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REVIEW OF SOILS DESIGN, CONSTRUCTION AND PERFORMANCE OBSERVATIONS JOHN H. KERR PROJECT (BUGGS ISLAND AND ISLAND CREEK DAMS).

R. W. Cunny, et al

Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

August 1957

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REVIEW OF SOILS DESIGN, CONSTRUCTION AND PERFORMANCE OBSERVATIONS JOHN H. KERR PROJECT (BUGGS ISLAND AND ISLAND CREEK DAMS) VIRGINIA

TECHNICAL REPORT NO. 3-464

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U. S. Army Engineer Waterways Experiment Station

CORPS OF ENGINEERS

Vicksburg, Mississippi

ARMY-MRC VICKSBURG, MISS.

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PREFACE

This report is one of a number of reports on studies of the construction and behavior, from the standpoint of foundations and soil mechanics, of earth dams completed by the Corps of Engineers since 1950. These studies are conducted by the Waterways Experiment Station as part of the Civil Works Investigations program of the Office, Chief of Engineers, CWI Item 505, "Prototype Analyses (Soils)." The purpose of these studies is to compare the performance of the completed structure with design predictions, and from these comparisons gain information that will be valuable in the design and construction of future projects for the Corps of Engineers.

This report is a review of the design features of Buggs Island and Island Creek embankments for the John H. Kerr project, and includes certain observations made during and after construction relating to soil mechanics and foundation design. The dams were designed by and built under the supervision of the U. S. Army Engineer District, Norfolk, Va., and data on the design and construction and prototype observations were furnished by that district. No evaluation or analysis of data has been made by the Waterways Experiment Station. Design details not included may be found in: Definite Project Report on Buggs Island Reservoir, dated February 1956, Analysis of Design, Island Creek Dam, dated June 1950, Plans for Island Creek Dam, dated July 1950, and Plans for Buggs Island Dam, dated September 1950.

This report was prepared by Messrs. R. W. Cunny and A. G. Altschaeffl (formerly of Waterways Experiment Station) under the direction of Messrs. W. J. Turnbull, W. G. Shockley, and C. I. Mansur (formerly of Waterways Experiment Station), Soils Division, Waterways Experiment Station. The report was reviewed prior to publication by personnel of the Norfolk District and North Atlantic Division, and was reviewed and approved by the Office, Chief of Engineers.

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SUMMARY

The Project

The John H. Kerr project consists of a retaining structure, Buggs Island Dam, and a backwater protection structure, Island Creek Dam. Buggs Island Dam is a concrete gravity control structure 2800 ft long flanked by 19,700 ft of rolled-fill wing and saddle dikes having heights up to 45 ft and containing 1,200,000 cu yd of earth. The spillway has a capacity of 770,000 cfs. Island Creek Dam is a rolled-fill embankment 2100 ft long, 90 ft high, containing 566,000 cu yd of earth. Drainage from the protected area is pumped into the reservoir through a 12-ft-diam, reinforced concrete conduit.

Foundations

The rolled-fill portions of Buggs Island Dam are founded on residual soils grading from clay at the surface to sand and silt over fractured rock at relatively shallow depths. Design shear strengths were $c = 0.29$ ton per sq ft, $\phi = 30^{\circ}$ for the upper zone, and $c = 0.24$ ton per sq ft, $\phi = 32^{\circ}$ for the lower zone of foundation soils. Coefficients of permeability used in seepage analyses ranged from 35×10^{-4} cm per sec for the sand and silt zone to 350×10^{-4} cm per sec for the fractured rock. The upper clay zone was assumed to become remolded during construction and thus be equivalent to rolled fill having a $k = 0.0024 \times 10^{-4}$ cm per sec. Seepage is controlled by a deep downstream toe drain and an upstream impervious blanket.

Island Creek Dam is founded on residual soils underlain by weathered and sound rock. Design shear strengths for the foundation soils were $c = 0.05$ ton per sq ft, $\phi = 27^{\circ}$. Laboratory tests indicated that the coefficient of permeability for the foundation soil would be less than 1.00×10^{-4} cm per sec. Where foundation soils were less than 15 ft thick, a grout curtain was placed to prevent passage of excessive seepage through the fractured rock; otherwise, the overburden was considered capable of preventing significant seepage into the weathered rock.

Embankments

The dikes for Buggs Island Dam are unzoned embankments consisting of a relatively impervious mixture of sand, silt, and clay. Stability of the embankments was determined by means of the circular arc method. Minimum factors of safety of 1.01 and 1.74 were obtained for rapid drawdown and operating conditions, respectively. Approximately 93 per cent of the field control tests indicated that the moisture content of the compacted embankment was within the specified 2 per cent dry to 4 per cent wet of standard optimum water content. Field requirements of a minimum density of 95 per cent of standard density were generally met with the specified 6 passes of a 500-psi tamping roller. Consolidated-drained direct shear tests on 9 record samples indicated average values of c = 0.29 ton per sq ft, $\phi = 28^{\circ}$, as compared with design values of c = 0.10 ton per sq ft, $\phi = 27^{\circ}$. The upstream slope is protected by 2 ft of dumped riprap over a 1-ft crushed stone blanket; the downstream slope is planted in grass. Seepage is collected by a downstream toe drain located generally in areas where the flood control pool contacts the dikes. Observations of one drain section (2500 ft) indicate a flow of 30 gpm with the dike under a head of 15 to 20 ft; this head is 20 to 25 ft less than the maximum head expected against the dike section. All other toe drains have been dry.

The design for the Island Creek Dam embankment provided for a central impervious core flanked by relatively pervious shells. The circular arc method indicated minimum factors of safety of 1.45 and 1.35 for the construction and operating conditions, respectively. Stability for the rapid drawdown condition was not analyzed. The core was constructed of relatively impervious materials found in the borrow areas. Results of a limited number of permeability tests on record samples of shell and core materials indicate that the permeabilities in the two sections are not significantly different. At least 95 per cent of the field density tests indicated satisfactory densities of the embankment, but only 76 per cent indicated satisfactory moisture control. Consolidated-drained direct shear tests on 5 record samples indicated average values of c = 0.31 ton per sq ft, $\phi = 28^{\circ}$, compared with design values of c = 0.05 ton per sq ft, $\phi = 27^{\circ}$. The lower landside slope is protected by 20 in. of crushed stone; the upper 57 ft of slope was mulched and seeded. The reservoir slope is protected by 18 in. of dumped riprap on a 6-in. crushed stone filter from the top of the dam to 3 ft below the minimum power pool. No provisions other than the impervious core were made for seepage control.

Spillway and Outlet Works

Spillways and outlet work structures are founded on sound rock. No engineering measurement devices were provided for these structures.

REVIEW OF SOILS DESIGN, CONSTRUCTION, AND PERFORMANCE OBSERVATIONS
JOHN H. KERR PROJECT
(BUGGS ISLAND AND ISLAND CREEK DAMS) VIRGINIA

PART I: GENERAL INFORMATION

1. The John H. Kerr project, a multipurpose development within the general plan for the Roanoke River Basin, is located in Mecklenburg County, approximately 20 miles downstream from Clarksville, Virginia. Location of the project is shown on the vicinity map in fig. 1. The project consists of Buggs Island Dam, the major retaining structure, located on the Roanoke River, and Island Creek Dam about 18 miles upstream on Island Creek, a tributary of the Roanoke River. Island Creek Dam provides backwater protection for tungsten mining operations in the area upstream.

Buggs Island Dam

2. Buggs Island Dam consists of a 2800-ft-long central concrete gravity section containing the spillway, flanked on each side by a series of earth wing and saddle dikes totaling 19,700 ft in length. The maximum height of the dikes is 45 ft with crown at el 332.* The crown widths of the dikes are 40 ft for those sections topped by a roadway and 15 ft for the other sections. The dikes generally have side slopes of 1 on 2.5 and 1 on 2, except for the smaller dikes which have slopes of 1 on 1.5. Upstream slopes of the dikes are protected by 24 in. of dumped riprap on a 12-in. crushed stone blanket. Downstream slopes are planted in grass. Earth placed in the dikes totaled 1,200,000 cu yd. A plan of the dikes is shown in fig. 1.

3. The spillway is 1150 ft long and has a gate-controlled overflow section of the ogee type equipped with 22 individually operated tainter gates. The elevation of the spillway crest is 288 ft, and its design capacity is 770,000 cfs at maximum surcharge water-surface elevation.

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* All elevations are in feet above mean sea level.

Ten sluices, controlled by slide gates, provide control of the reservoir pool when the stage is below spillway crest and furnish flow to the powerhouse. The power installation has a generating capacity of 206,000 kw. Pertinent data concerning the dam and reservoir are listed in table 1.

4. Construction of the rolled fill was begun on 16 January 1951. The left wing dike was completed 12 July 1951, the right wing dikes were completed 1 April 1952, and the earth construction including saddle dikes was accepted on 1 July 1952. Fig. 2. is an aerial photograph of the concrete gravity section and wing dikes.

Fig. 2. Aerial view of Buggs Island Dam with the concrete gravity section in the center flanked by earth wing dikes

Island Creek Dam

5. Island Creek Dam is a 2100-ft-long, rolled fill section, 90 ft high with crown at el 332. The crown is 32 ft wide and supports a roadway. The embankment has side slopes of 1 on 2 at the top, 1 on 2.5 near mid-height, and 1 on 3 for the lower portions. The lower upstream or landside (Island Creek) slopes are protected by 20 in. of crushed stone; the upper 57 ft is mulched and seeded. The slope on the reservoir or downstream side is protected by 18 in. of dumped riprap on a 6-in.-thick

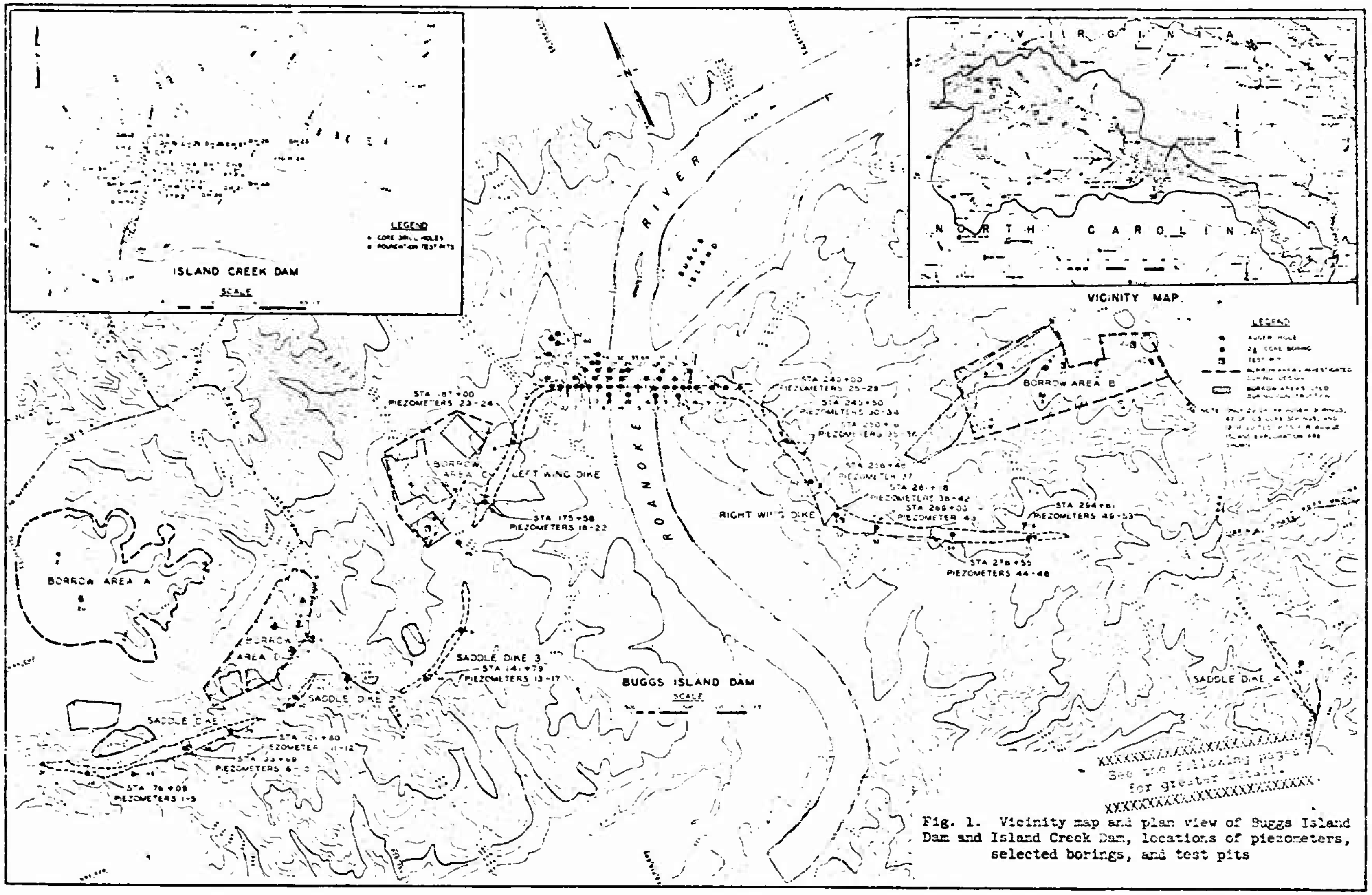

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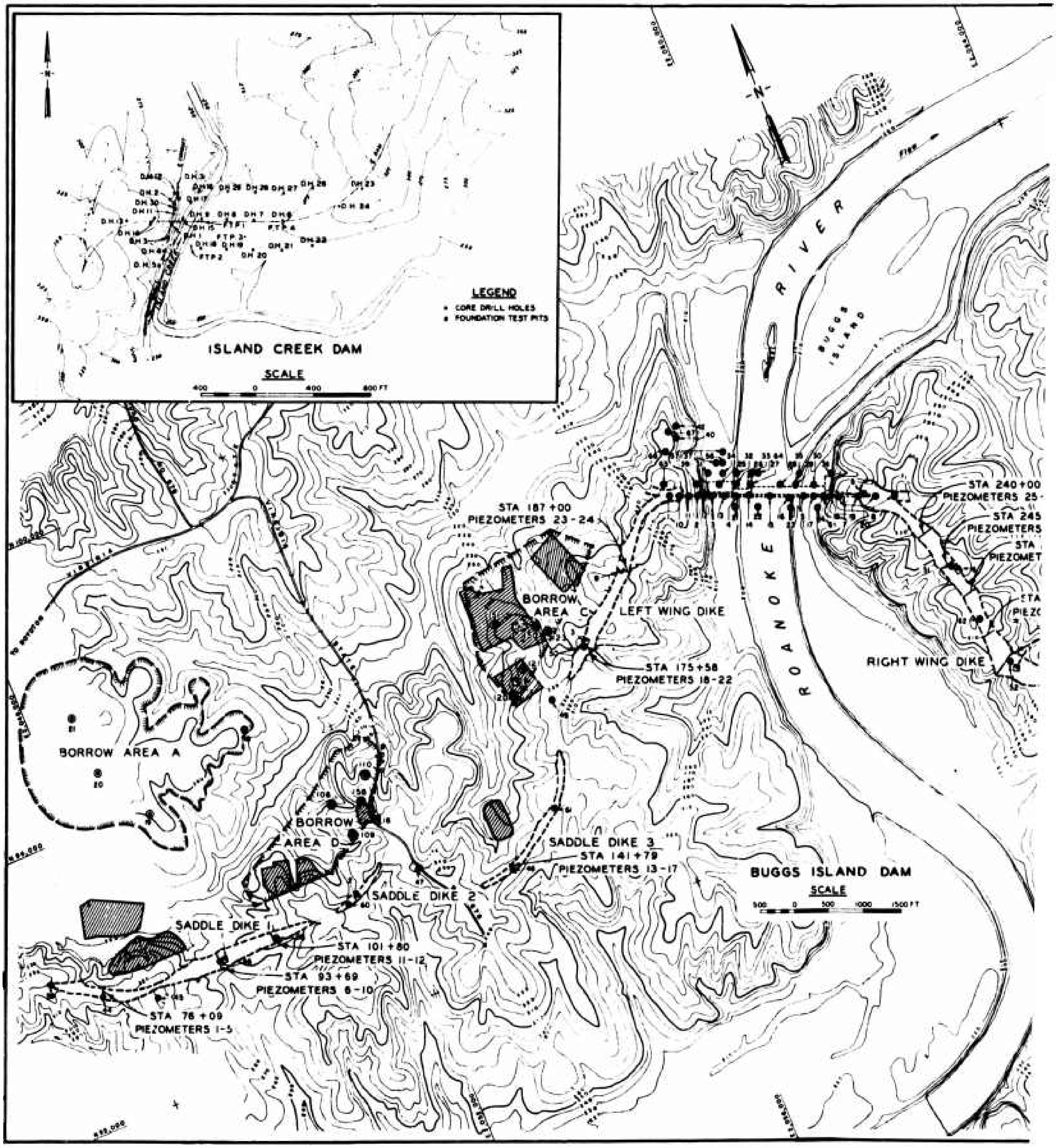
Fig. 1. Vicinity map and plan view of Buggs Island Dam and Island Creek Dam, locations of piezometers, selected borings, and test pits



ISLAND CREEK DAM
LEGEND
CORE DRILL HOLES
FOUNDATION TEST PITS
SCALE
400 0 400 800 FT
ROANOKE RIVER
BUGGS ISLAND
STA 240+00
STA 187+00
PIEZOMETERS 23-24
BORROW AREA C
LEFT WING DIKE
RIGHT WING DIKE
STA 175+58
PIEZOMETERS 18-22
BORROW AREA A
BORROW AREA D
SADDLE DIKE 3
STA 141+79
PIEZOMETERS 13-17
BUGGS ISLAND DAM
SCALE
500 0 500 1000 1500 FT
SADDLE DIKE 2
SADDLE DIKE 1
STA 101+80
PIEZOMETERS 11-12
STA 93+69
PIEZOMETERS 6-10
STA 76+09
PIEZOMETERS 1-5

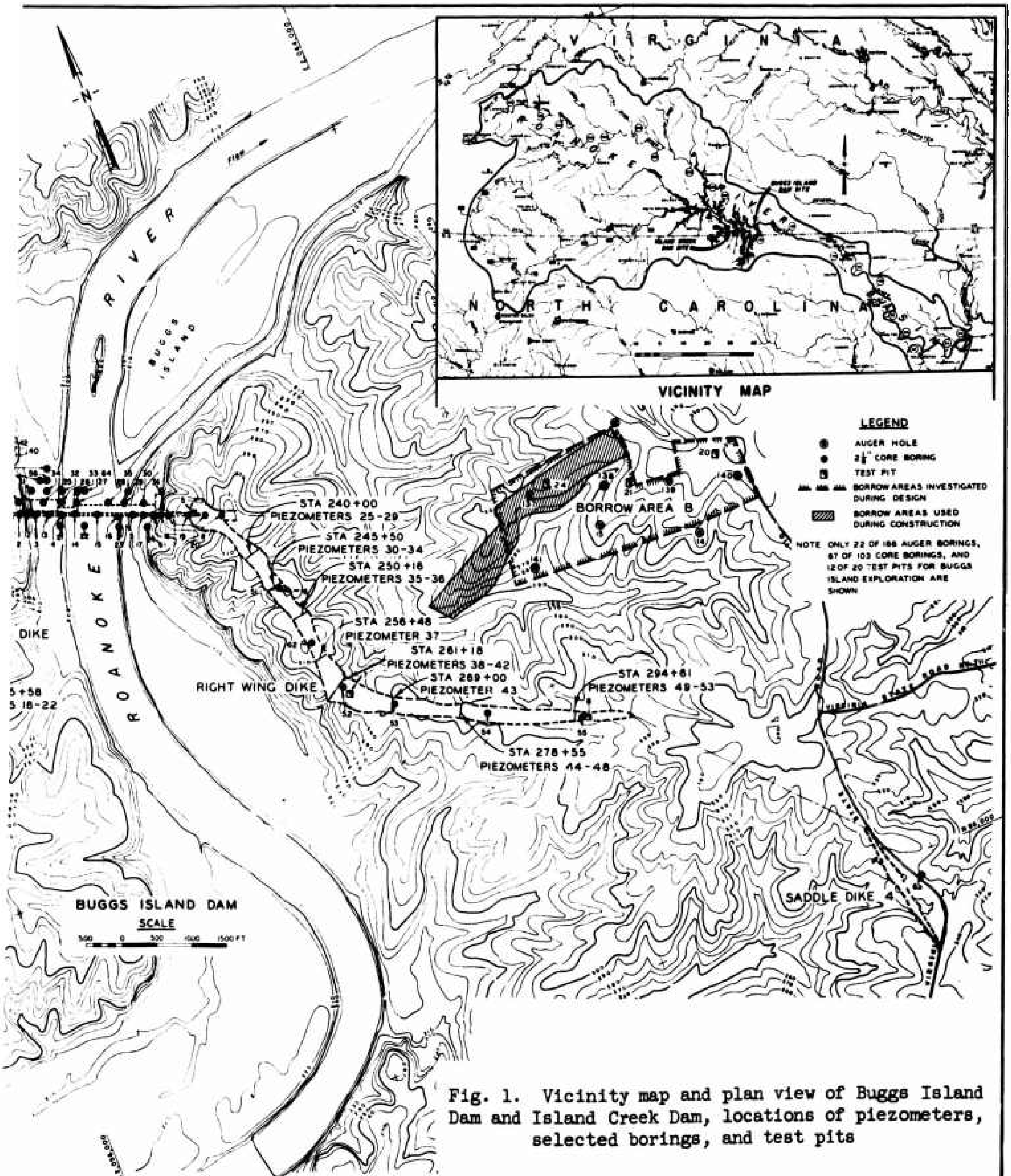


Fig. 1. Vicinity map and plan view of Buggs Island Dam and Island Creek Dam, locations of piezometers, selected borings, and test pits

stone blanket between el 265 and top of dam.

