, Tube Pushed Hard)
St No.8 (Clay, Brown, Stiff, Moist W/Rock Fragments and W/ Wood Fragments (? Tree), CL)
St No.9 (Rock Badly Damaged Tube)
St No.10 (Clay, Brown, Stiff, Moist W/Rock Fragments, CL)
St No 11 (Clay, Brown, Soft, Very Wet)
Refusal
*St - Shelby Tube
Note: Bag Samples Obtained From Depth of 1 to 8, 9, 10, 11, 12, 13, 13 to 14, 14.3, 15.3, 16.3, 17.3, 22, 31.5, 38, 43.3, 53 to 58 Feet.
Hole 7 (Station 183 + 00, 12 Ft. Right of Centerline)
El. = 825.8 Ft.
Gravel Shoulder of Roadway.
Rockfill, Gray To Brown Rock Fragments, Mostly Sandy Shale Fragments, Friable.
(Hard Rock Layer)
Spt (Pushed 0.2', 50 Blow, Refusal, Rock Line)
Sandstone (Siltstone), Fine Grained, Gray W/a Few Sandy Shale Streaks Grading into Sandy Shale
Rec= 52%
Shale, Sandy, Gray
Shale, Dark Gray W/ Fine Grained Sandy Streaks
Sandstone (Siltstone), Fine Grained
Rec= 94%
Sandy Shale W/ Sandstone Streaks, Fine Grained
Sandstone (Siltstone), Fine Grained W/ Shale Streaks
Shale, Sandy W/ Fractured Layers and 0.2 Clay Layer
Sandstone (Siltstone), Fine Grained, Gray
Rec= 95%
(Weathered Clay Layer)
Shale, Sandy W/ Sandstone Streaks, Fine Grained