6. Water from Island Creek is pumped into the reservoir through a reinforced concrete conduit 12 ft in diameter and 353 ft long located in the west end of the embankment. Hydraulically operated gates in the powerhouse, coupled to the pumps, control discharge and prevent back flow of water from the reservoir. An emergency gate on the reservoir side of the conduit is used during repair work and is operated by a mobile crane. The maximum differential head anticipated is 45 ft. Pertinent features of the dam are given in table 1.

7. Construction was begun in September 1950 and was completed in September 1951. Fig. 3 shows the partially completed embankment.

Fig. 3. Island Creek Dam from the downstream (reservoir) side. The 48-in. pipe, shown in the upper center, was used to discharge water through the cofferdam and conduit after closure of the dam during final stages of construction

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Table 1

Dam and Reservoir Data

Drainage area	7,800 sq mi
Design flood:	
Maximum inflow	320,000 cfs
Maximum controlled outflow	60,000 cfs
Maximum surcharge pool el	325 ft
Maximum flood control pool el	320 ft
Maximum power pool el	300 ft
Minimum power pool el	268 ft
Buggs Island Dam:	
Top of dam, el	332 ft
Length	
Concrete gravity section	1,800 ft
Earth wing and saddle dikes	19,700 ft
Maximum height	
Concrete gravity section	144 ft
Earth wing and saddle dikes	45 ft
Volume	
Concrete gravity section	700,000 cu yd
Earth wing and saddle dikes	1,200,000 cu yd
Spillway	
Length	1,150 ft
Crest el	288 ft
Crest gate el	320 ft
Crest gate dimensions	42 ft x 32 ft
Sluices	
Number	10
Dimensions	5 ft 8 in. x 10 ft
Island Creek Dam:	
Top of dam, el	332 ft
Length	2,100 ft
Maximum height	90 ft
Volume earth embankment	566,000 cu yd
Conduit	
Diameter	12 ft
Inlet el	259.5 ft

A diversion channel east of the creek channel carried water until the spring of 1951 when the concrete conduit was completed. A 48-in.-diam steel pipe was used to pass water into the conduit and through the embankment when the diversion channel was closed. Embankment fill was placed the full length of the dam in the spring of 1951. A total of 566,000 cu yd of compacted fill was placed. The riprap was placed under a separate contract. Heavy rains during the summer of 1951 eroded a portion of the protective stone layer and necessitated repeated repairs. Approximately 37,500 sq yd of riprap and 12,500 sq yd of stone blanket were placed.

PART II: BUGGS ISLAND DAM

Design

Foundation

8. Geology. The John H. Kerr reservoir is within the Piedmont physiographic area consisting mainly of igneous and metamorphic rock underlying a residual mantle of moderately thick to thick overburden. Rounded hills and moderately wide valleys characterize the terrain; schists appear to form resistant ridges. Rock strata dip steeply westward and strike slightly east of north. Long ridges of residual soil and decomposed rock extend southeastward and westward from the dam site. Wing and saddle dikes are founded on these ridges.

9. Field explorations. The foundation exploration included 103 core borings, 78 auger borings, and 20 test pits. Locations of selected preliminary borings are shown in fig. 1. The foundation soils grade from clay at the surface, to sand and silt at depths greater than 5 ft, to weathered and sound rock at depths in the range of 30 ft. Generalized profiles along the dikes are shown in figs. 4, 5, 6, and 7; locations of all borings along the line of the profiles are shown in these figures.

10. A special investigation was initiated to determine the in-place permeability of the foundation soils by a "falling-head" test in a cased auger hole. However, the method was not successful with the residual soils present at the site. Ground-water observations revealed that the seasonal water-table fluctuations were minor. Even though the water table was within 5 ft of the surface in many instances, no construction difficulties were anticipated. Piezometers were installed at 14 locations shown in fig. 1 to measure the hydrostatic head in the foundation and to check the efficacy of the drainage system.

11. Laboratory tests. The laboratory testing program for foundation soils included classification, shear strength, permeability, and consolidation tests. The results of the tests for which individual test data are available are presented in table 2. The gradations of the

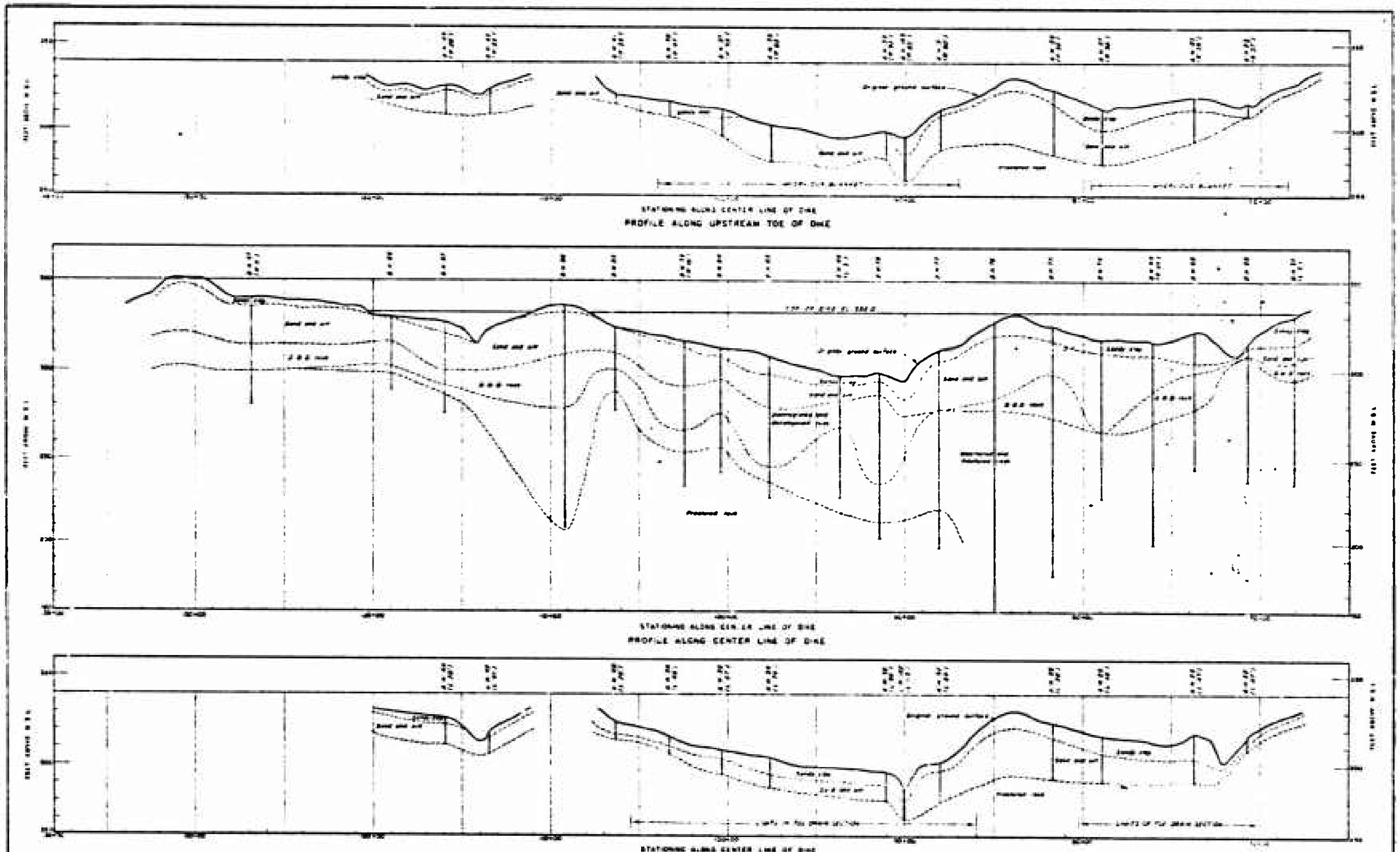

Fig. 4. Generalized soil profiles, Buggs Island Dam, sta 65-00-135+00

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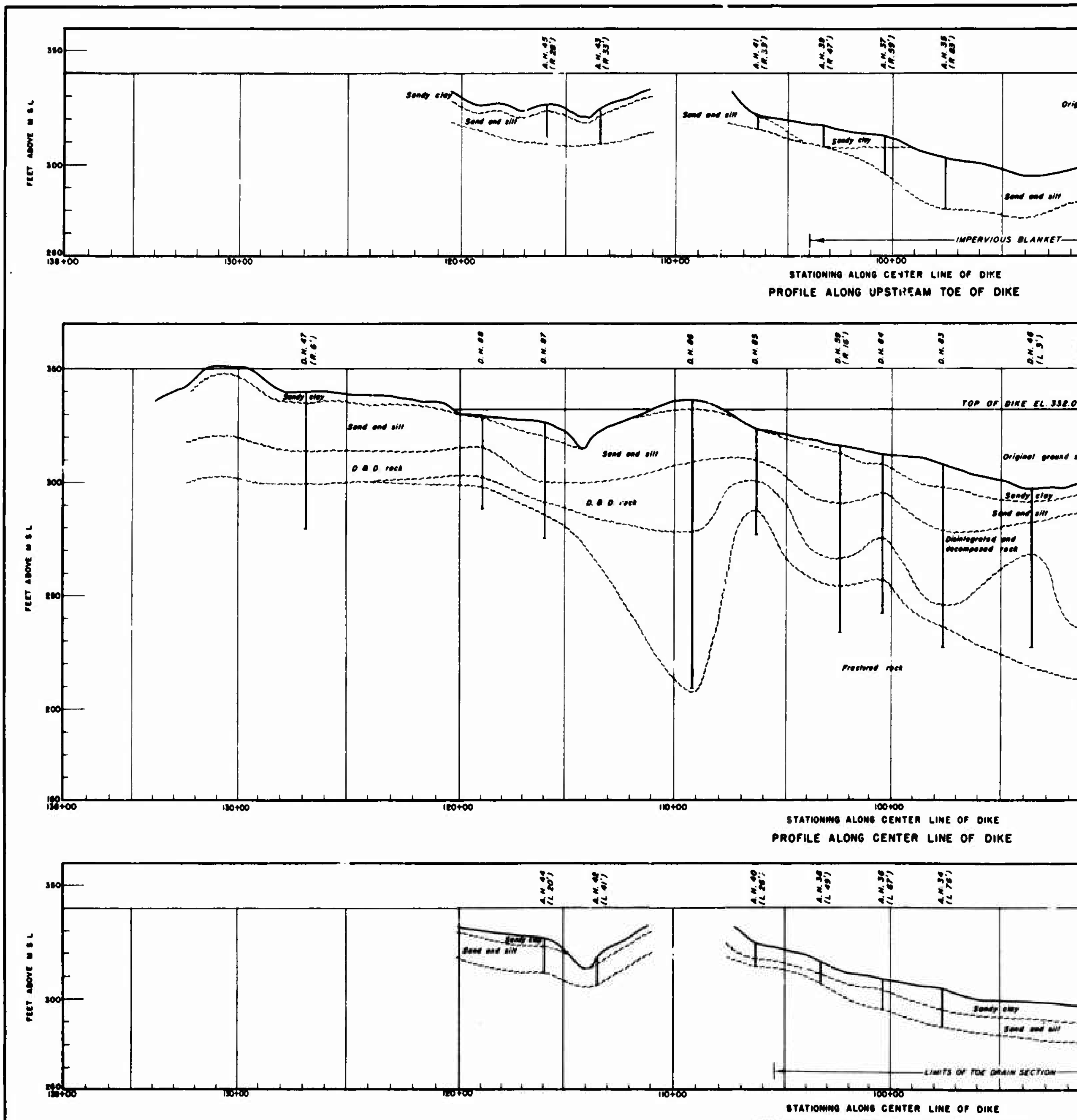

NOTE LOCATIONS OF BORINGS ARE REFERENCED TO CENTER LINE OF DIKE
SELECTED BORINGS ARE SHOWN IN PLAN ON FIG. 1

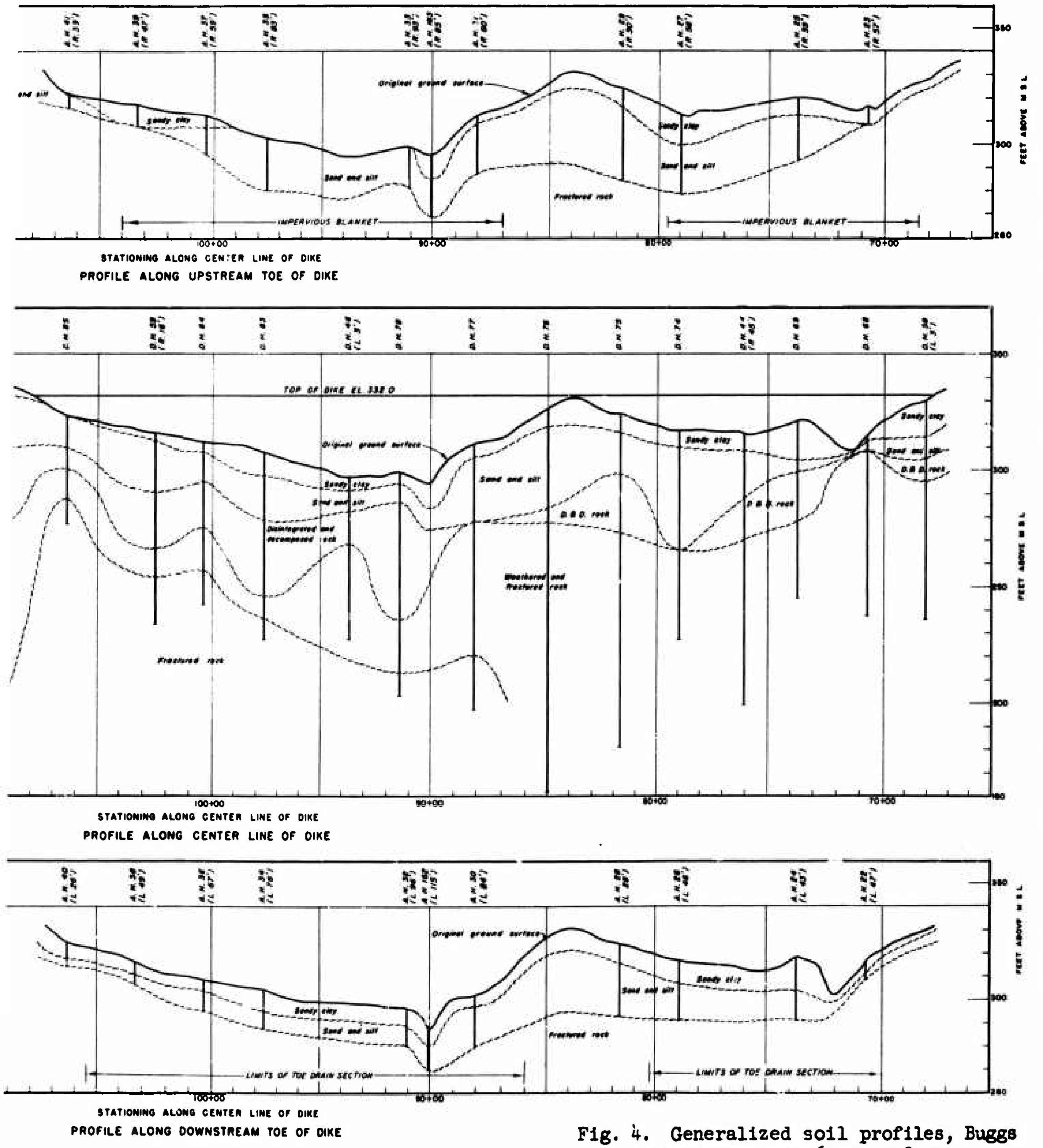

Fig. 4. Generalized soil profiles, Buggs Island Dam, sta 65+00-138+00

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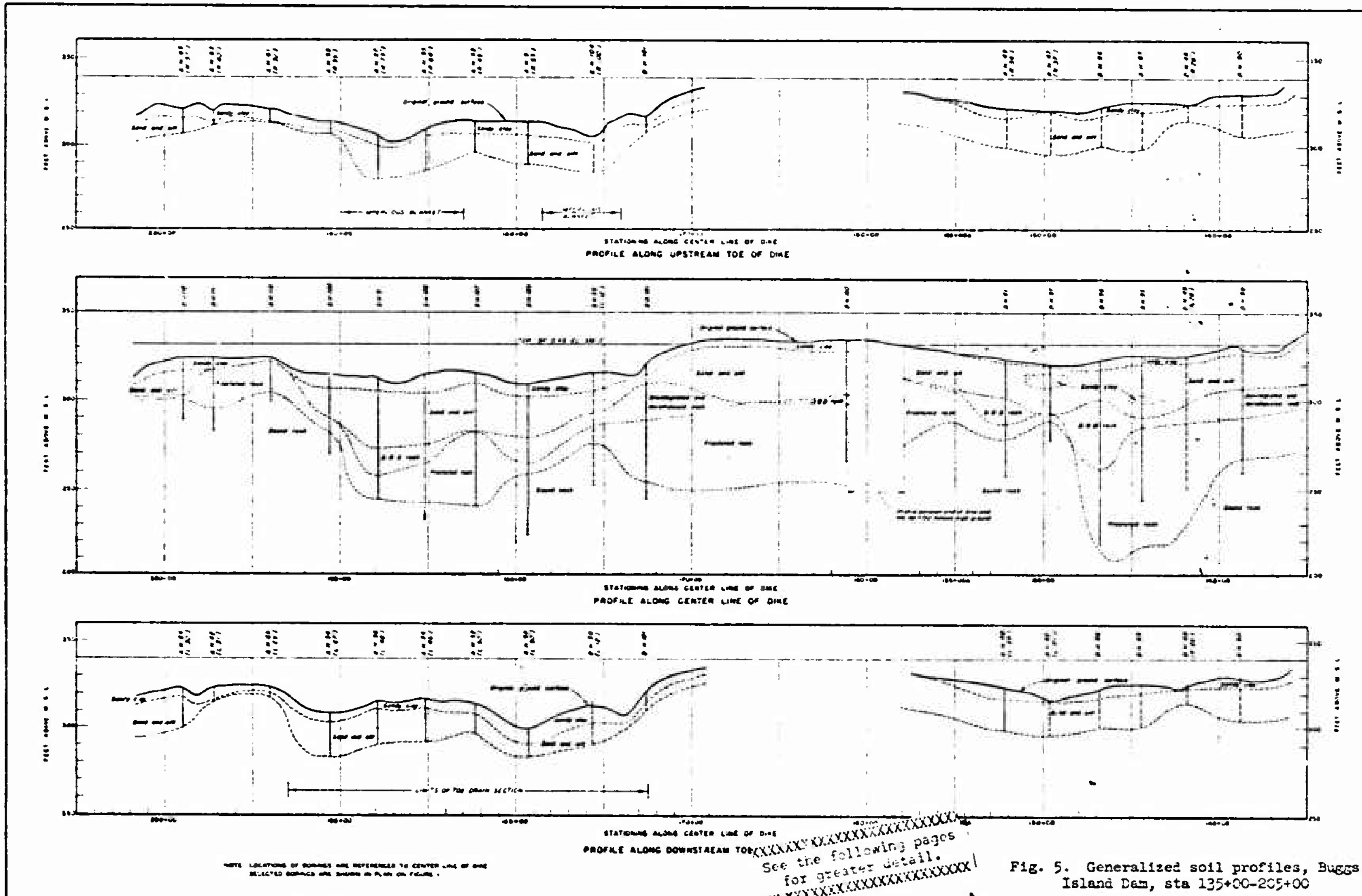