Figure A.2

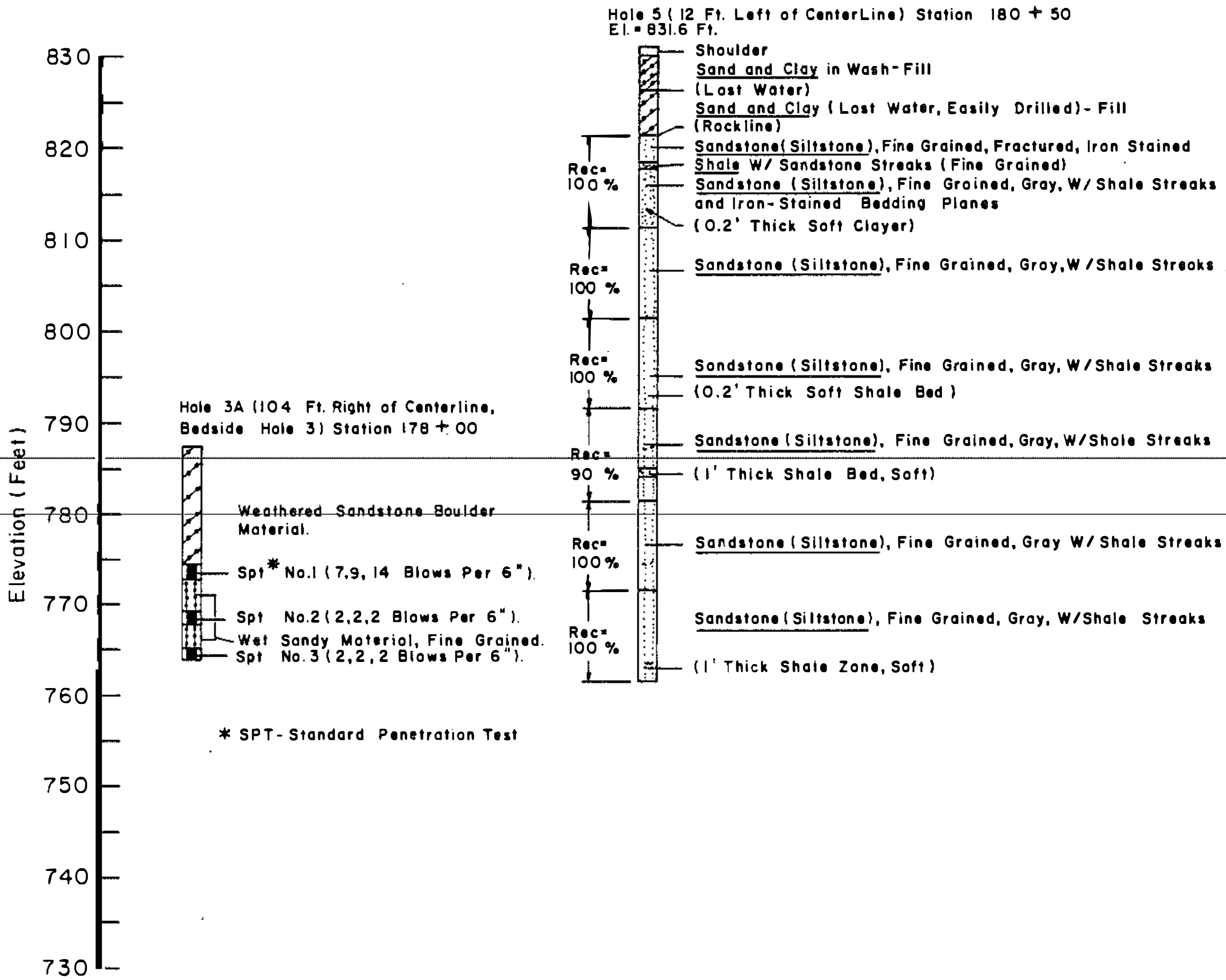
Hole 5 (12 Ft. Left of CenterLine) Station 180 + 50
El. = 831.6 Ft.
Shoulder
Sand and Clay in Wash-Fill
(Lost Water)
Sand and Clay (Lost Water, Easily Drilled) - Fill
(Rockline)
Sandstone (Siltstone), Fine Grained, Fractured, Iron Stained
Shale W/ Sandstone Streaks (Fine Grained)
Sandstone (Siltstone), Fine Grained, Gray, W/ Shale Streaks and Iron-Stained Bedding Planes
(0.2' Thick Soft Clayer)
Sandstone (Siltstone), Fine Grained, Gray, W /Shale Streaks
Sandstone (Siltstone), Fine Grained, Gray, W / Shale Streaks
(0.2' Thick Soft Shale Bed)
Sandstone (Siltstone), Fine Grained, Gray, W/Shale Streaks
(1' Thick Shale Bed, Soft)
Sandstone (Siltstone), Fine Grained, Gray W/ Shale Streaks
Sandstone (Siltstone), Fine Grained, Gray, W/Shale Streaks
(1' Thick Shale Zone, Soft)
Rec= 100 %
Rec= 100 %
Rec= 100 %
Rec= 90 %
Rec= 100 %
Rec= 100 %
Hole 3A (104 Ft. Right of Centerline, Bedside Hole 3) Station 178 + 00
Weathered Sandstone Boulder Material.
Spt* No.1 (7,9, 14 Blows Per 6").
Spt No.2 (2,2,2 Blows Per 6").
Wet Sandy Material, Fine Grained.
Spt No.3 (2,2,2 Blows Per 6").
* SPT - Standard Penetration Test
Elevation (Feet)
830
820
810
800
790
780
770
760
750
740
730

Figure A.3

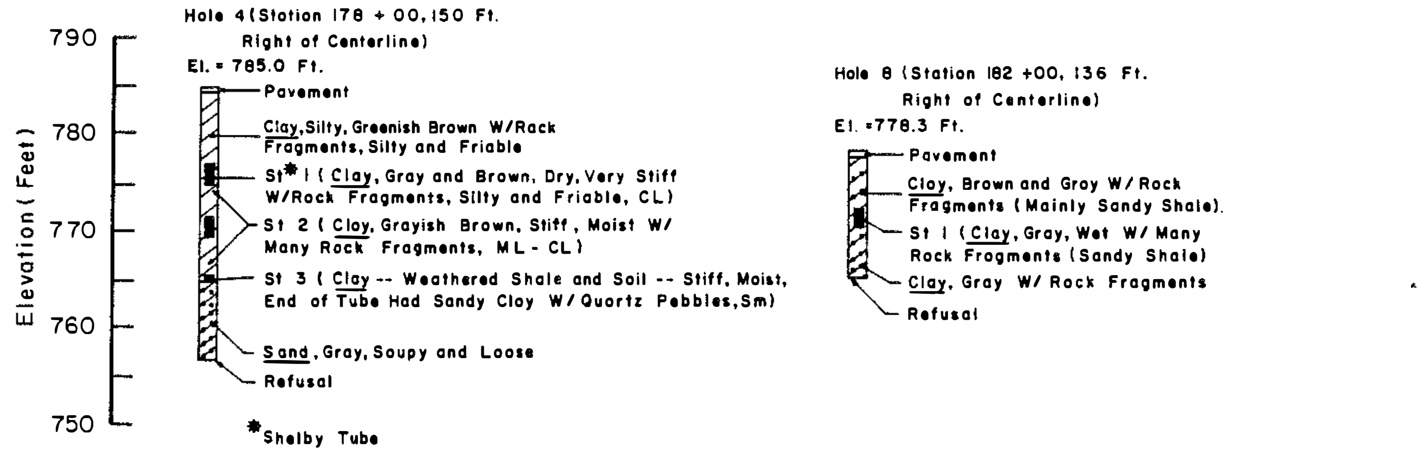

Figure A.4

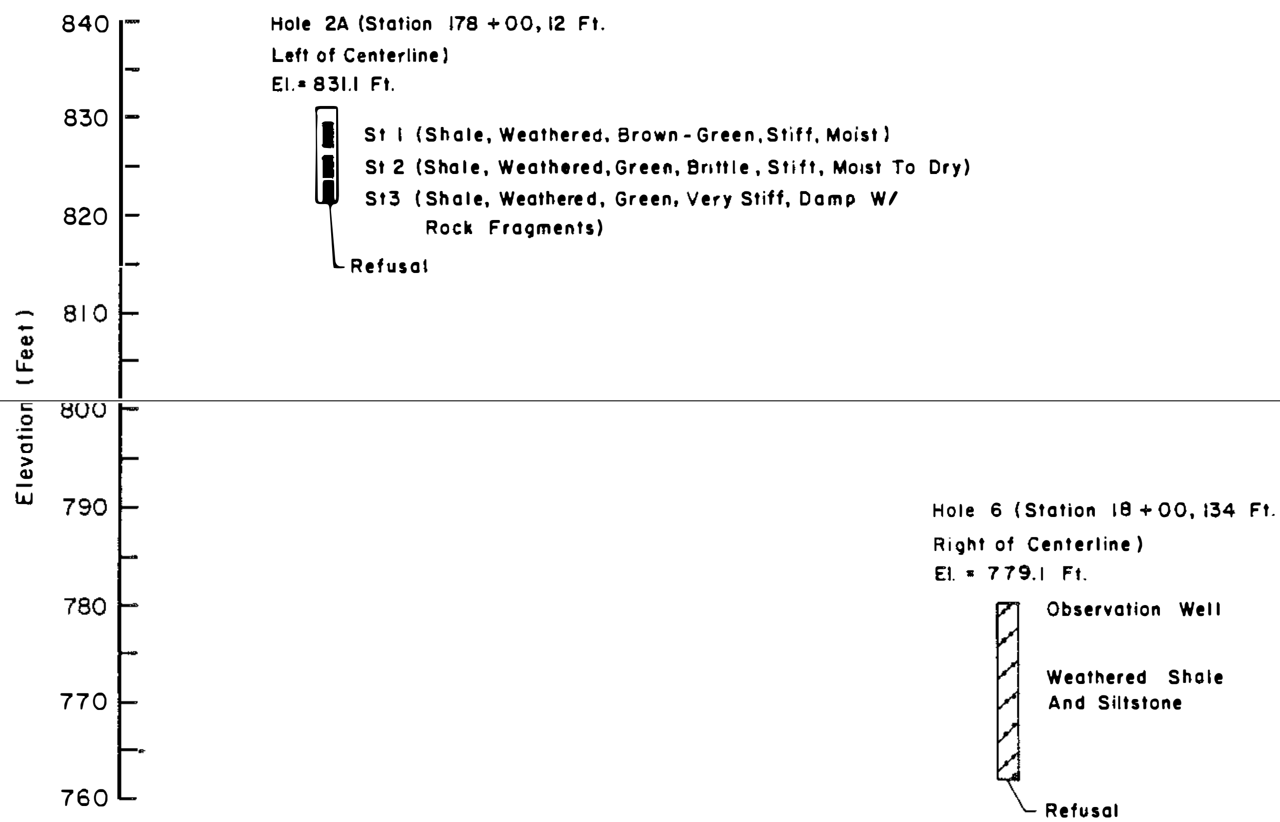

Figure A.5

APPENDIX B

GROUNDWATER OBSERVATIONS AS A FUNCTION OF TIME

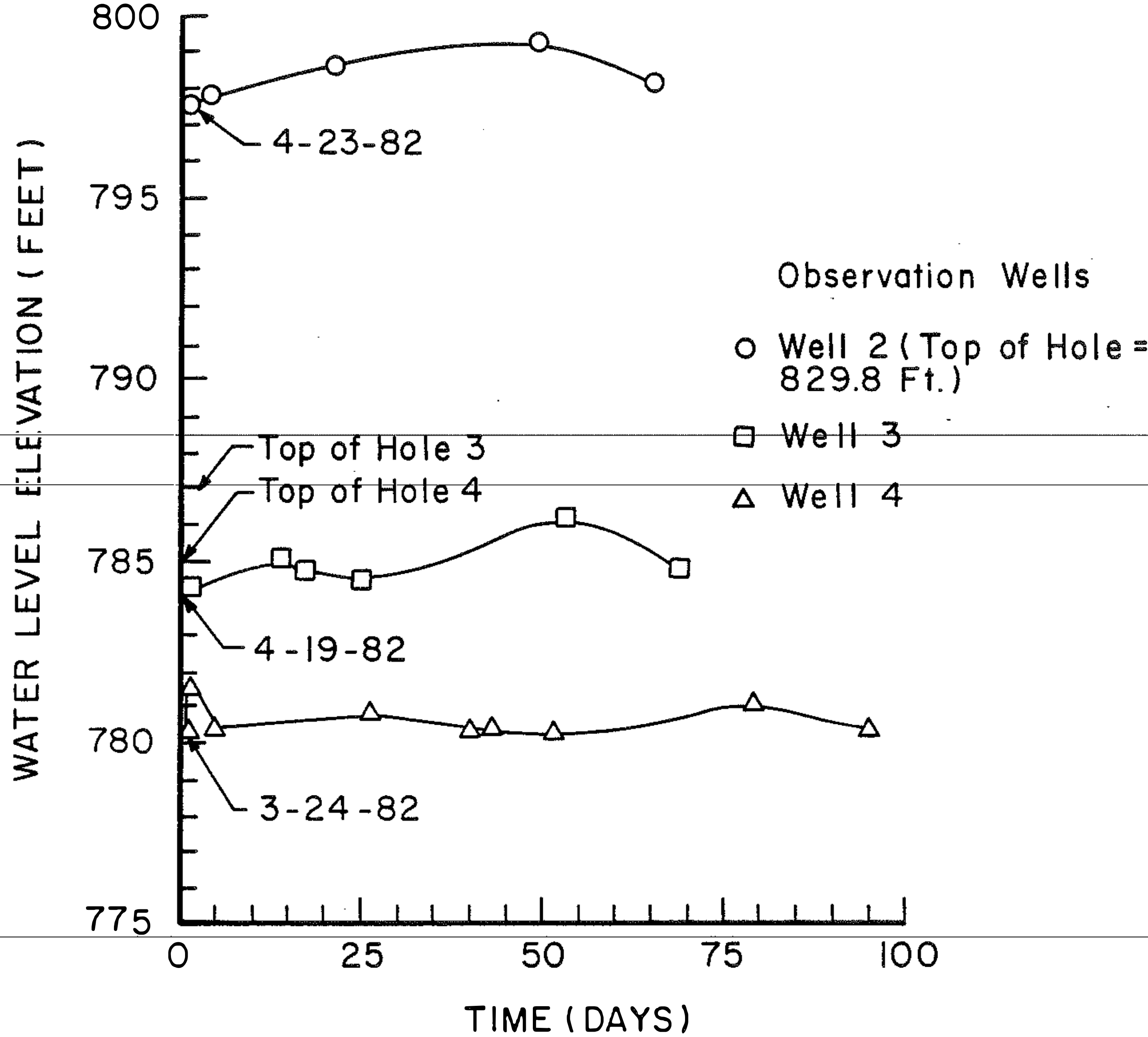
800
795
790
785
780
775
0
25
50
75
100
WATER LEVEL ELEVATION (FEET)
TIME (DAYS)
4-23-82
Top of Hole 3
Top of Hole 4
4-19-82
3-24-82
Observation Wells
Well 2 (Top of Hole = 829.8 Ft.)
Well 3
Well 4