Fig. 5. Generalized soil profiles, Buggs Island Dam, sta 135+00-205+00

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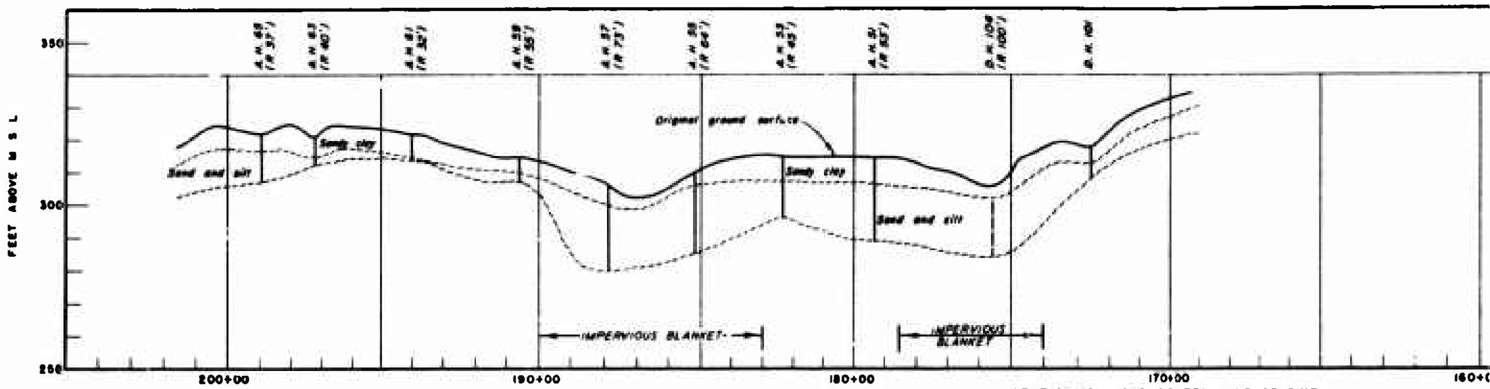
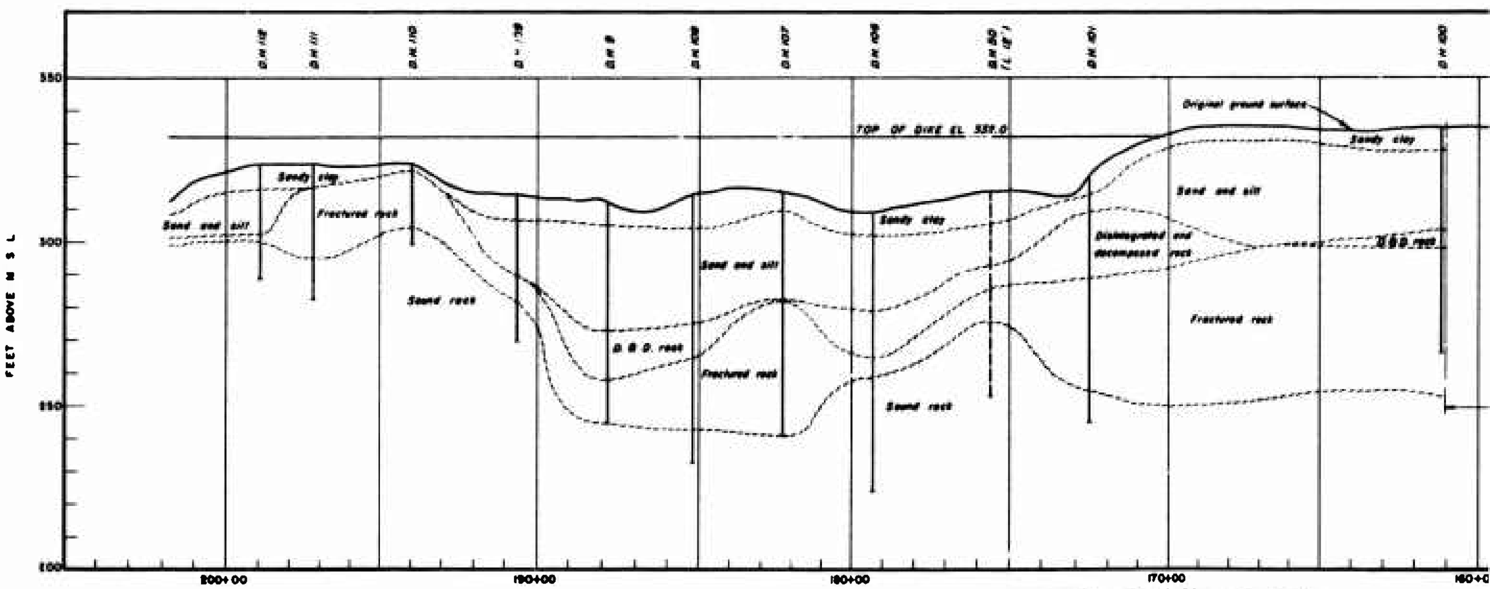
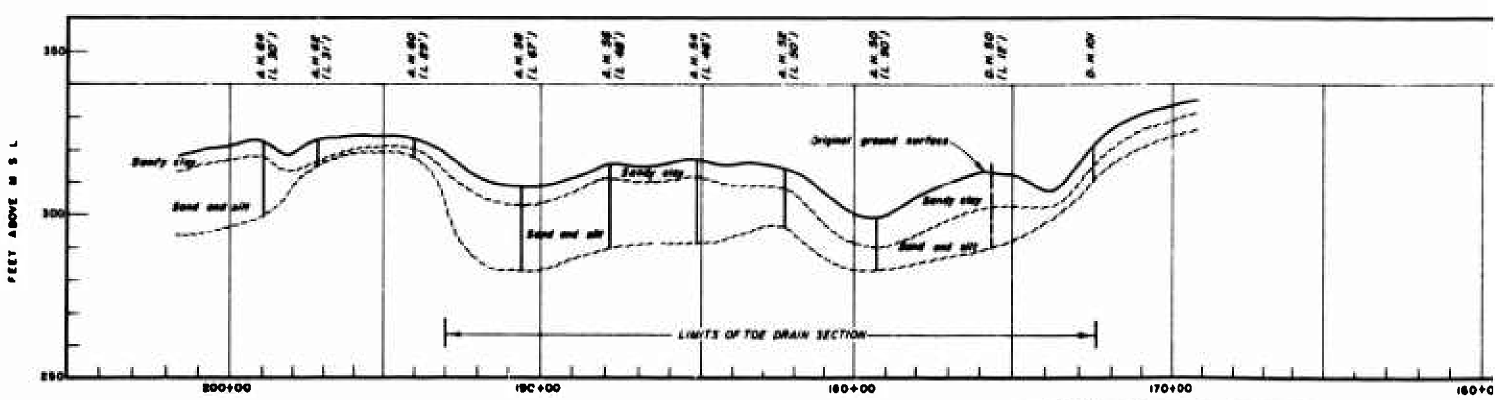


PROFILE ALONG UPSTREAM TOE OF DIKE



PROFILE ALONG CENTER LINE OF DIKE



PROFILE ALONG DOWNSTREAM TOE OF DIKE

NOTE: LOCATIONS OF BORINGS ARE REFERENCED TO CENTER LINE OF DIKE
SELECTED BORINGS ARE SHOWN IN PLAN ON FIGURE 1

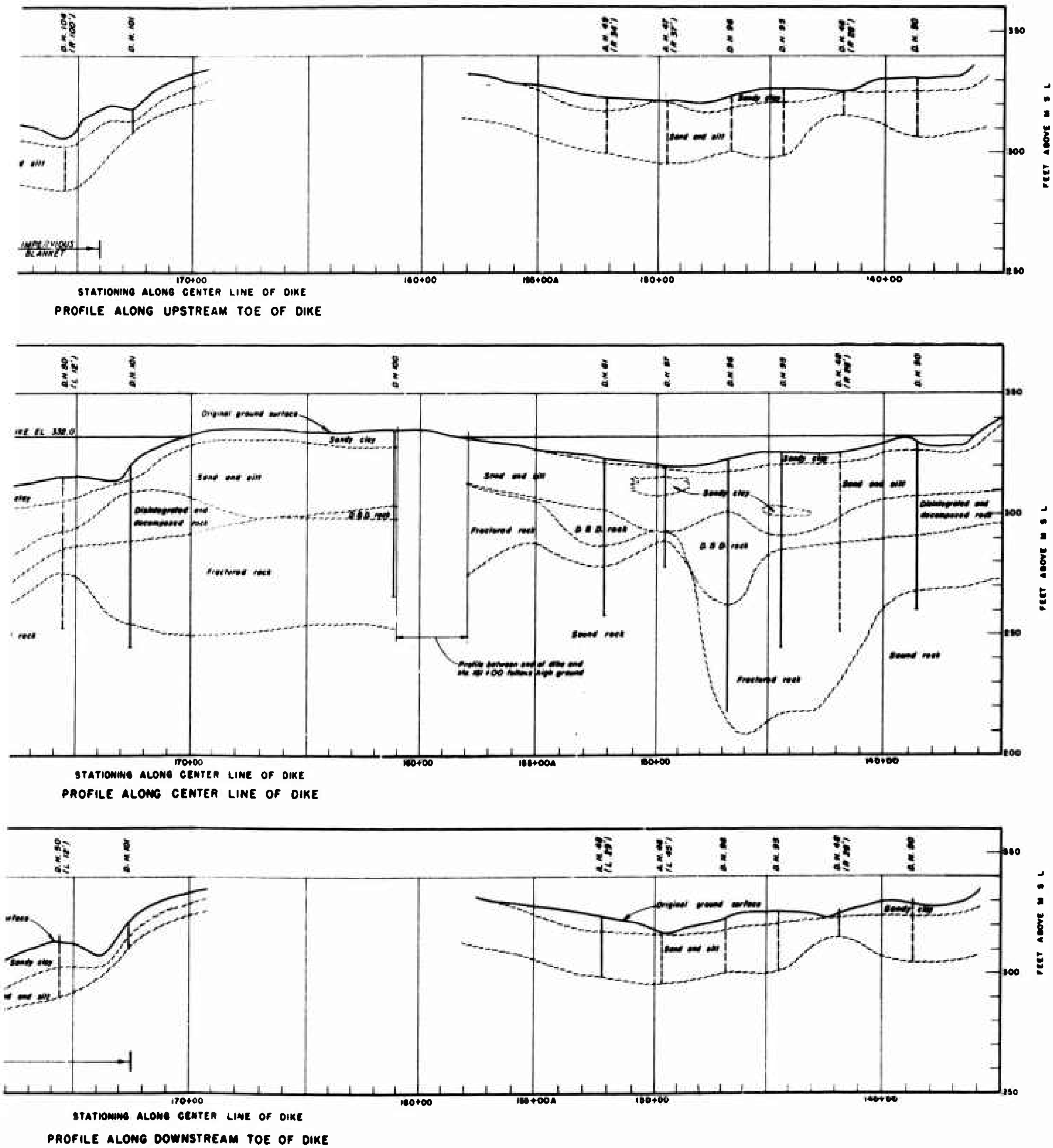


Fig. 5. Generalized soil profiles, Buggs Island Dam, sta 135+00-205+00

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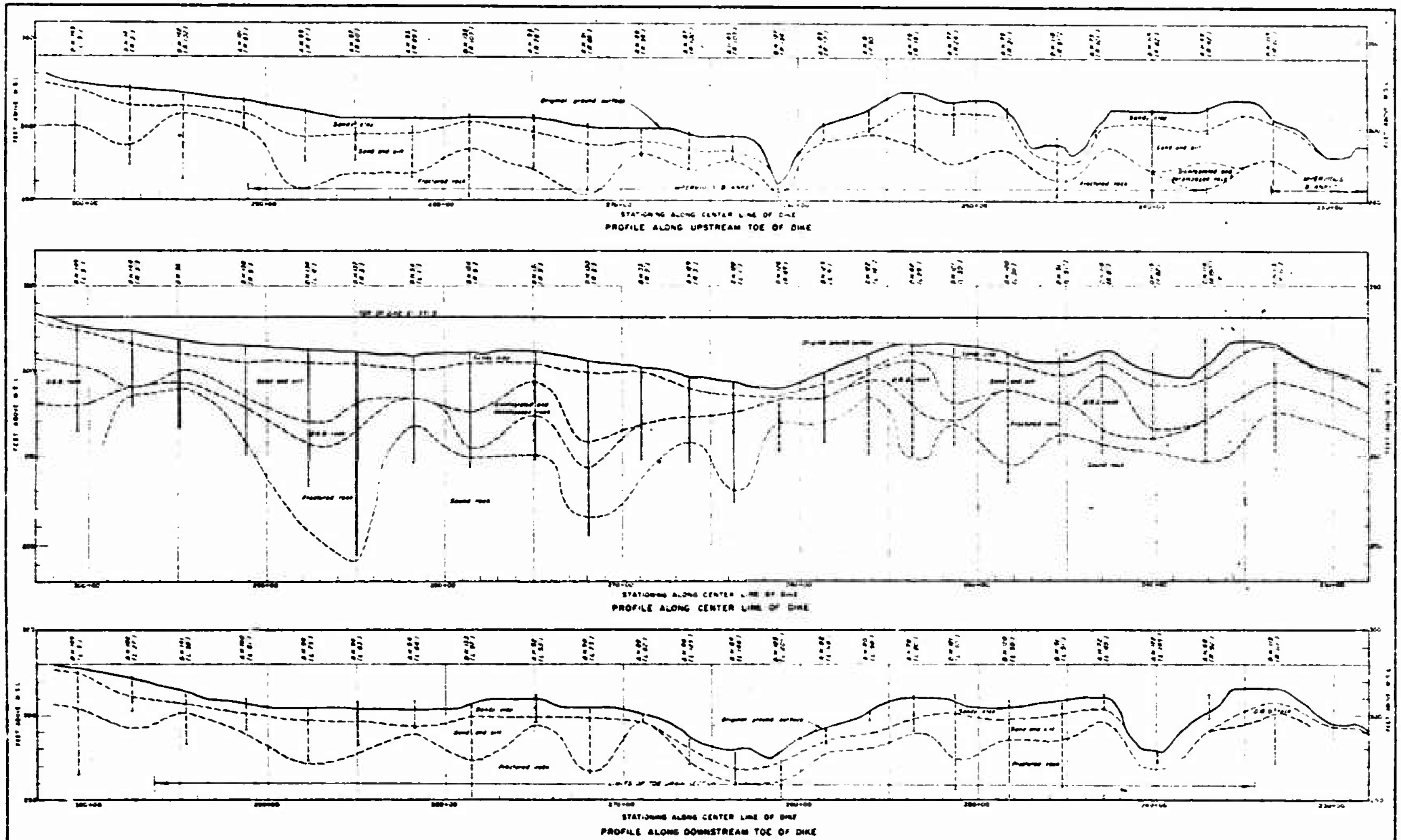

Fig. 6. Generalized soil profiles, Buggs Island Dam, sta 228+00-303+00

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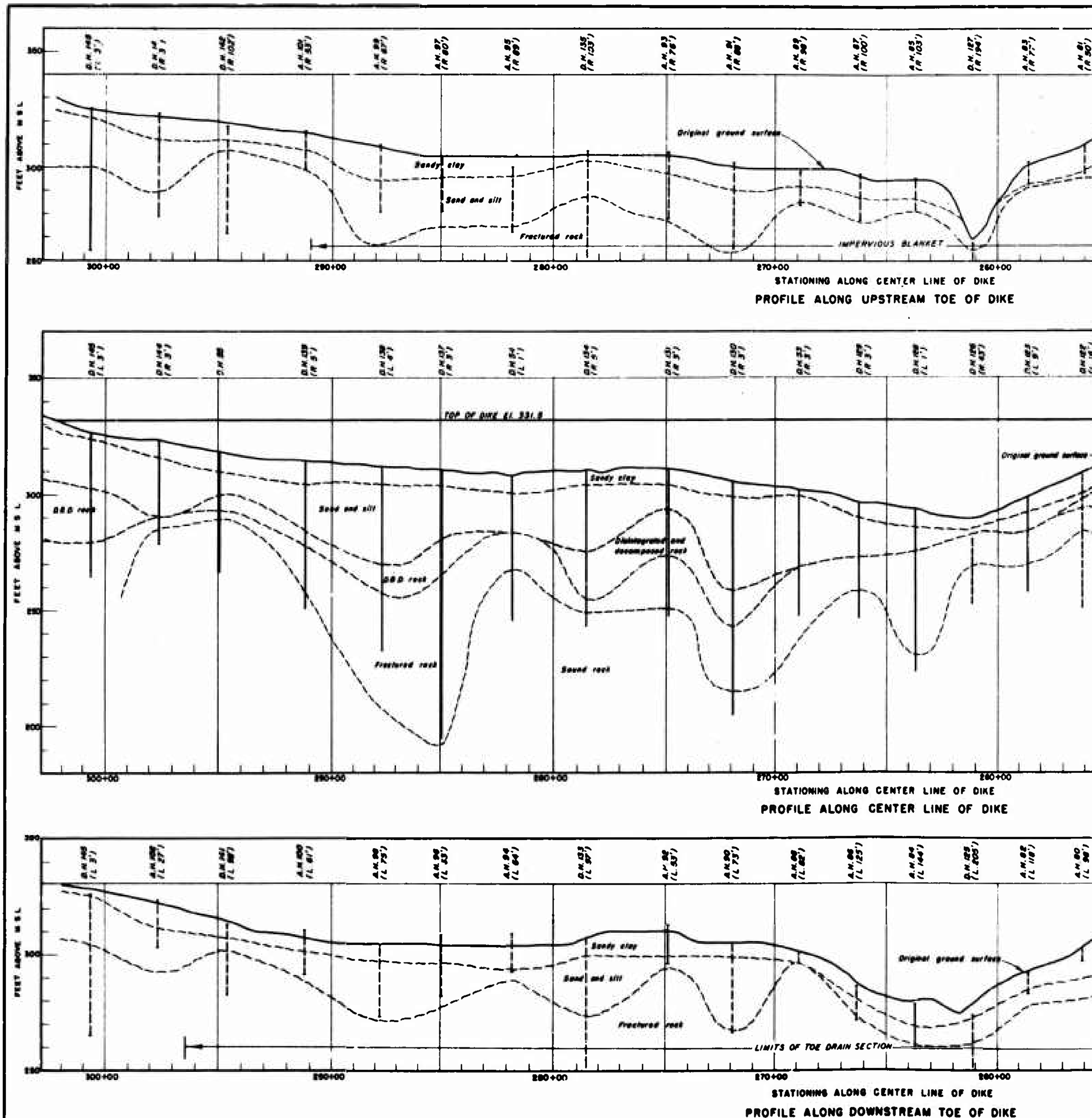

NOTE LOCATIONS OF BORINGS ARE REFERENCED TO CENTER LINE OF DIKE.
SELECTED BORINGS ARE SHOWN IN PLAN ON FIGURE 1.

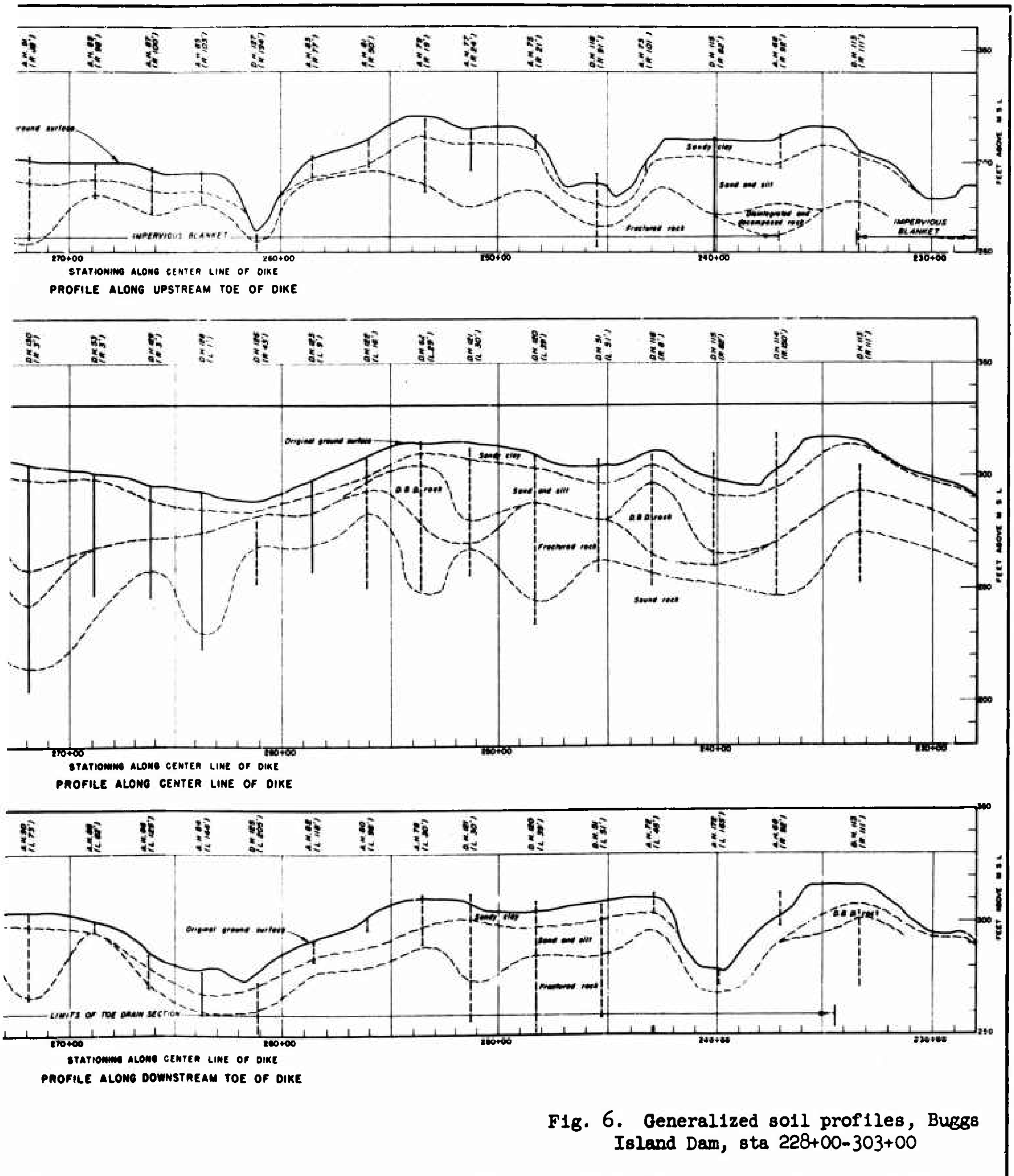

Fig. 6. Generalized soil profiles, Buggs Island Dam, sta 228+00-303+00

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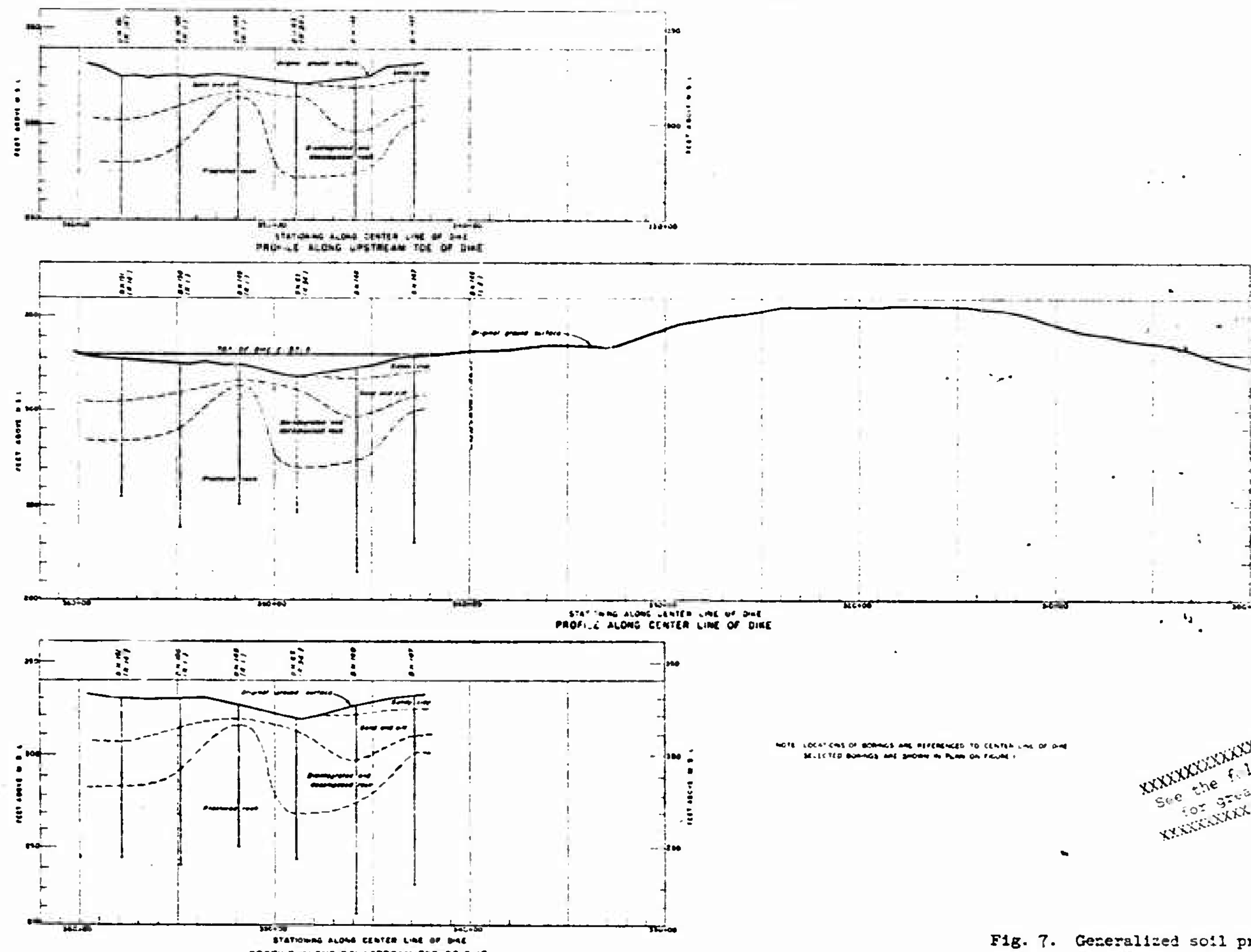

See the following pages for greater detail.

Fig. 7. Generalized soil profiles, Buggs Island Dam, sta 300+00-362+00

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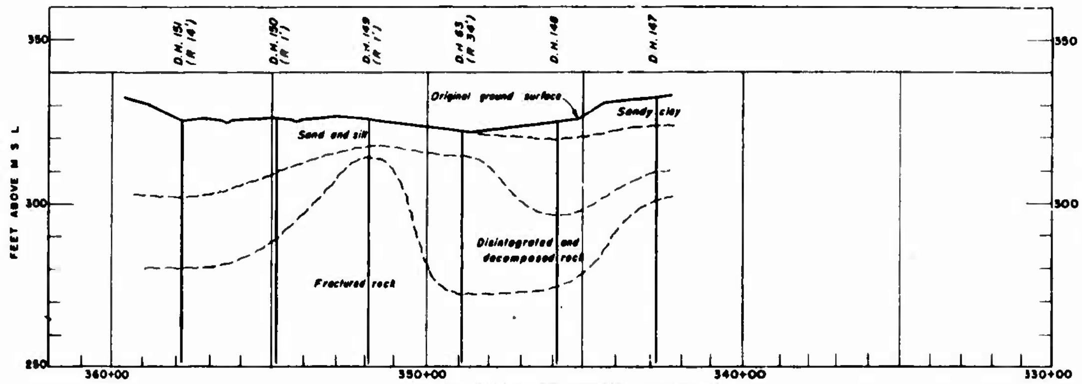
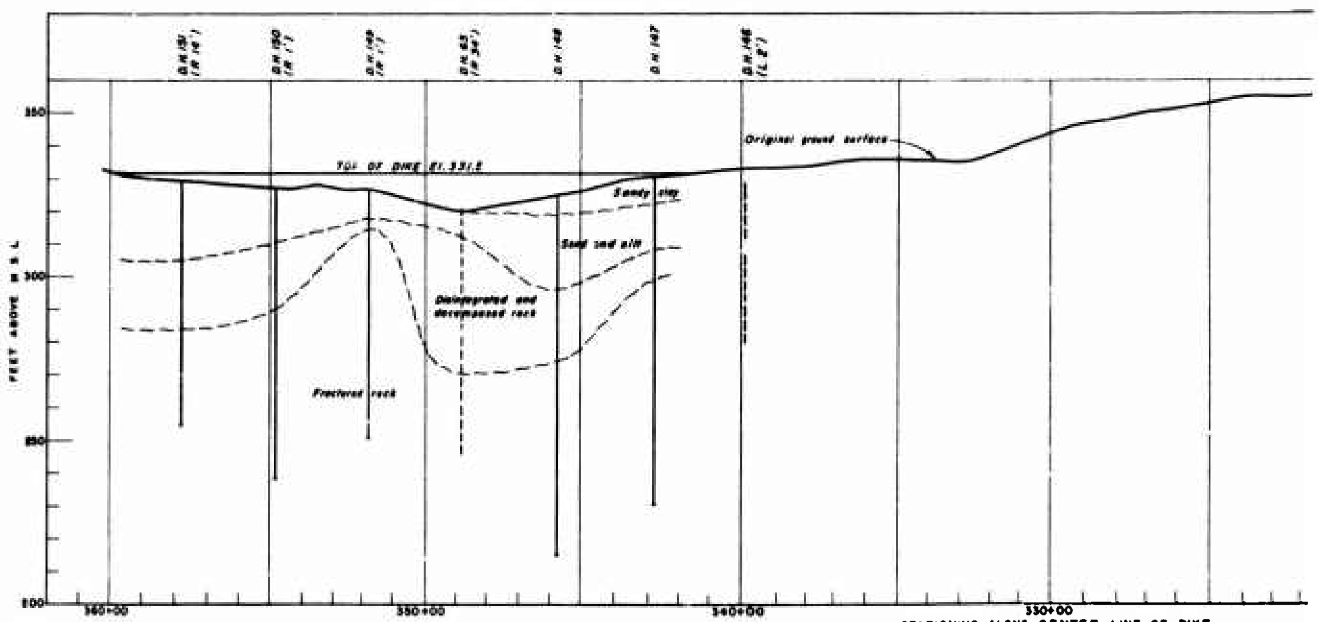
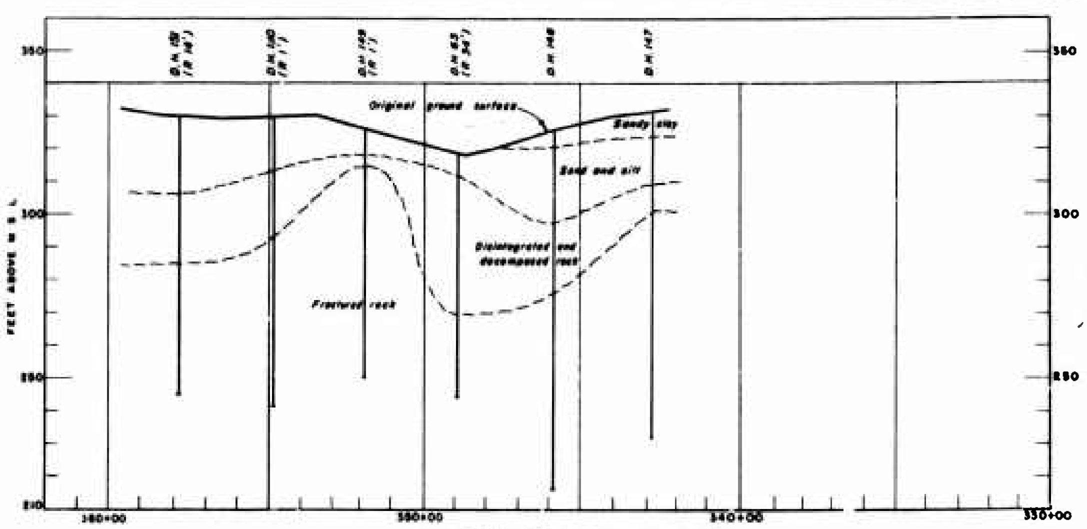


PROFILE ALONG UPSTREAM TOE OF DIKE



PROFILE ALONG CENTER LINE OF DIKE



PROFILE ALONG DOWNSTREAM TOE OF DIKE

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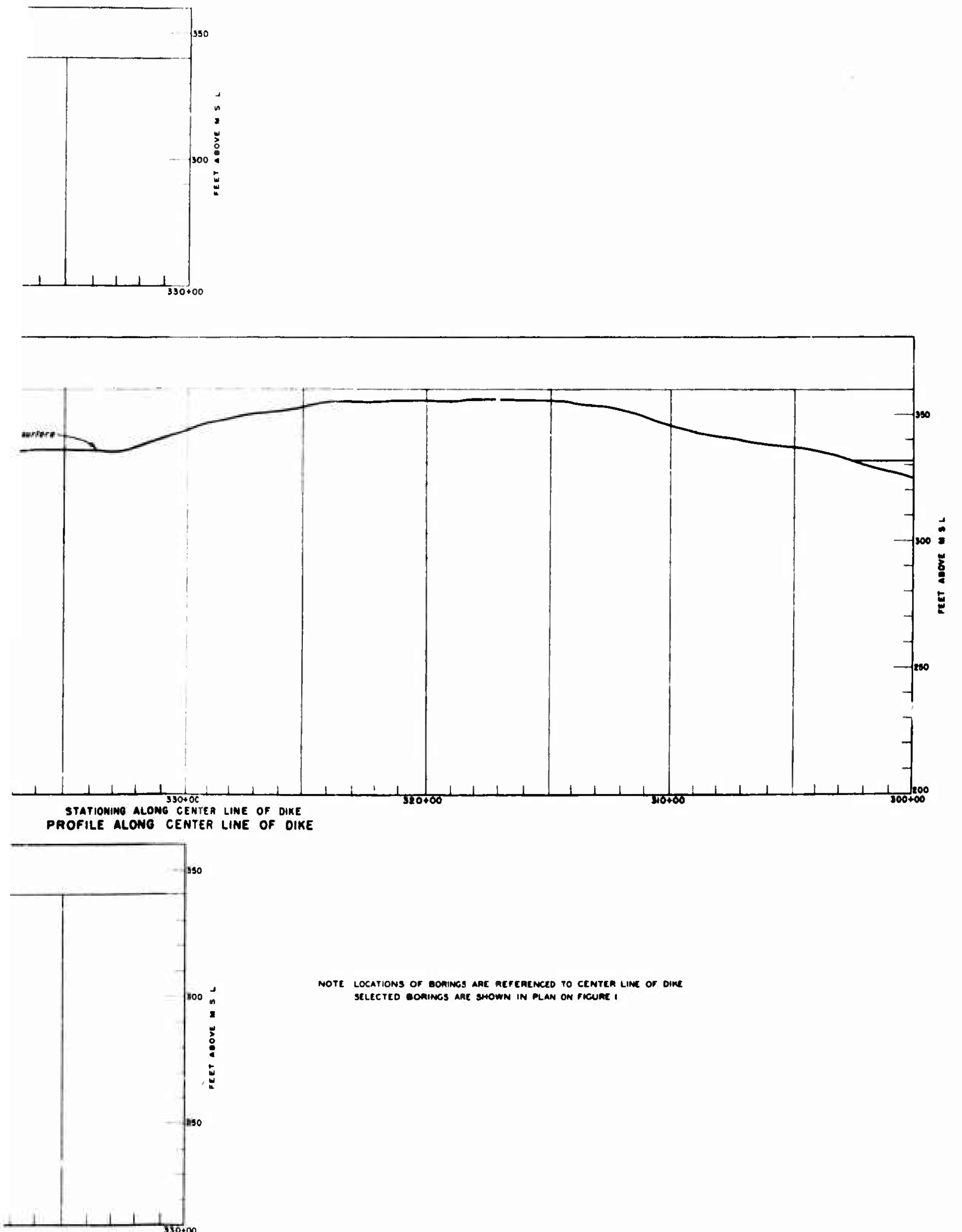

Fig. 7. Generalized soil profiles, Buggs Island Dam, sta 300+00-362+00

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Table 2

Summary of Test Data, Foundation and Borrow Areas, Buggs Island Dam

Boring	Sample	Sample Depth FT	(1) Laboratory Classification	Mechanical Analysis (2) Gvl %	Sand %	Fines %	D_{10} MM	Atterberg Limits (3) LL	PL	SP GR	Nat w %	Nat Dry Density LB/CU FT	Compaction Opt w %	Max Dry Density LB/CU FT	Shear Data Initial e	Dry Density LB/CU FT	w_i %	w_f %	S_i %	Type Test	Specimen Size IN.	No.	σ_{III} TONS/SQ FT	σ_I TONS/SQ FT	c TONS/SQ FT	ϕ DEG	Permeability e	k(5) CM/SEC X 10^{-4}	Consolidation Data P_o TONS/SQ FT	P_c TONS/SQ FT	C_v(6) SQ FT/MIN X 10^{-4}	r (7)	Remarks
TP-1	1-A	1.5	Silty clayey sd†	4	52	44	--		28		17.5																0.63	12.2			24.0	0.36	
	1	4.0	Silty sd	5	55	40	0.004		23		25.9							29.5		DS-CD	(4)				0.40	27.2	0.71	5.3					
																		22.3		DS-CD					0.23	29.4							
	2	9.0	Silty grvly sd	13	67	20	0.018				17.5							19.4		DS-CD	(4)				0.00	32.6	0.66	1.8					
TP-2	1-A	1.5	Silty clayey sd	2	48	50	--		31		21.2																0.66	253.1			11.8	0.27	Cracks in smpl
	1	4.0	Silty sand	1	72	27	0.011				23.8							28.1		DS-CD	(4)				0.45	33.0	0.78	2.6					
																		27.6		DS-CD					0.15	28.8							
	2	11.5	Gravelly sd	19	74	7	0.075				9.9							14.8		DS-CD	(4)				0.17	37.8	0.31	121.9					
																		19.1		DS-CD					0.00	32.0							
TP-3	1-A	1.5	Sdy silty clay	--	38	62	--		29		17.0																0.62	59.6			6.0	0.36	
	1	4.6	Silty clayey sd	--	72	28	0.001		27		22.5							27.5		DS-CD	(4)				0.00	33.2	0.84	128.5					
			Silty clayey sd	6	67	27												23.1		DS-CD					0.25	29.4							
	2	16.1	Gravelly sd	17	81	2	0.190				6.5																0.66	103.7					
	3	24.5	Gravelly sd	17	77	6	0.090				11.3																	18.2					
TP-6	1-A	1.5							37																			24.5					
	1	4.0	Silty clayey sd	--	54	46	--				25.5							30.1		DS-CD	(4)				0.38	25.4	0.86	24.8					
																		28.6		DS-CD					0.08	27.7							
	2	8.1	Silty sd	--	69	31	0.012				20.7							34.0		DS-CD	(4)				0.08	29.9	0.97	130.7					
																		26.7		DS-CD					0.00	28.8							
	3	11.5	Silty sd	--	74	26	0.017				18.9							31.5		DS-CD	(4)				0.45	24.2	1.14	70.0*					
																		26.8		DS-CD					0.00	27.7							
	4	15.5	Silty sd	2	78	20	0.021				18.6																1.11	95.9*					
	5	22.0	Sd	3	83	14	--				13.8																1.02	11.2					
	6	25.5	Silty sd	4	79	17	--				13.3							24.8		DS-CD	(4)				0.40	28.3	0.94	14.8					
																				DS-CD					0.00	32.0							
	7	32.0	Silty sd	6	76	18	0.027				23.1																						
TP-7	1-A	1.5							29																			4.8					
	1	4.5	Sd, silt, clay	1	43	56	--		40		29.0							28.7		DS-CD	(4)				0.38	28.8	0.91	11.9					
																		31.4		DS-CD					0.30	24.9							
	2	8.5	Silty sd	9	77	14	0.042				22.9							35.2		DS-CD	(4)				0.25	33.8	0.96	53.2					
																		24.7		DS-CD					0.05	32.0							
	3	13.0	Sd	2	87	11	0.056				21.4							24.6		DS-CD	(4)				[illegible]	35.5	0.89	7.5					
																		23.3		DS-CD					0.00	28.2							
B-A‡	Comp	0-24	Silty sd	1	62	37	--						14.0	112.5		109.8	11.0	22.8		DS-CD	(4)				0.60	25.5							
B-A	B-4	-	Sand, clay										21.5	100.5																			
B-A	B-35	-	Sand & silt										14.0	110.0																			
B-A	B-23	-	Sand & silt										15.0	109.2																			
B-B‡	Comp	0-24	Silty sd	--	64	36	--						12.8	116.0		113.5	9.8	20.7		DS-CD	(4)				0.32	29.4							
B-B	B-6	-	Sandy clay										21.4	99.5																			

NOTES: (1) When not shown, laboratory classification is not available.
(2) Sizes based on MIT classification: sand 2.0 to 0.06 mm; fines include silt and clay sizes < 0.06 mm.
(3) When not shown, limit data not available.
(4) Remolded test.
(5) Vertical permeability unless marked with asterisk which denotes horizontal permeability.
(6) Coefficient of consolidation at 0.8 T/sqft.
(7) Primary compression ratio at 0.8 T/sqft.

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† Sd = sand

‡ B-A is borrow area A; B-B is borrow area B.

DS—Direct Shear
CD—Consolidated Drained

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samples tested fall within the gradation bands for foundation soils shown in fig. 8. It may be noted that the top stratum which is nominally referred to as sandy clay contains 10-40 per cent clay sizes (<0.002 mm); the lower sand and silt stratum contains less than 10 per cent clay sizes.