Figure B.1

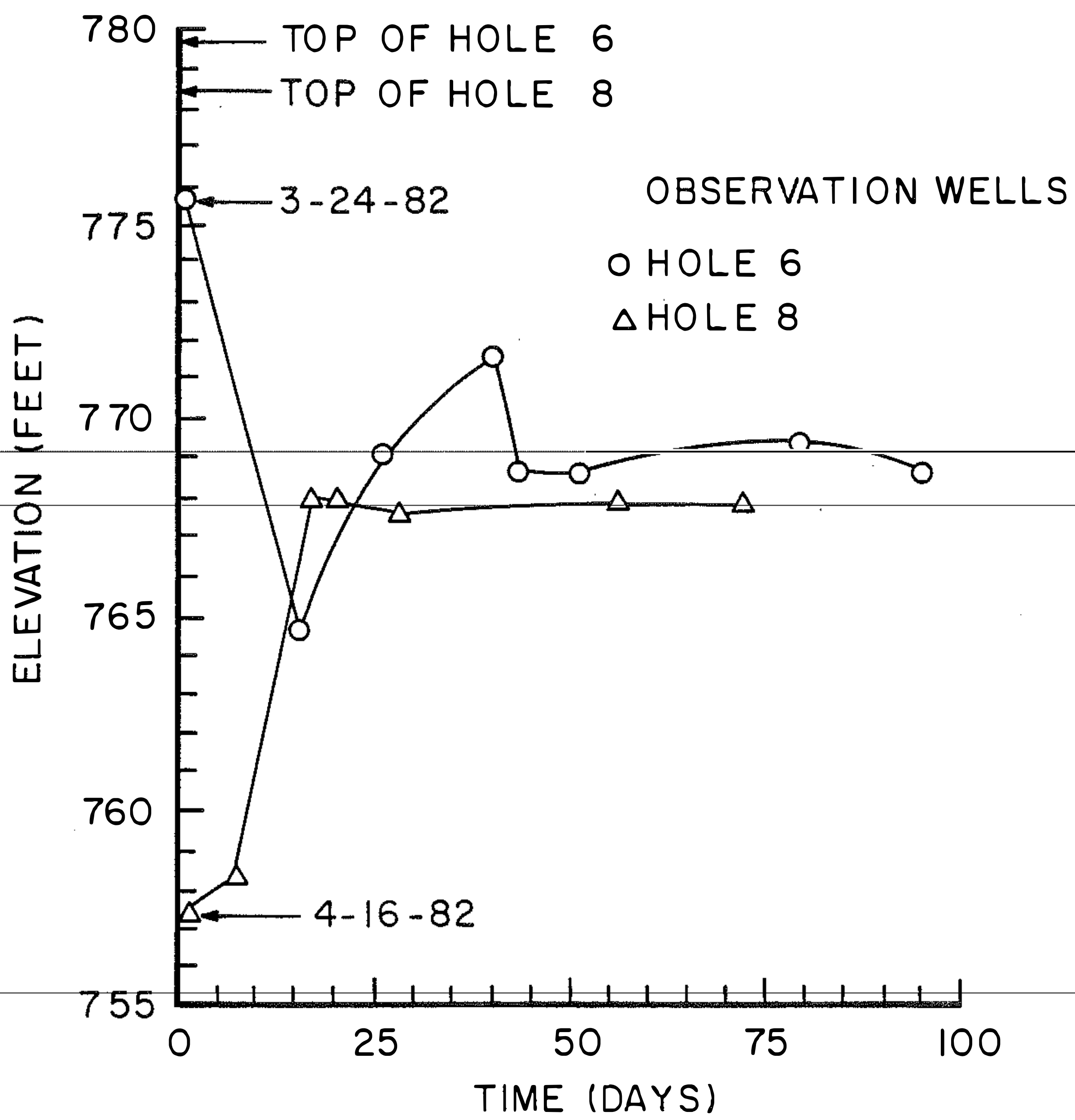
780
775
770
765
760
755
TOP OF HOLE 6
TOP OF HOLE 8
3-24-82
4-16-82
OBSERVATION WELLS
HOLE 6
HOLE 8
ELEVATION (FEET)
0
25
50
75
100
TIME (DAYS)

Figure B.2

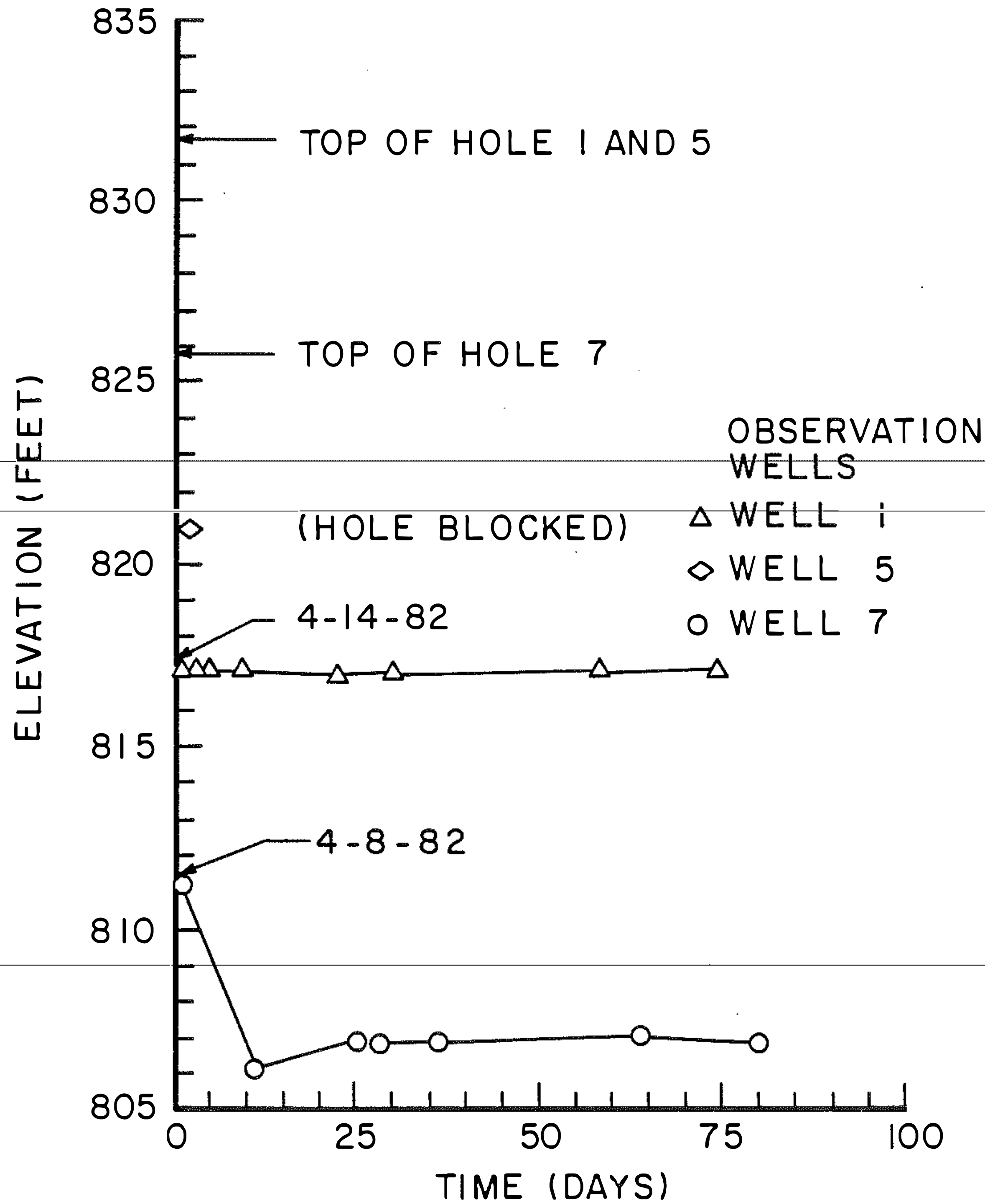
835
830
825
820
815
810
805
ELEVATION (FEET)
TOP OF HOLE 1 AND 5
TOP OF HOLE 7
(HOLE BLOCKED)
4-14-82
4-8-82
OBSERVATION WELLS
WELL 1
WELL 5
WELL 7
0
25
50
75
100
TIME (DAYS)