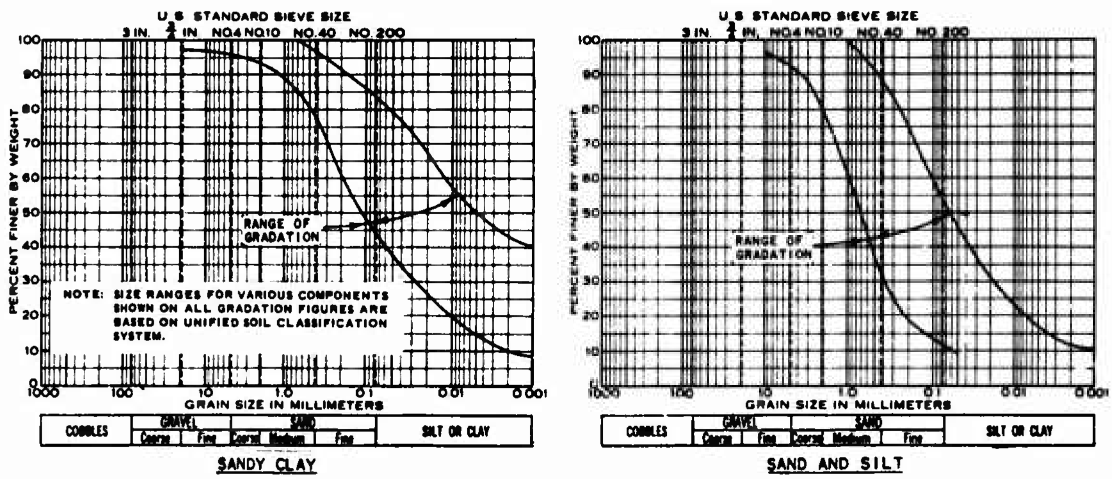


Fig. 8. Gradation of foundation soils, Buggs Island Dam

12. Nineteen consolidated-drained direct shear tests were performed on representative saturated samples of undisturbed foundation soils. All laboratory specimens were sheared at a rate of 0.001 in. per min. Representative shear strength curves are shown in fig. 9, as are appropriate

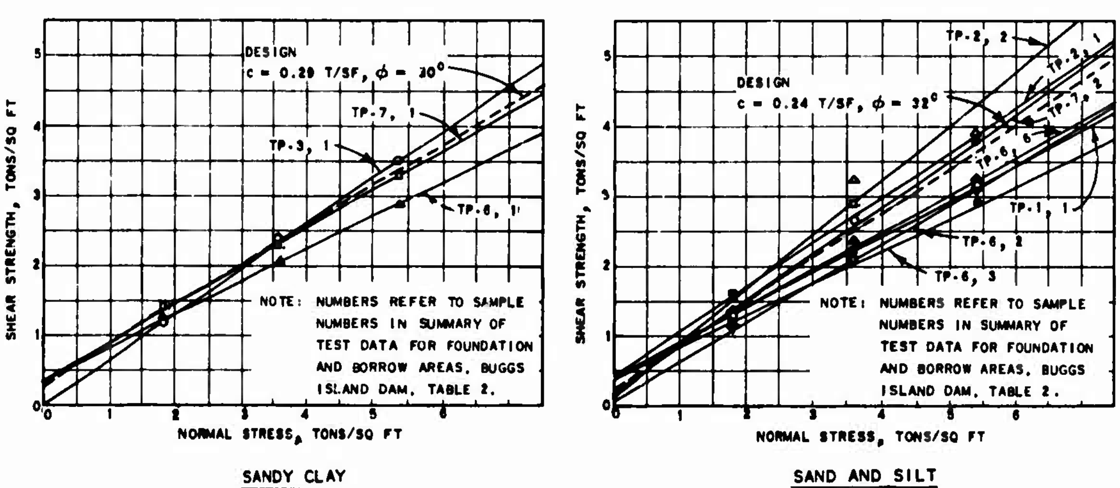


Fig. 9. Consolidated-drained direct shear strength curves and design shear strength curves for undisturbed foundation soils, Buggs Island Dam

design curves. A design shear strength of cohesion (c) = 0.29 ton per sq ft, angle of internal friction (ϕ) = 30°, was used for the upper clayey foundation soils, while a strength of c = 0.24 ton per sq ft, ϕ = 32°, was used for the lower sand and silts; the design shear strengths were the average results obtained from the shear tests. Shear tests were not performed on samples from the weathered rock zone.

13. Forty-one permeability tests were performed on undisturbed samples of the foundation materials. Generally the coefficient of permeability in the vertical direction was determined although a few horizontal permeability tests were also performed. The coefficients of permeability in the horizontal direction were generally less than 1.7 times the permeability in the vertical direction and for design purposes it was assumed that they were equal. Coefficients of permeability (k) varied from 2×10^{-4} cm per sec to 130×10^{-4} cm per sec with average values of 17×10^{-4} cm per sec and 36×10^{-4} cm per sec being selected for the upper and lower foundation soil strata, respectively. The zone of weathered rock was estimated to have a coefficient of permeability of 350×10^{-4} cm per sec; this estimate was based on data obtained from a "falling-head" test in a cased hole, and from pressure tests performed in the weathered zones of drill holes.

14. Standard laboratory consolidation tests were performed on 18 undisturbed foundation soil specimens. Plots of void ratio vs pressure (e vs log p) and consolidation vs time were made. Typical e vs log p curves are shown in fig. 10.

15. <u>Settlement and seepage analyses.</u> Since the dikes generally are low, little settlement in the foundation was expected. It was stated in the analysis of design that settlement of the dikes due to application of embankment load would be relatively small, but as a result of disturbances in the foundation

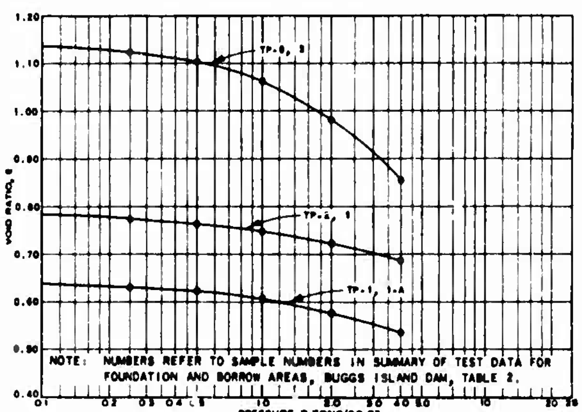


Fig. 10. Typical void ratio-pressure curves for undisturbed foundation soils, Buggs Island Dam

created by equipment used during construction operations some deformation could be expected to occur. It was further stated that it was not possible to determine in advance exactly how much the dike foundation would be disturbed by construction operations, but based on considerations of estimated depth of disturbance of foundation materials and on computed consolidation of deeper foundation zones, it was estimated that the total settlement would be 6-12 in. for most of the dikes, and that the deformation would be essentially complete at the end of construction. Methods used for estimating the amount and time rate of settlement are not known.

16. The analysis of the stability of the foundation is included in that made for the embankment and will be discussed in detail in the section on "Stability, settlement, and seepage analyses" under "Embankment."

17. Since the coefficient of permeability of the substratum of underlying weathered rock was estimated to be 350×10^{-4} cm per sec, underseepage was considered a potential problem. A flow net was drawn to estimate the quantity of seepage that would pass through the foundation and an analysis was made assuming the embankment to be impervious, the permeability of a 17-ft-thick topstratum of sand and silt to be 35.6×10^{-4} cm per sec, and the permeability of a 36-ft-thick stratum of weathered rock to be 350×10^{-4} cm per sec. Based on the flow net analysis for the maximum dike section with the reservoir pool at el 320, underseepage was estimated to be about 33,000 gpm. As the average dike section is considerably smaller than the maximum section analyzed, it was recognized that the total quantity of seepage would be less than 33,000 gpm, but nevertheless it was considered that an upstream impervious blanket would be desirable to reduce the underseepage.

18. Upstream impervious blanket. The upstream blanket was to be constructed by scarifying and recompacting the natural top stratum to a depth of approximately 12 in. in certain reaches below maximum power pool el 300 and those reaches above maximum power pool elevation that may be inundated by flood stages of relatively frequent occurrence; recompacted areas were to be topped with 6 in. of topsoil. The criterion for determining the length of reaches to be scarified and compacted for the upstream blanket was that the average hydraulic gradient should not be

greater than 0.1 between the upstream edge of the blanket and the downstream toe of the dike with the reservoir pool at el 320. Locations of the reaches so treated are shown on the profile along the upstream toe of the dike in figs. 4-6. A flow net and an estimate of underseepage considering the effect of the upstream blanket were not prepared.

19. Downstream toe drain. A downstream toe drain was also provided to penetrate the upper foundation clay stratum and contact the more pervious substrata of silt and sands. However, a substantial portion of the underseepage was expected to pass under the toe drain and seep out farther downstream from the dike.

Embankment

20. Borrow materials. Materials for the embankment were to be obtained from borrow areas located downstream from the embankments and from the required excavation for the toe drain. Four borrow areas investigated in the design study, A, B, C, and D, located on hills to the left and right of the reservoir area, are shown in plan in fig. 1; locations of 22 of 88 auger borings, made during exploration of the borrow areas, are also shown. The materials from both excavation and borrow areas were found to be similar; thus the material from the excavation for the toe drain was also used in the embankment. The materials from the borrow areas and toe drain excavation graded from residual sandy and silty clay near the surface to residual sand and silt to depths of about 10-30 ft; the strata of sandy and silty clay, and sand and silt are not distinct but grade gradually from one to the other. Therefore, these materials were to be placed in an unzoned embankment section, and placement was to be controlled to produce a uniform, well-knit embankment of low permeability and relatively high stability. The expected types and volume of embankment materials as well as the sources are given in table 3.

Table 3

Sources and Quantities of Embankment Materials, Buggs Island Dam

Type	Quantity, cu yd	Source
Earth fill	970,000	Dike toe-drain excavation and borrow areas
Filter gravel and riprap bedding	65,000	Concrete aggregate plant
Dumped riprap	110,000	Rock excavation for masonry dam

21. Laboratory tests. Laboratory tests on borrow material were performed for purposes of classification and determination of remolded properties. A summary of test results is presented in table 2. The range of gradation of the borrow soils is shown in fig. 11. The finer grained soils are the top stratum clays and the coarser soils are the underlying sand and silt.

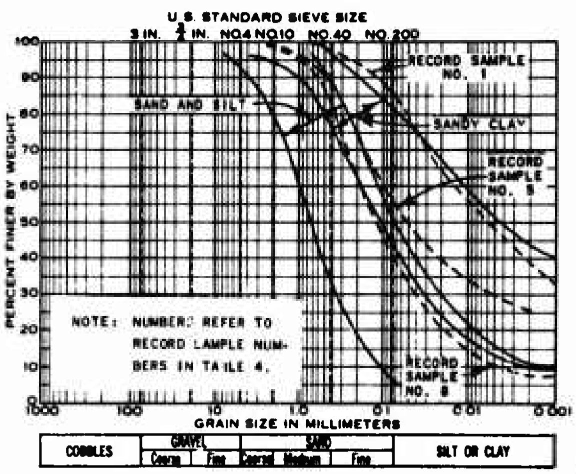


Fig. 11. Gradation of borrow materials and typical embankment record sample gradation, Buggs Island Dam

22. The standard effort compaction test was used in the determination of laboratory compaction characteristics. Typical laboratory compaction curves for the borrow materials are shown in fig. 12. The standard density ranged from 99.5-112 lb per cu ft and the optimum water content ranged from 21-14 per cent.

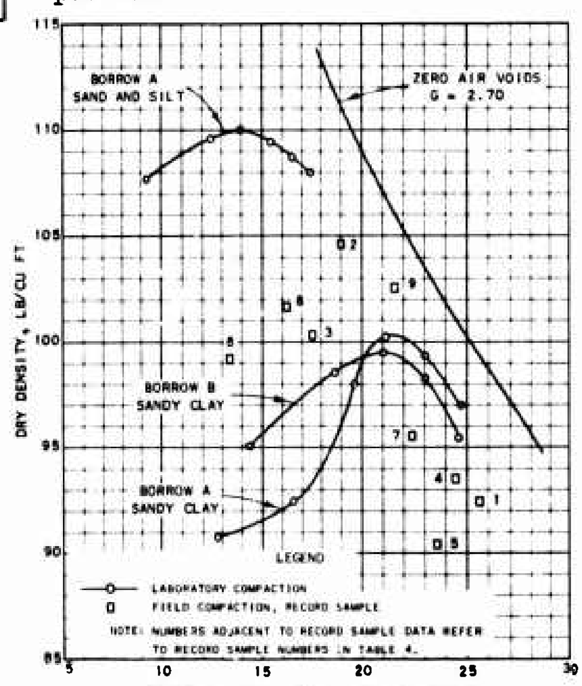
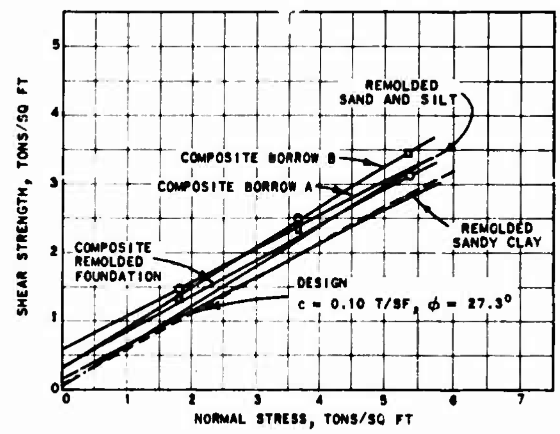


Fig. 12. Typical laboratory standard effort compaction curves for borrow materials and record samples, Buggs Island Dam

23. Results of permeability tests on compacted borrow material were not available for this review. However, 25 permeability tests were performed on remolded samples of foundation materials which were similar to the materials to be used for the embankment. Coefficients of permeability normal to the direction of the compaction planes ranged from 3.8×10^{-4} cm per sec to 0.0001×10^{-4} cm per sec; the coefficient of permeability parallel to the direction of the

compaction planes was not determined. An average permeability of 0.0024×10^{-4} cm per sec, in both the horizontal and vertical directions, was used in seepage analyses for the embankment.

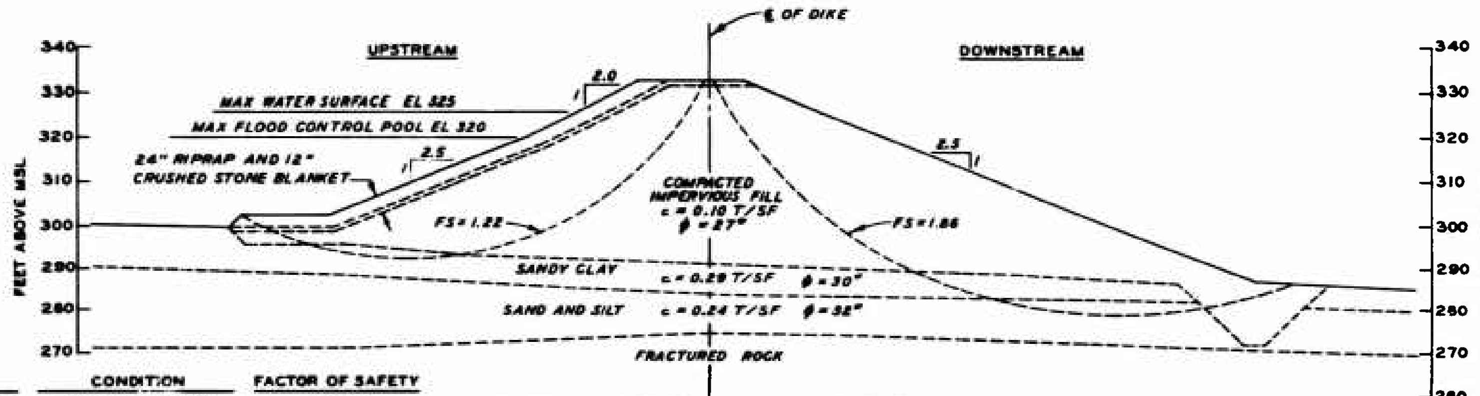
24. Two consolidated-drained direct shear tests were performed on saturated composite samples of materials from each borrow area with the samples compacted to 98 per cent of modified AASHO density and at water contents 3 per cent dry of optimum; these test data are shown in table 2. Twenty-six other consolidated-drained direct shear tests were performed on remolded samples of borrow area and foundation soils; results of these tests were also considered in selecting the design shear strength for the embankment. The strength curves for the compacted borrow material and the average remolded strength curves for the foundation soils, as well as the design shear strength curve for the embankment, c = 0.10 ton per sq ft, $\phi = 27^{\circ}$, are shown in fig. 13.



MATERIALS	% COMPACTION	SHEAR STRENGTH c, T/SF	SHEAR STRENGTH ϕ°	AVERAGE γ_D	AVERAGE w_i	AVERAGE w_f
COMPOSITE A	98% MOD	0.60	25.4	109.8	11.0	22.8
COMPOSITE B	98% MOD	0.35	29.4	113.5	9.8	20.7
AVG REMOLDED SANDY CLAY	—	0.18	25.0	—	—	—
AVG REMOLDED SAND AND SILT	—	0.07	30.3	—	—	—
AVG REMOLDED COMPOSITE FDTN	—	0.35	27.3	—	—	—
DESIGN	—	0.10	27.3	—	—	—

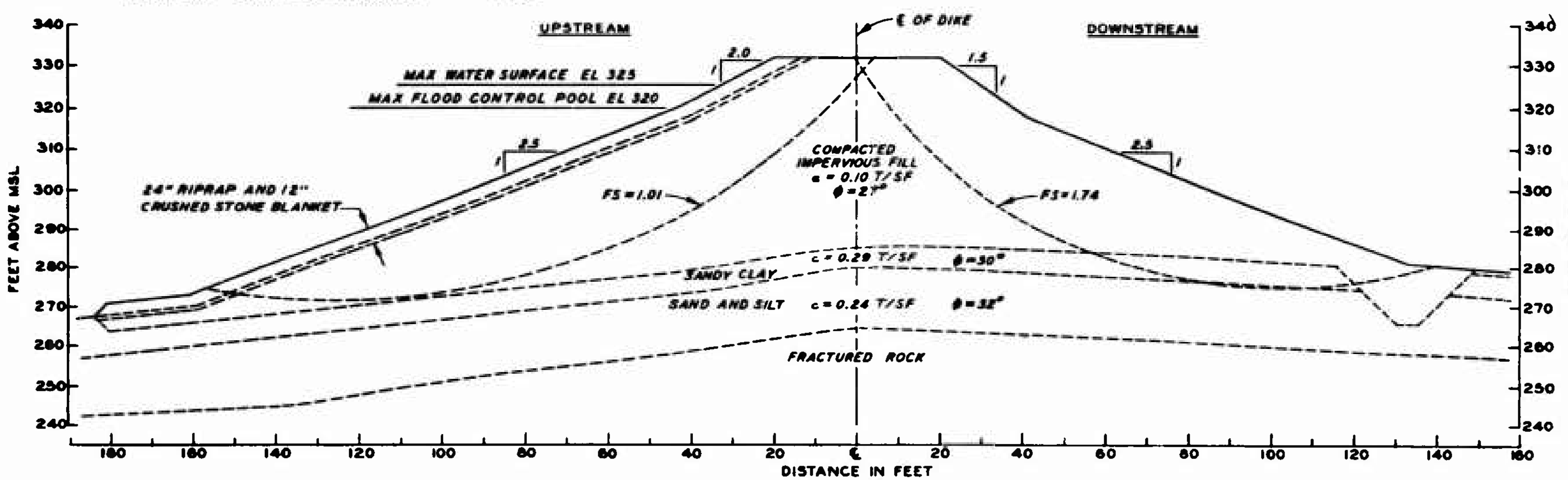
Fig. 13. Consolidated-drained direct shear strength curves for compacted composite samples from borrow areas A and B, average remolded foundation soils and design shear strength curve for embankment, Buggs Island Dam

25. Stability, settlement, and seepage analyses. Stability analyses for embankments and foundations of the dikes were made using the circular arc method and shear strengths determined by the consolidated-drained direct shear test. Analyses were made for maximum dike sections on both left and right banks of the river. Stability arcs for these sections are shown in fig. 14. Upstream slopes were analyzed for a condition of rapid drawdown from a flood control pool at el 320. Downstream slopes were analyzed for the condition of complete saturation. Computed minimum factors of safety are shown in fig. 14. A minimum factor of safety of



MAXIMUM SECTION - LEFT DIKES

ARC		CONDITION	FACTOR OF SAFETY
UPSTREAM	LEFT DIKE	RAPID DRAWDOWN	1.22
	RIGHT DIKE	RAPID DRAWDOWN	1.01
DOWNSTREAM	LEFT DIKE	COMPLETE SATURATION	1.86
	RIGHT DIKE	COMPLETE SATURATION	1.74



MAXIMUM SECTION - RIGHT DIKES

Fig. 14. Stability analyses for embankments and foundations, Buggs Island Dam

1.01 was obtained for the upstream slope of the right dike for the rapid drawdown condition. Because the assumption of rapid drawdown is conservative, and since the maximum section applies only to a limited length of dike (approximately 45 ft on the right bank), the factor of safety of 1.01 for this condition was considered satisfactory.

26. The dikes are relatively low and little settlement was expected from consolidation within the embankment. No settlement computations were made.