Figure B.3

APPENDIX C

ESTIMATION OF THE LOWERING OF THE SEEPAGE LINE IN THE UPSTREAM SHELL ZONE DURING RESERVOIR DRAWDOWN

ESITMATION OF THE LOWERING OF THE SEEPAGE LINE IN THE UPSTREAM SHELL ZONE DURING RESEVOIR DRAWDOWN

EQUATIONS (16):

$$X = (H - \Delta H_D) / 100 \text{ and}$$

$$P_D = k / n_e V$$

in which

X = dimensionless height ratio (that is, the ratio of height of the saturation line at face of core at end of drawdown expressed as a percentage of drawdown),
P_D = dimensionless parameter,
H_D = height of drawdown,
ΔH_D = change in height of saturation line at face of impervious core.
k = coefficient of permeability of the shell material,
n_e = $n(w - w)/100w$ = effective porosity; that is the ratio of void space drained to unit volume of soil where n is porosity, w_1 is saturated water content, and w_2 is water content after drainage, and
V = velocity of pool drawdown.

COMPUTATIONS:

Assume the pool elevation is to be lowered from normal pool (elevation 820.5 feet) to an elevation of 800 feet. The estimated (average) water content, w_2, of the soil after drainage is 11.5 percent w_1, is equal to 18 percent. The porosiity, n, is estimated from

$$n = 100\,e / (1 + e)$$

in which e = void ratio.

Values of void ratio were obtained from triaxial test specimens of the shell and

$$n = 100 \times 0.700 / 1.7 = 0.41$$

The effective porosity is

$$n = (41 / 100)(18 - 11.5) / 18 = 0.15.$$

The velocity of pool drawdown, V, is (assuming 60 days) for a drawdown period

$$V = (820.5 - 800.0)\text{ft} / 60 \text{ days} \times (24 \text{ hr/day}) (60 \text{ min/hr})$$

$$= 0.00024 \text{ ft/min}$$

and

$$P_D = 1.9 \times 10^{-8} \text{cm/sec} / 0.15 \times (0.00024 \text{ ft/min})$$

$$(\text{min}/60\text{sec}) (12 \text{ in./ft}) (2.54 \text{ cm/in.}) = 0.001.$$

From Chart III-4, Reference 16, and for a 2:1 slope,

$$X = 98\% = 100 (20.5 - \Delta H_D) / H_D$$

and

$$\Delta H_D = 0.4 \text{ ft.}$$

Repeating the above calculations for a drawdown time of 36,500 days, then

$$V = 20 \times 36{,}500 \times 24 \times 60 = 3.9 \times 10^{-7} \text{ft/min},$$

$$P_D = (1.9 \times 10^{-8} \text{cm/sec}) (60 \text{ sec/min}) (\text{ft}/12 \text{ in.})$$

$$(\text{in.}/2.54\text{cm}) / 0.15 \times 3.9 \times 10^{-6} \text{ft/min} = 0.63,$$

and

$$X = 78 = (20.5 - \Delta H_D) / 20.5$$

$$\Delta H_D = 4.1 \text{ ft.}$$