27. The quantity of seepage through the embankment was not considered significant since the coefficient of permeability of the pervious substrata was approximately 8000 times that of the compacted embankment. However, a flow net for the embankment was prepared assuming that the contact between the base of the embankment and the foundation was a free-draining face; on the basis of this assumption, it was estimated that the through seepage with the reservoir pool at el 320 would be about 0.3 gpm per 100 ft of dike for the maximum dike section. Actually the contact between the base of the embankment and the foundation is not a free-draining face because significant hydrostatic pressures will exist in the foundation when the pool stage is above the base of the embankment; for this reason the flow net for the embankment is not included in this report.

28. Drainage facilities. A toe drain approximately 15 ft deep was installed to collect through seepage and reduce hydrostatic pressures at the toe. It was to penetrate the upper clay material and extend a short distance into the more pervious sand and silt. The drain consists of an outer layer of filter sand and an inner layer of 3/4-in. filter stone surrounding a 12-in.-diameter, perforated drainpipe. Gradations of the filter materials are shown in fig. 15; the gradations are such that the D_{15} size of the filter is less

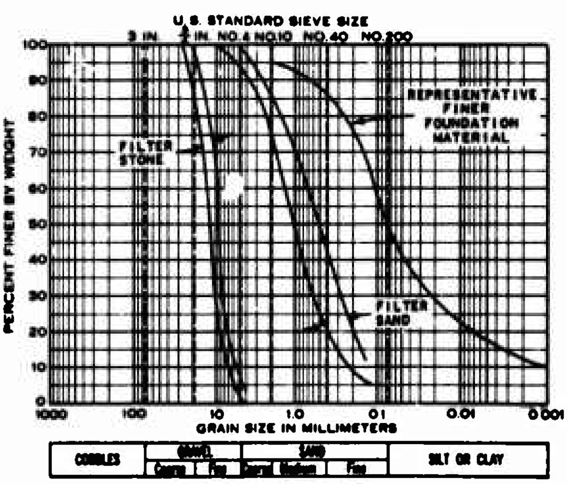


Fig. 15. Gradation of toe drain filter materials, Buggs Island Dam

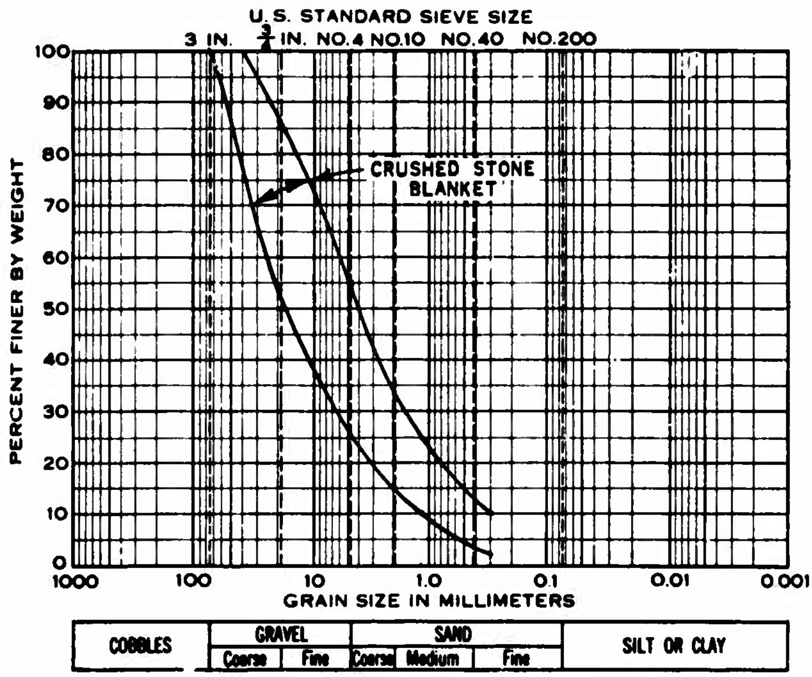


Fig. 16. Gradation of crushed stone blanket for riprap, Buggs Island Dam

than four times the D_{85} size of the protected material. (A detail of the toe drain is shown in fig. 18.)

29. Slope protection. Upstream or reservoir slopes are protected by 24 in. of dumped riprap placed upon a 12-in. crushed stone blanket; downstream slopes are protected by vegetative cover. The report, Slope Protection for Earth Dams,* was used for design of the upstream slope protection. Waves four feet in height were anticipated over a fetch of four miles at a wind velocity of 40 mph. The minimum average riprap size was to be 15 in.; gradation of the crushed stone blanket is shown in fig. 16.

Spillway and outlet works

30. A discussion of the foundation for the spillway and outlet works is not included in this report since these structures are founded on rock. No engineering measurement devices were provided for these structures.

Construction

Borrow materials

31. Borrow materials were obtained from parts of borrow areas B, C, and D, which had been investigated during design of the structures, and also from other areas located closer to the embankments. Locations of borrow areas actually used during construction are shown as shaded

* Corps of Engineers, Office, Chief of Engineers, Slope Protection for Earth Dams, Vicksburg, Miss., March 1949.

areas in fig. 1. The materials placed in the embankment were a composite of clay with sand and silt obtained by mixing borrow materials during both excavation and placement. Draglines and belt loaders were used in borrow area excavation. Crushed rock for riprap was obtained from the excavation for the masonry dam and a nearby quarry. No difficulty was experienced during excavation of borrow materials. The materials obtained were satisfactory.

Embankment

32. Earthwork. The construction of the embankment was begun after initial clearing and grubbing were completed. The dikes were compacted by a minimum of 6 passes of tamping rollers. Moisture content of the compacted material was to be within 2 per cent dry and 4 per cent wet of the optimum moisture content as determined by the standard compaction test. The embankment material was compacted to 95 per cent of standard density as determined by the standard compaction effort test.

33. Each lift of material was required to have the proper moisture content before being compacted. When too dry, the material was sprayed by truck-mounted sprinkler tanks; when too wet, or when the preceding layer was too smooth, the material was disked or harrowed prior to rolling. Stones greater than 6 in. in diameter, rock, trash, and debris were removed from each lift before rolling.

34. The material was placed loose in 8-in. lifts and compacted with sheepsfoot rollers providing a contact foot pressure of 500 psi on L-shaped feet having a cross-sectional area of 7 sq in. No difficulties were encountered during the compaction operations. No records are available to indicate whether additional rolling was necessary to meet the specifications.

35. A total of 529 field density tests were made on the compacted dikes; this is an average of one test for about each 2200 cu yd of material placed. The sand-density method was used in performing the tests. A bag sample of soil was taken adjacent to each density test location for laboratory compaction tests. The measured field density was then compared with results of the laboratory tests. Moisture contents were determined by rapid drying in the field; samples were also taken for

laboratory check of field determinations. As the work of field testing progressed, field drying of samples was discontinued and oven drying was then used as a check of visual field determinations. Thus, moisture content was used as the basic control measure, but density measurements were used to determine whether the specified rolling was adequate to provide the desired 95% of standard density.

36. Densities equal to or greater than 95% of standard density were obtained for 99% of the field density samples. Only two tests showed less than 95% of standard density; soil in these locations was removed from the fill because of inadequate compaction and replaced with adequately compacted material. Field tests also indicated that 93% of all tests were within the specified moisture content range.

37. Record samples. Nine undisturbed box samples were taken for record purposes during various stages of construction at various locations. A summary of results of tests on the record samples is given in table 4. Mechanical analyses and density, shear, permeability, and consolidation tests were made on all record samples. A comparison of the gradation of typical record samples with the ranges of borrow material gradation is shown in fig. 11; the materials as actually placed were predominantly sandy clays, a result of the normal mixing of the borrow material as it was placed in the embankment.

38. Density tests were made on record samples as obtained from the embankment. A comparison of these densities with laboratory compaction curves obtained from composite borrow materials is shown in fig. 12.

39. A consolidated-drained direct shear test was made on a saturated specimen from each record sample at a rate of strain of 0.001 in. per min. Shear strengths varied from c = 0.15 ton per sq ft to 0.60 ton per sq ft and $\phi = 25^{\circ}$ to 31°. The design strength was c = 0.10 ton per sq ft and $\phi = 27^{\circ}$. Record sample shear strength curves and the design strength curve are plotted in fig. 17. A comparison of these curves shows that record sample strengths were greater than the design strength.

40. The coefficient of horizontal permeability was determined for a specimen from each record sample; values ranged from 0.001×10^{-4} cm per sec to 0.070×10^{-4} cm per sec. An average value of $k = 0.015 \times 10^{-4}$

Table 4

Summary of Test Data, Record Samples, Buggs Island Dam

BORING	SAMPLE	SAMPLE EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS (1) GVL %	SAND %	FINES %	D_{10} MM	ATTERBERG LIMITS LL	PI	SP GR	NAT w %	NAT DRY DENSITY LB/CU FT	COMPACTION OPT w %	MAX DRY DENSITY LB/CU FT	SHEAR DATA INITIAL e	DRY DENSITY LB/CU FT	w_i %	w_f %	S_i %	TYPE TEST	SPECIMEN SIZE IN.	NO.	σ_{iii} TONS/SQ FT	σ_i TONS/SQ FT	c TONS/SQ FT	ϕ DEG	PERMEABILITY e	k_v CM/SEC $\times 10^{-4}$	CONSOLIDATION DATA P_o TONS/SQ FT	P_c TONS/SQ FT	C_v(2) SQ FT/MIN $\times 10^{-4}$	r(3)	LOCATION
	1	302.4	CH	0	15	85	-	65	22	2.70	25.6	92.5						28.3		DS-CD					0.60	25.4	0.83	0.003			8.8	0.46	90+35 - 10 US
	2	315.4	ML	1	29	70	-	35	9	2.68	19.0	104.8						20.3		DS-CD					0.28	30.5	0.58	0.001			19.8	0.36	90+00 - ℄
	3	314.5	SM	5	53	42	-	NP		2.67	17.8	100.3						19.7		DS-CD					0.25	30.7	0.67	0.070			15.8	0.36	180+00 - 12.5US
	4	318.0	ML	0	45	55	-	47	17	-	24.4	93.3						25.5		DS-CD					0.25	26.6	0.74	0.003			13.2	0.38	190+00 - 12.5US
	5	300.0	ML	0	45	55	-	44	12	2.64	23.6	90.6						27.0		DS-CD					0.20	27.2	0.80	0.014			11.8	0.44	238+00 - 12 DS
	6	315.0	ML	0	43	57	-	NP		2.65	17.1	101.8						21.7		DS-CD					0.22	28.8	0.63	0.002			13.6	0.38	238+00 - ℄
	7	291.0	MH	0	39	61	-	44	11	2.69	22.4	95.5						24.9		DS-CD					0.35	25.4	0.78	0.015			7.9	0.35	261+00 - ℄
	8	311.0	SM	0	57	43	0.005	NP		2.62	13.4	99.1						22.9		DS-CD					0.15	29.4	0.64	0.025			15.3	0.39	261+00 - ℄
	9	311.0	ML	0	39	61	-	46	15	2.64	21.8	102.6						23.4		DS-CD					0.30	29.4	0.62	0.005			7.2	0.43	282+00 - ℄

NOTES: (1) Soils were classified according to Unified Soil Classification System of Corps of Engineers in use at the time tests were made. Sand size range was 2.0-0.074 mm; fines include silt and clay sizes, <0.074 mm.
(2) C_v = Coefficient of consolidation at 1 T/sq ft.
(3) r = Primary compression ratio at 1 T/sq ft.

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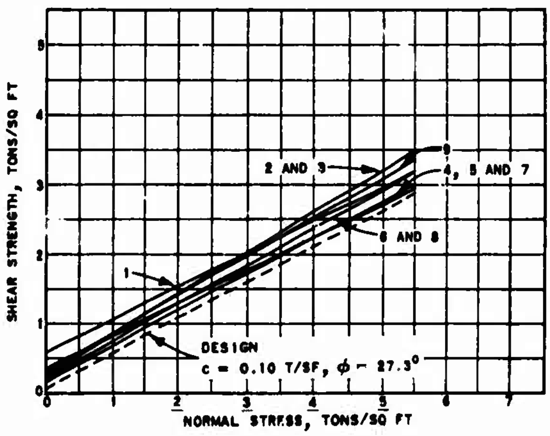
DS – Direct Shear
CD – Consolidated Drained



SAMPLE NO.	SHEAR STRENGTH c, T/SF	$\phi°$	AVERAGE γ_D	w_f
1	0.60	25.4	92.5	28.3
2	0.28	30.5	104.8	20.3
3	0.25	30.7	100.3	19.7
4	0.25	26.6	93.3	25.5
5	0.20	27.2	90.6	27.0
6	0.22	28.8	101.8	21.7
7	0.35	25.4	95.5	24.9
8	0.15	29.4	98.1	22.9
9	0.30	29.4	102.6	23.4
DESIGN	0.10	27.3	—	—

Fig. 17. Record sample shear strength curves, Buggs Island Dam

cm per sec was determined for the embankment. The design coefficient of permeability was k = 0.0024 x 10^{-4} cm per sec. Even though the actual permeability is approximately six times as great as the design permeability, the quantity of seepage through the embankment would be minor since an estimated quantity of only 0.3 gpm per 100 ft of embankment (60 gpm for the entire embankment) was obtained using the design permeability and maximum section.

Downstream toe drain

41. The filter sand and filter stone surrounding the pipe in the toe drain section were compacted by hand-operated tampers. Embankment fill was compacted by lightweight equipment to a depth of 5 ft above the filter materials. Compaction around manhole riser pipes was performed with hand-operated pneumatic tampers.

Upstream impervious blanket

42. The upstream impervious blanket (see paragraph 18) was constructed of materials in place. The blanket area was scarified to a depth of 12 in., rolled in accordance with embankment compaction requirements, and topped by 6 in. of topsoil.

Slope protection

43. Material for the crushed stone blanket was obtained from a quarry which was also the source of concrete aggregate and most of the riprap. A small amount of riprap for the left bank embankment was obtained also from the toe drain excavation for the left wing dike. The crushed stone blanket consisted of minus 3-in. crushed rock graded as shown in fig. 16.

44. Blanket material was hauled to the embankment in dump trucks. Some of the material was placed by a crane-operated clamshell bucket and skip pan but generally this material was placed by controlled dumping from loaded dump trucks which were backed down the embankment. Before a truck was backed down the embankment it was attached by a cable to a tractor so that it could be safely and accurately lowered to the desired location. After the truck had been lowered to placement position the tailgate was opened and the material was distributed as the truck was pulled up the embankment. Riprap was placed by a crane-operated clamshell and skip pan. Final grading of the blanket and placement of riprap were accomplished by hand.

Prototype Observations

Settlement

45. Computations of embankment and foundation settlement were not made. It was considered that estimated foundation settlements of only 6-12 in. would occur during construction of the dikes. No provision was made for settlement observations during or after construction.

Seepage

46. As stated earlier, underseepage was estimated at 33,000 gpm for an anticipated maximum head of 40 ft, and little seepage was expected through the embankment. The toe drain installation, located as shown in figs. 4, 5, 6, and 7, was expected to intercept some underseepage.

47. To date, measurable seepage has been observed from only one outfall pipe at sta 263+00 which drains about 2500 ft of the right wing dike. On 19 August 1953, with the reservoir pool at el 296.5 and an average head of about 12 ft on this section of dike, a flow of 30 gpm was measured. On 29 February 1956, with the reservoir pool at el 289.7 and an average head of about 8 ft on the dike, the flow was about 20 gpm.

48. The highest reservoir stage to date, el 305, occurred in August 1955. Readings from piezometers installed in the foundation have been taken at various times and now are being read monthly. The piezometer tips are located immediately above the decomposed rock stratum and

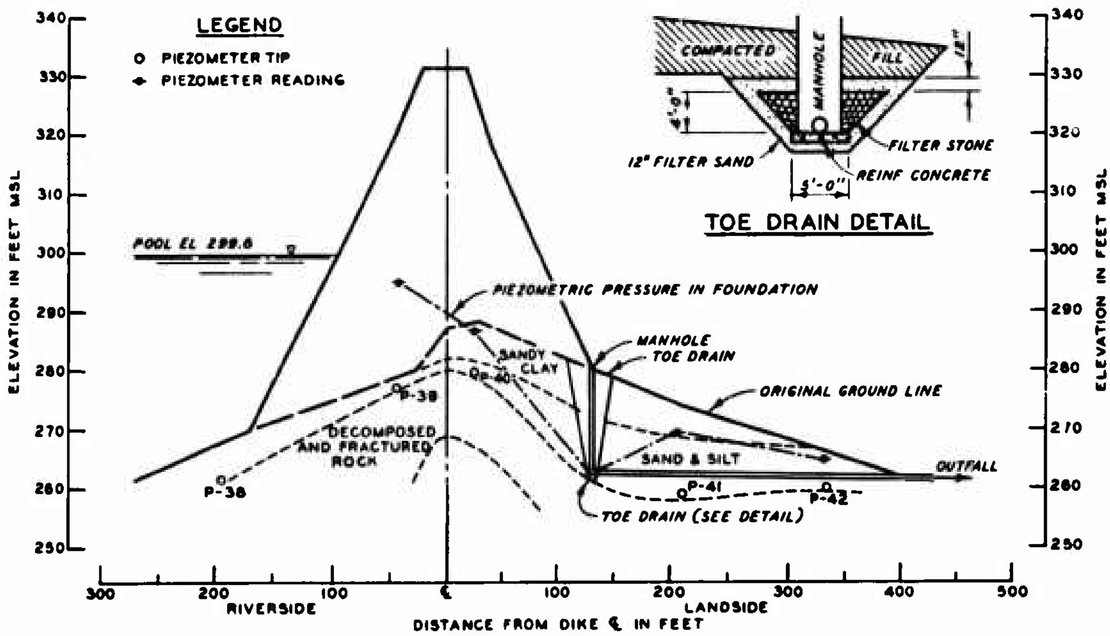


Fig. 18. Piezometer readings at sta 261+18 for pool elevation of 299.6 ft, Buggs Island Dam

consist of 1-1/2-in.-diam by 18-in.-long, brass-jacket wellpoints placed in 6-in.-diam by 2-ft-6-in.-long pockets of clean coarse sand. Piezometers at sta 261+18 are shown in cross section in fig. 18, and a plot of piezometric pressure observed in July 1953 for a pool elevation of 299.6 is also shown. No apparent excess pressures are present in the dike foundation at this maximum section. Variations in reservoir level and piezometer readings vs time at sta 261+18 are shown in fig. 19. The data show

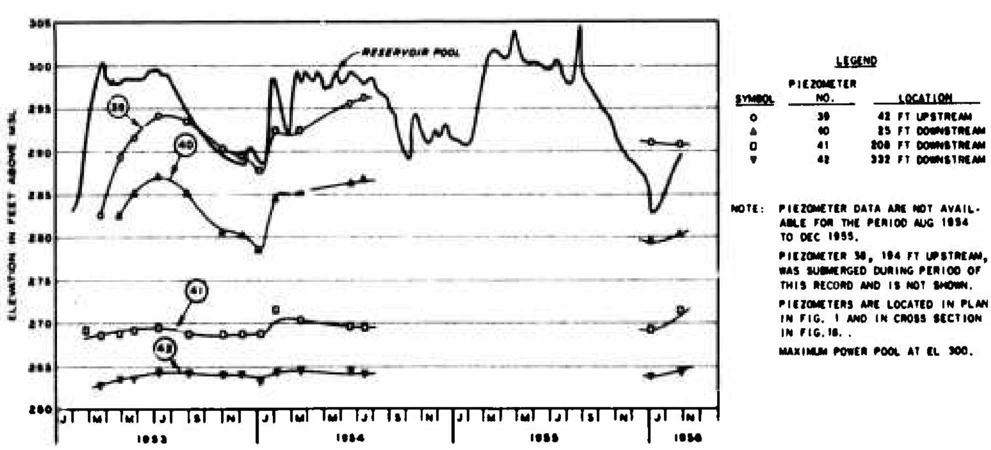


Fig. 19. Piezometer readings and reservoir pool, sta 261+18, Buggs Island Dam

that hydrostatic pressures in the foundation generally vary with the reservoir stage and to date no excess pressures at the downstream toe have been observed.

Riprap

49. Maximum wave heights observed to date are estimated to have been about 2 ft; no disturbance of the riprap has been reported.

PART III: ISLAND CREEK DAM

Design

Foundation

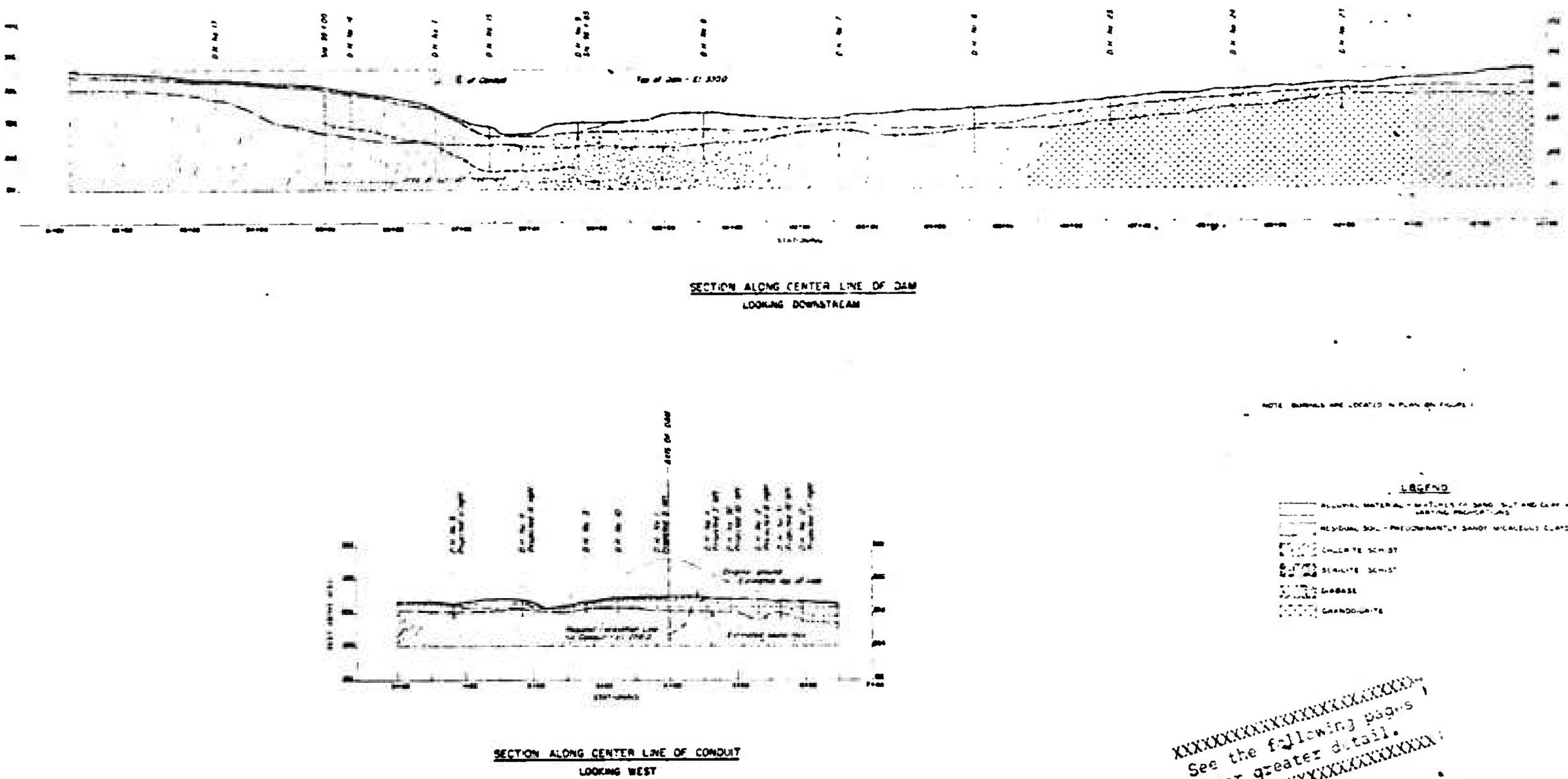
50. Geology. Island Creek Dam also lies within the Piedmont physiographic area consisting mainly of igneous and metamorphic rock underlying a moderately thick to thick overburden. Schists, diabase, and granodiorite form the rock foundation for the structure. The west abutment area is steep and has little overburden. The east abutment area is more gently sloping and has an average depth of overburden of 16 ft. Large faults or shear zones were not found within the dam site, but local minor shear zones were found along the conduit foundation in the weathered rock. The foundation for the conduit was excavated below grade to reach sound rock in areas where minor shear zones were found.

51. Field explorations. The field exploration program to determine foundation conditions along the dam and outlet conduit consisted of 31 core drill holes and 10 test pits. Locations of the borings are shown in fig. 1; generalized soil profiles along the dam and conduit are shown in fig. 20.

52. The overburden beneath the dam was found to vary from 2-36 ft in depth and to consist of sands, micaceous silts, and clays with no apparent uniform stratification. Where the overburden was greater than 15-20 ft in depth it was considered sufficiently thick to prevent critical seepage into the weathered rock. Where the overburden was less than 15 ft in depth, a grout curtain was placed to limit the seepage through the weathered rock of the foundation. The location of the cutoff treatment is shown in profile in fig. 20.

53. Laboratory tests. The laboratory testing program for foundation materials consisted of mechanical analyses, classification, shear strength, permeability, and consolidation tests. Results of tests on undisturbed foundation soils are presented in table 5.

54. Mechanical analyses were made on eight representative samples of foundation soils. The ranges of gradation for the two predominant soil



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for greater detail.
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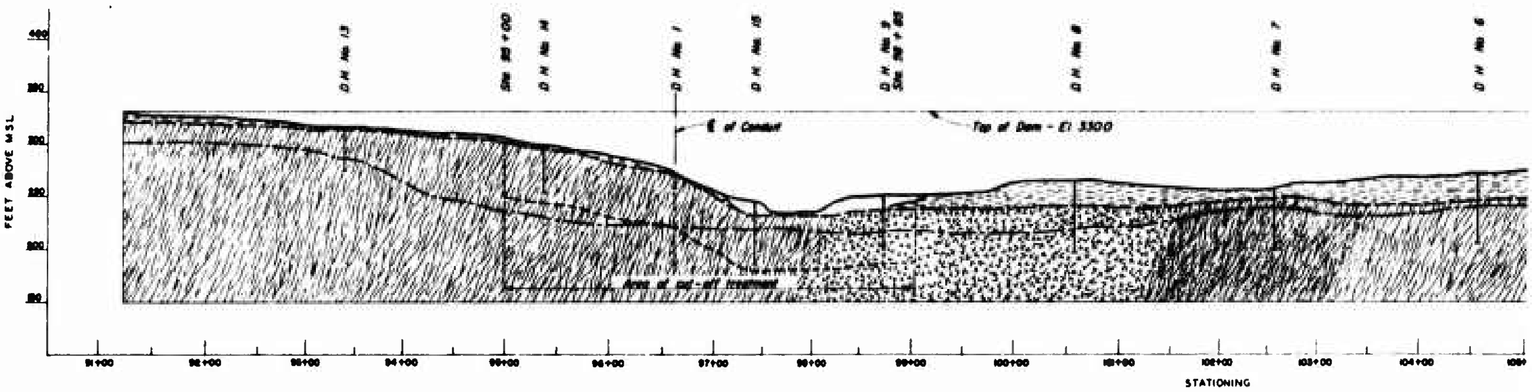
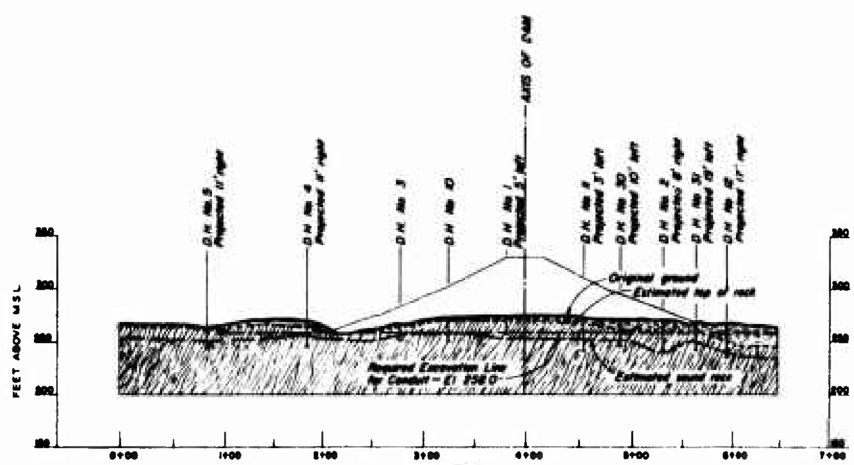
Fig. 20. Generalized soil profiles, Island Creek Dam



SECTION ALONG CENTER LINE OF DAM
LOOKING DOWNSTREAM



SECTION ALONG CENTER LINE OF CONDUIT
LOOKING WEST

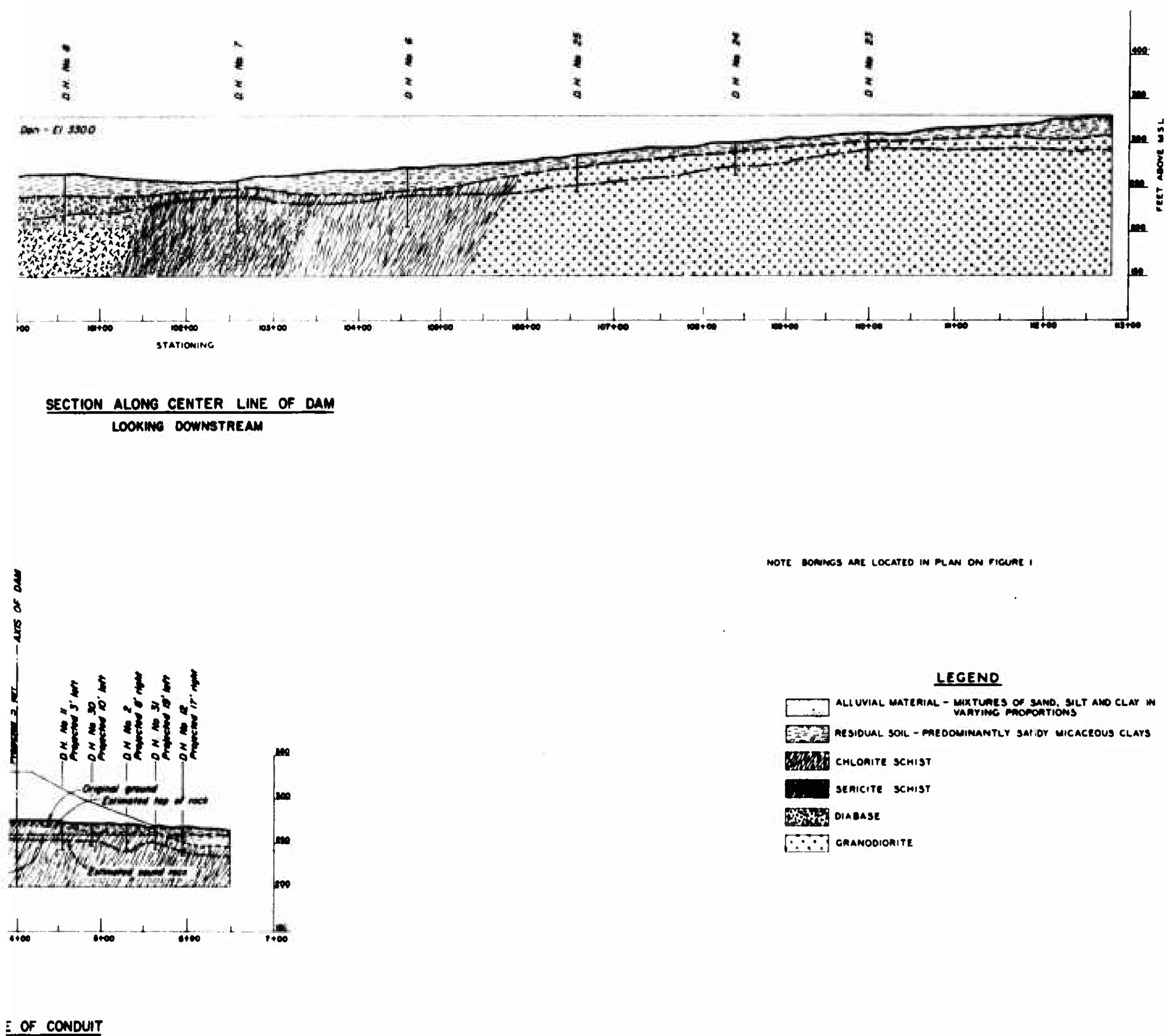


Fig. 20. Generalized soil profiles, Island Creek Dam

Table 5

Summary of Test Data, Foundation and Borrow Areas, Island Creek Dam

Boring	Sample	Sample Depth Ft	(1) Laboratory Classification	Mechanical Analysis (2) Gvl %	Sand %	Fines %	D_{10} mm	Atterberg Limits LL	PL	Sp Gr	Nat w %	Nat Dry Density lb/cu ft	Compaction Opt w %	Max Dry Density lb/cu ft	Shear Data Initial e	Dry Density lb/cu ft	w_i %	w_f %	S_i %	Type Test	Specimen Size in.	No.	σ_{III} tons/sq ft	σ_I tons/sq ft	c tons/sq ft	ϕ deg	Permeability e	k cm/sec × 10^{-4}	Consolidation Data P_o tons/sq ft	P_c tons/sq ft	C_v(3) sq ft/min × 10^{-4}	r(4)	Remarks
FTP-1	UB-1	3.0	Sandy clay CL	-	31	69		38	13	2.75	21.7	98.6						22.0		DS-CD					0.17	28.8	0.73	0.56			7.0	0.38	Alluvial
																												0.29*					
	UC-1	2.8									22.3	96.6																					
	UB-2	8.0	Sandy clay OL	-	44	56		38	11	2.77	26.5	92.2						26.8		DS-CD					0.30	28.2	0.87	0.20			4.5	0.32	Lean, organic
																												0.02*					
	UC-2	2.8									22.2	97.4																					
	UB-3	14.5	Sandy clay OL	-	39	61		32	6	2.74	28.4	94.6						25.1		DS-CD					0.15	28.8	0.79	0.04			8.4	0.42	Lean, organic
																												0.05*					
	B-4	5.5						38	13	2.74																							
	UC-5	7.8									27.0	93.6																					
	UC-6	7.8									26.6	94.7																					
	B-8	14.5						28	2	2.71																							
	UC-9	14.5									27.6	95.8																					
	UC-10	14.5									32.4	90.8																					
FTP-2	UB-1	5.5	Sandy clay CL	-	34	66		31	11	2.74	19.9	100.1						21.0		DS-CD					0.10	31.0	0.70	0.06			5.9	0.38	Tr. mica
																												0.05*					
	UC-1	5.5									21.3	98.5																					
	UC-2	5.5									20.6	102.3																					
FTP-3	UB-4	5.5	Sandy clay ML	15	32	53		42	13	2.68	19.5	99.3																					
	UC-1	5.5									29.3	92.0																					
	UC-2	5.5									24.0	97.0																					
FTP-4	UB-4	5.5	Clay MH	-	14	86		54	17	2.74	24.3	98.2						26.6		DS-CD					0.57	22.8	0.73	0.04			10.9	0.32	Micaceous
																												0.002*					
	UC-1	5.5									22.4	95.1																					
	UB-8	10.5	Sandy clay ML	-	32	68		45	16	2.72	28.9	86.5						29.3		DS-CD					0.30	25.4	0.96	0.03			6.1	0.30	
																												0.01					
	UC-2	5.5									26.0	95.6																					
	B-3	5.5						66	37	2.74																							
	UC-5	10.5									30.8	84.6																					
	UC-6	10.5									30.2	85.1																					
	B-7							44	17	2.68																							
FTP-5	UB-4	5.5	Silty sand SM	-	79	21		NP		2.71	16.1	93.0						18.1		DS-CD					0.00	31.0	0.80	1.53			9.1	0.37	
																												2.40*					
	UC-1	5.5						NP			15.2	91.6																					
	UC-2	5.5						NP			14.0	90.9																					
	B-3	5.5						NP		2.65																							
BTP-1	B-3	1-5	Clay MH	-	5	95		66	25	2.79			31.2	86.0		78.6	41.4	36.2		DS-CD					0.25	24.2	1.34	0.0002					Std. effort
													21.6	98.0		94.0	28.5	31.2		DS-CD					0.50	24.2	0.85	0.00004					Mod. effort
	UC-1	2.8									37.0	77.4																					
	UC-2	2.8									38.4	79.6																					
	B-3	2.8								2.46																							
	B-4	5.5	Clay MH	-	7	93		53	16	2.80			25.5	86.0		78.1	40.4	35.8		DS-CD					0.20	28.3	1.24	0.02					Std. effort
													29.7	97.2		92.4	28.5	30.0		DS-CD					0.28	28.8	.84	0.003					Mod. effort
	UC-4	5.4									34.6	79.4																					
	UC-5	5.4									36.2	77.4																					

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OCTOBER 1966

DS—Direct Shear
CD—Consolidated Drained

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Table 5 (Continued)

Boring	Sample	Sample Depth FT	(1) Laboratory Classification	Mechanical Analysis (2) Gvl %	Sand %	Fines %	D_{10} MM	Atterberg Limits LL	PL	Sp Gr	Nat w %	Nat Dry Density LB/CU FT	Compaction Opt w %	Max Dry Density LB/CU FT	Shear Data Initial e	Dry Density LB/CU FT	w_i %	w_f %	S_i %	Type Test	Specimen Size In.	No.	σ_{iii} Tons/Sq Ft	σ_1 Tons/Sq Ft	c Tons/Sq Ft	ϕ Deg	Permeability e	k CM/SEC X 10^{-4}	Consolidation Data P_o Tons/Sq Ft	P_c Tons/Sq Ft	C_v(3) SQ FT/MIN X 10^{-4}	r(4)	Remarks
BTP-1	B-6	5.4						56	18	2.65																							
	B-5	1-6	Clay MH	-	9	91		55	16	2.81			26.7	87.6		80.4	38.6	36.4		DS-CD					0.24	27.4	1.22	0.003					Std. effort
													17.8	98.4		93.5	27.1	32.0		DS-CD					0.38	27.7	0.84	0.0007					Mod. effort
	UC-7	10.5									29.2	92.4																					
	UC-8	10.5									36.4	85.6																					
	UC-21	15.5									18.1	114.6																					
	UC-22	15.5									21.3	109.6																					
	B-23	15.5						39	8	2.81																							
	B-6-23	-						46	13	2.73																							
	UC-25	20.5									23.8	102.5																					
	UC-26	20.5									21.4	108.2																					
BTP-2	B-4&5	3.5	Clay MH	-	6	94		50	18	2.76			24.0	96.0		87.0	32.1	27.7		DS-CD					0.15	27.7	1.06	0.0002					Std. effort
													16.6	107.0		100.4	24.4	24.1		DS-CD					0.30	26.8	0.69	0.00004					Mod. effort
	UC-1	4.5									23.0	92.9																					
	UC-2	4.5									26.6	89.1																					
	B-3	4.5						47	15	2.62																							
	B-11&12	10.5	Silt ML	-	4	96		NP		2.74			24.0	90.2													1.07	0.097					Std. effort
													19.5	98.7													0.87	0.056					Mod. effort
	UC-9	10.5									18.9	83.8																					
	UC-10	10.5									17.0	81.4																					
	B-15&16	15.5	Silt ML	-	2	98		NP		2.75			22.5	91.2		82.4	33.0	32.7							0.23	29.5	1.09	0.085					Std. effort
													18.5	100.0													0.83	0.40					Mod. effort
	UC-13	15.5									20.5	81.9																					
	UC-14	15.5									20.5	81.7																					
	B-18&19	19.5	Silt ML	-	2	98		NP		2.75			23.0	90.3													1.12	0.10					Std. effort
													18.5	98.0		93.9	24.9	29.1							0.12	31.2	0.85	0.045					Mod. effort
	UC-17										31.0	76.3																					
	UC-18										33.2	73.4																					
	B-19							42	12	2.65																							
	B-3&19							44	11	2.70																							
X(5)	1		Silt ML	1	15	84		NP		2.72			23.2	87.8		86.7	28.9	31.4		DS-CD					0.15	28.8							From serricite
	2		Clay MH	2	13	85		45	12	2.74			23.0	94.7		92.3	27.1	27.3		DS-CD					0.38	26.0							25% No 1 & CL
	3		Clay MH	-	10	90		58	15	2.79			25.6	93.9		90.2	30.5	29.8		DS-CD					0.48	21.1							Red & gray cl
	4		Clay MH	-	13	87		57	9	2.93			31.2	86.0		84.6	35.5	35.2		DS-CD					0.73	24.9							Red cl

NOTES: (1) When not shown, laboratory classification is not available.
(2) Soils classified according to Unified Soil Classification System of Corps of Engineers in use at time tests were made. Sand size range was 2.0-0.074 mm; fines include silt and clay sizes, <0.074 mm.
(3) Coefficient of consolidation at 1 T/sq ft.
(4) Primary compression ratio at 1 T/sq ft.
(5) Materials exposed in borrow area during construction.
* Vertical permeability.

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OCTOBER 1966

DS—Direct Shear
CD—Consolidated Drained

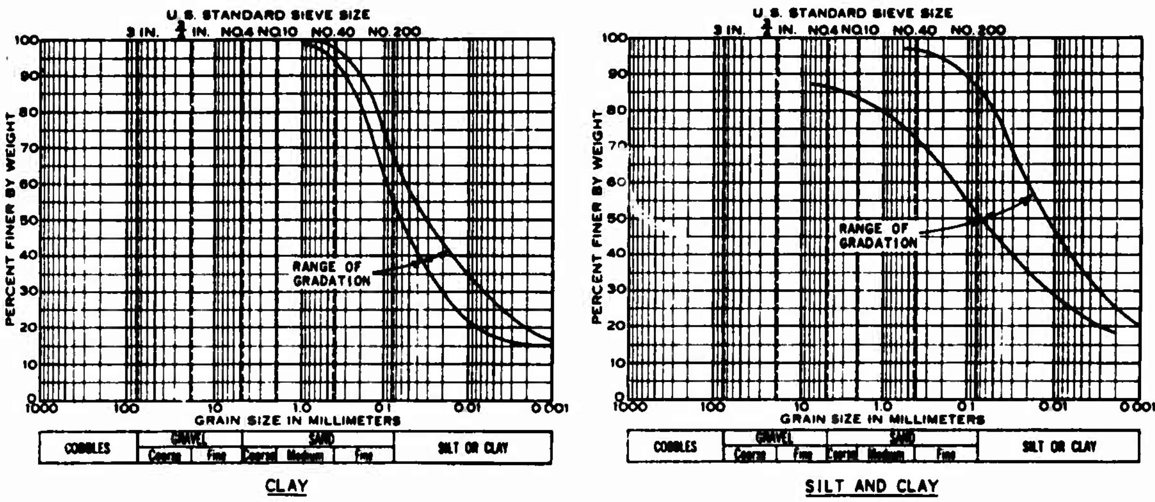


Fig. 21. Gradation of foundation soils, Island Creek Dam

types found within the area, clay, and silt and clay, are shown in fig. 21.

55. Seven consolidated-drained direct shear tests were performed on saturated undisturbed samples from the foundation. Specimens were sheared at a rate of 0.001 in. per min. Shear strength curves obtained from the laboratory tests are shown in fig. 22; the design strength curve, c = 0.05 ton per sq ft, $\phi = 27^{\circ}$, was selected so that it generally would be less than that obtained from each test on foundation soil.

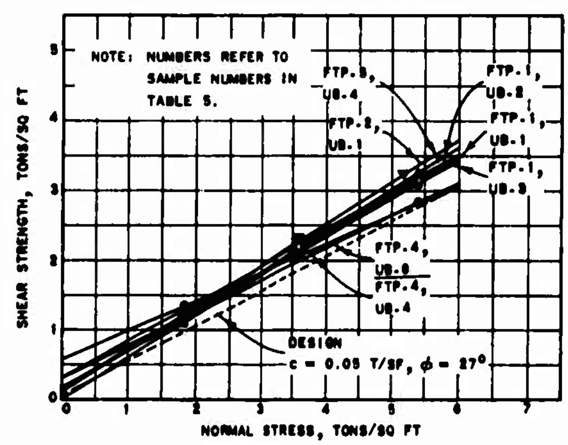


Fig. 22. Consolidated-drained direct shear strength curves and design shear strength curve for undisturbed foundation soils, Island Creek Dam

56. Fourteen permeability tests were performed to determine the coefficients of permeability in both horizontal and vertical directions. The coefficients of permeability in the horizontal and vertical directions were not significantly different. Coefficients of horizontal permeability at a normal load of 0.25 ton per sq ft varied from 1.53 x 10^{-4} cm per sec to 0.03 x 10^{-4} cm per sec. The larger value is for an alluvial sand which was essentially removed during construction. The coefficient of permeability for the residual soils found along the dam

axis varied from 0.56×10^{-4} cm per sec to 0.04×10^{-4} cm per sec, with the average value being 0.21×10^{-4} cm per sec.

57. Laboratory consolidation tests were performed on seven undisturbed samples from the foundation. Pressures up to 8 tons per sq ft were used in the testing, and plots of time vs rate of consolidation were made for the test specimens. Typical consolidation curves (e vs log p) are shown in fig. 23.

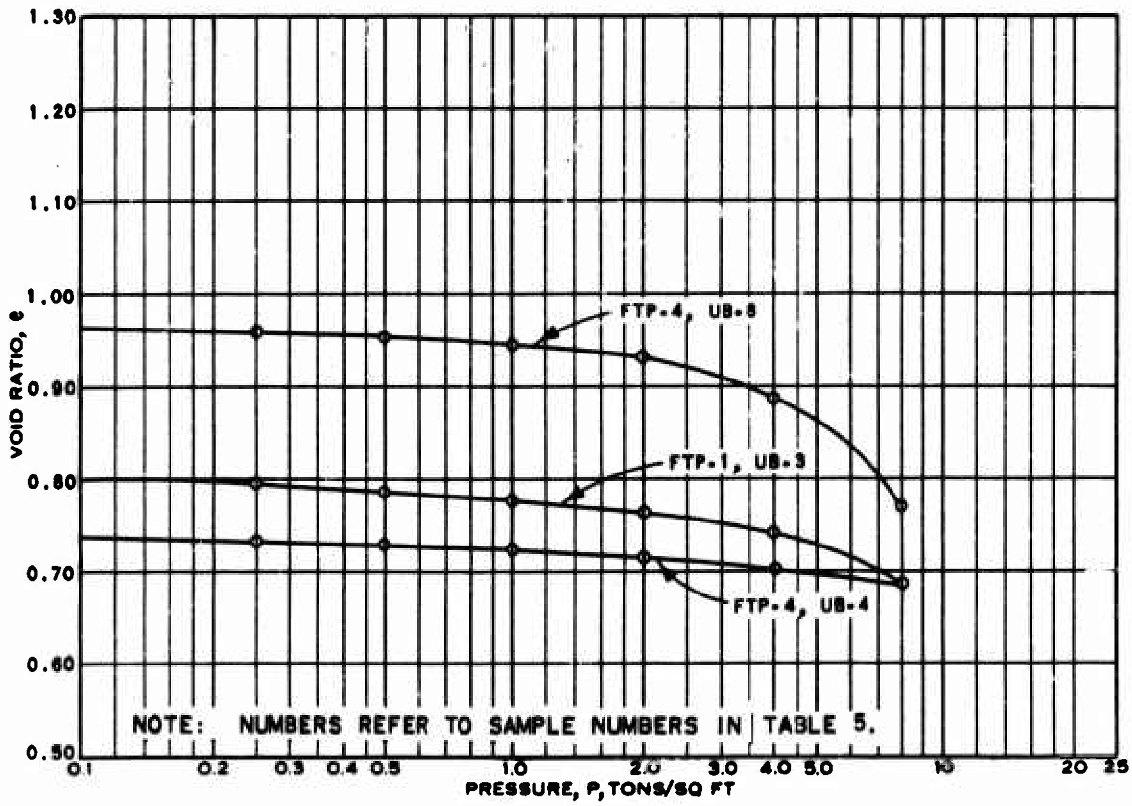


Fig. 23. Typical void ratio-pressure curves for undisturbed foundation soils, Island Creek Dam

58. <u>Seepage and settlement analyses.</u> Where overburden soils were less than 15 ft thick it was considered necessary to grout the weathered rock to limit seepage through this zone of the foundation. The trench which was excavated to the top of weathered rock in preparation for grouting of the foundation was later backfilled with impervious material. In areas where overburden soils were greater than 15 ft thick it was believed that the overburden was sufficiently thick to effectively limit

seepage entering the foundation. Computations to estimate the quantity of underseepage with and without the grout curtain were not made.

59. Computations of settlement in the foundation were not made, but a comparison with the materials, characteristics, and analyses used for a nearby railroad relocation indicated that the foundation settlement would be insignificant and would be complete at the end of construction. Any postconstruction settlement could be corrected by adding additional material for the road to be built on top of the dam.

Embankment

60. Borrow materials. Embankment materials were to be obtained from three borrow areas in the vicinity of the dam site. The borrow areas, two on the left bank and one on the right bank of Island Creek, were within one mile (downstream) of the dam site. Borrow area investigations included 85 auger borings and 2 test pits. The soils in all borrow areas were found to be similar, ranging from an upper zone of micaceous clay approximately 4-6 ft deep to a lower zone of micaceous silt extending to weathered rock at a variable depth of approximately 25 ft. The materials for slope protection were not available at the dam site and were brought from a quarry at Buggs Island Dam site.

61. Zonation. Design zonation of the embankment consisted of a central impervious core section flanked by more pervious shell sections on the landside and reservoir side. Laboratory studies indicated that if the core and shell were built with the same material the permeability of the shell section would be about 10 times greater than that of the core section if the shell were compacted to 90 per cent of standard density and the core were compacted to 95 per cent modified density. The studies also indicated that the same permeability ratio could be obtained by placement of the more impervious borrow material in the core section and the more pervious material in the shell section, both to the same relative density. Since the relative amount of more impervious material was not known, increased compaction effort for the core material was selected as the method to be used to obtain the desired permeability ratio.

62. Laboratory tests. Laboratory testing of borrow materials consisted of classification and compaction tests, and shear and permeability

tests on material compacted at various efforts in order to determine the feasibility of achieving zonation in the manner previously described. The results of the laboratory tests are presented in table 5. The ranges of gradations of the materials used in the embankment are shown in fig. 24. A predominance of fine-textured soils constituted the available borrow materials.

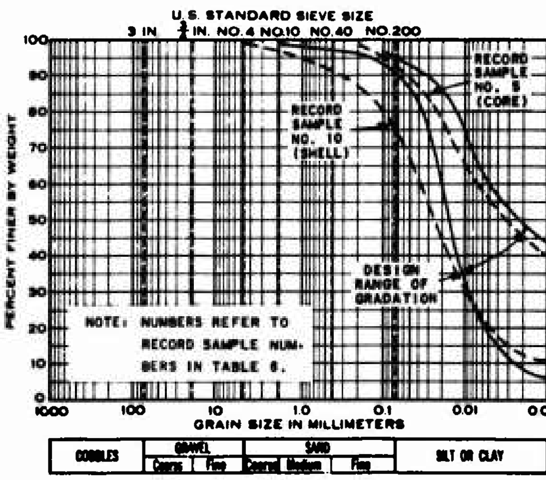


Fig. 24. Gradation of embankment borrow and record sample soils, Island Creek Dam

63. Eighteen standard effort and modified AASHO compaction tests were performed on borrow materials to determine their compaction characteristics for use in obtaining the desired permeability ratio between shell and core sections. Typical compaction curves for representative materials are shown in fig. 25. The modified AASHO tests indicated maximum dry densities 8-12 lb per cu ft higher and optimum moisture contents 4-10 per cent lower than standard effort values.

64. Permeability tests were performed on 14 samples of borrow material compacted at varying densities. These tests formed the basis for the recommended field compaction control to provide the desired zonation. Results of the permeability tests are presented in table 5. Tests on four samples of relatively impervious material

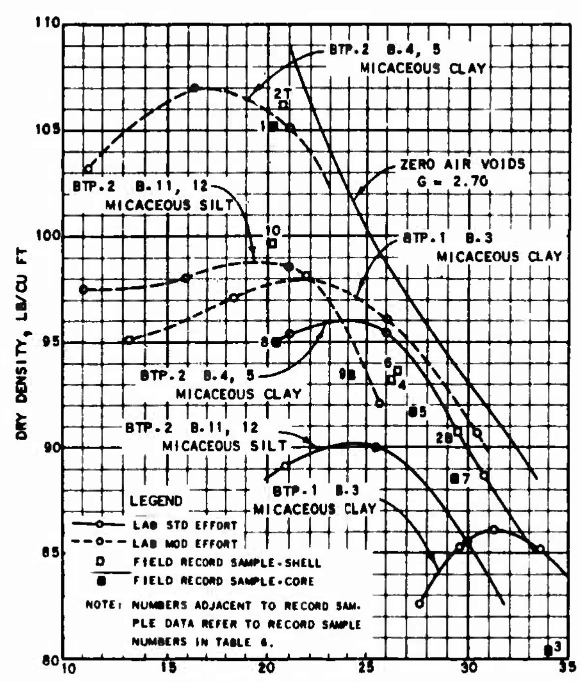
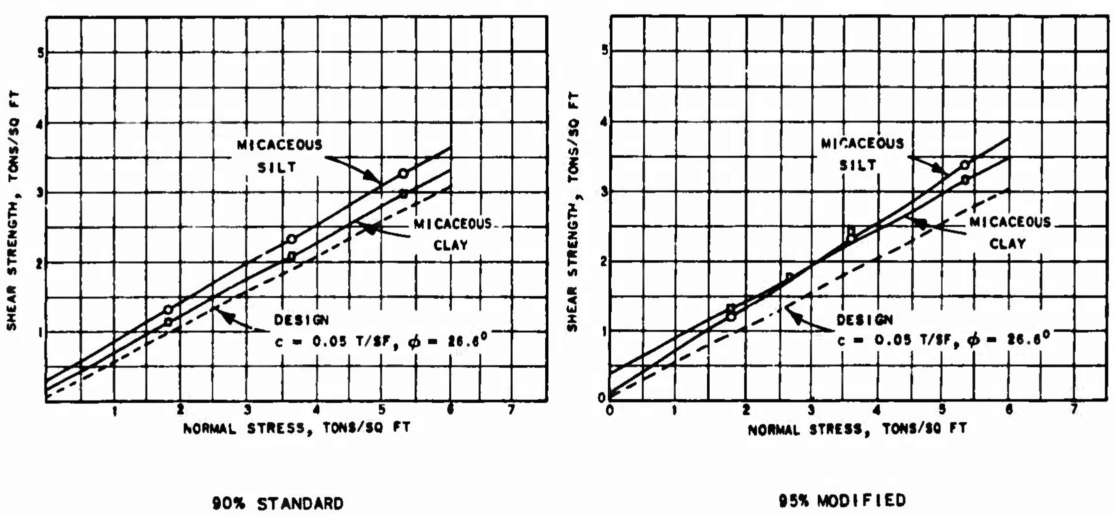


Fig. 25. Typical laboratory compaction curves for borrow materials, and record sample data, Island Creek Dam

compacted at 90 per cent of standard density indicated an average $k = 0.006 \times 10^{-4}$ cm per sec, while tests on the same material compacted at 95 per cent of modified AASHO density indicated $k = 0.0009 \times 10^{-4}$ cm per sec.

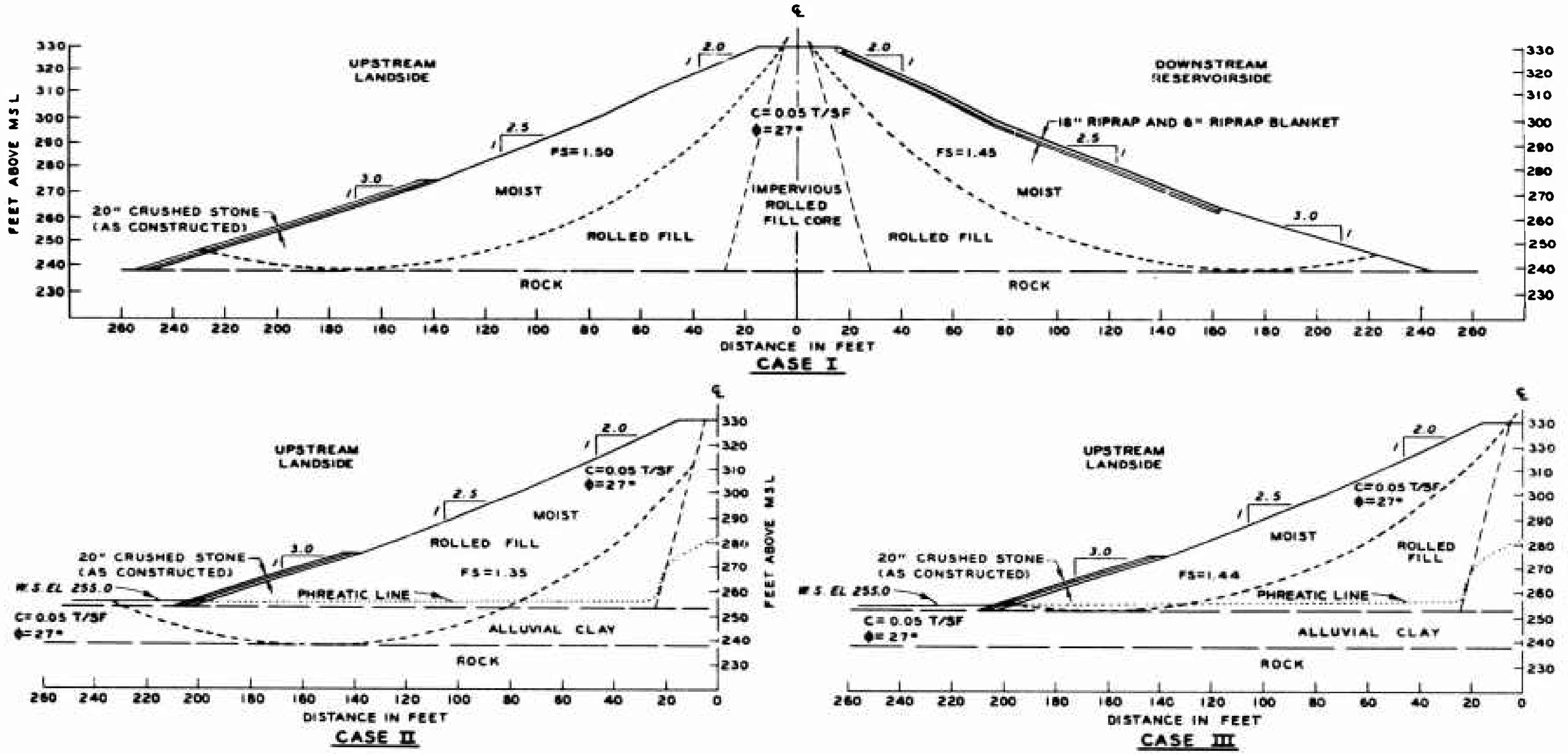
65. Ten consolidated-drained direct shear tests were performed on saturated borrow materials compacted to various densities. The shear strength curves resulting from tests on representative borrow materials compacted to 90 per cent of standard density and 95 per cent of modified AASHO density are shown in fig. 26; the embankment design strength curve is also shown in this figure. The design strength, c = 0.05 ton per sq ft, $\phi = 27^{\circ}$, was selected so that it generally would be less than that obtained from each test on compacted borrow soil.



MATERIAL	COMPACTION	SHEAR STRENGTH c, T/SF	SHEAR STRENGTH ϕ°	AVERAGE γ_d	AVERAGE w_f
MICACEOUS SILT	90% STD	0.23	29.5	82.4	32.7
MICACEOUS SILT	95% MOD	0.12	31.2	93.8	29.1
MICACEOUS CLAY	90% STD	0.15	27.7	87.0	27.7
MICACEOUS CLAY	95% MOD	0.38	27.7	93.5	31.8
DESIGN	—	0.05	26.6	—	—

Fig. 26. Typical consolidated-drained direct shear strength curves and design shear strength curve for compacted borrow material, Island Creek Dam

66. <u>Stability, settlement, and seepage analyses.</u> The circular arc method was used in making stability analyses of the embankment. Conditions analyzed and soil properties used in the analyses as well as the results are shown in fig. 27. Analyses were made for the maximum embankment



CONDITIONS ANALYZED

CASE	EMBANKMENT CONDITIONS	FACTOR OF SAFETY
I	END OF CONSTRUCTION; RESERVOIR EMPTY,	
	UPSTREAM	1.50
	DOWNSTREAM	1.45
II	UPSTREAM WATER AT EL 255; RESERVOIR STAGE EL 300, MOIST ABOVE PHREATIC LINE; SATURATED-SUBMERGED BELOW PHREATIC LINE; FAILURE ARC TANGENT TO ROCK SURFACE	1.35
III	HYDRAULIC CONDITIONS AS IN II; FAILURE ARC TANGENT TO TOP OF FOUNDATION CLAY.	1.44

SOIL PROPERTIES

MATERIAL	MOIST DENSITY LB/CU FT	SATURATED DENSITY LB/CU FT	SHEAR STRENGTH c_s, T/SF	SHEAR STRENGTH ϕ^o
ROLLED FILL	122	130	27	0.05
FOUNDAT ON	108	122	27	0.05
RIPRAP , D BEDDING	110	110	–	–

Fig. 27. Stability analysis for embankment, Island Creek Dam

section for the condition at the end of construction (pore pressure assumed zero) and an operating condition with the water surface at el 255 on the upstream landside slope and at el 300 on the downstream reservoir slope. The condition for rapid drawdown was not analyzed. Factors of safety for the conditions analyzed ranged from 1.50 to 1.35.

67. No calculations were made of the amount and rate of settlement within the embankment. A comparison with analyses for a nearby railroad relocation embankment with the same characteristics as the dam indicated that approximately 1 ft of settlement would occur within the embankment after construction. The amount of total settlement was not considered critical, since final grading and shaping of the dam crest was accomplished at the time of relocation of a highway on top of the dam.

68. The maximum anticipated differential head on the dam was 45 ft. No seepage analyses were made and no facilities for drainage were provided in the structure. The quantity of seepage expected through the embankment was not considered significant because the core was relatively impervious, foundation soils were relatively impervious, and a grout curtain was used to limit seepage where the foundation soils were shallow.

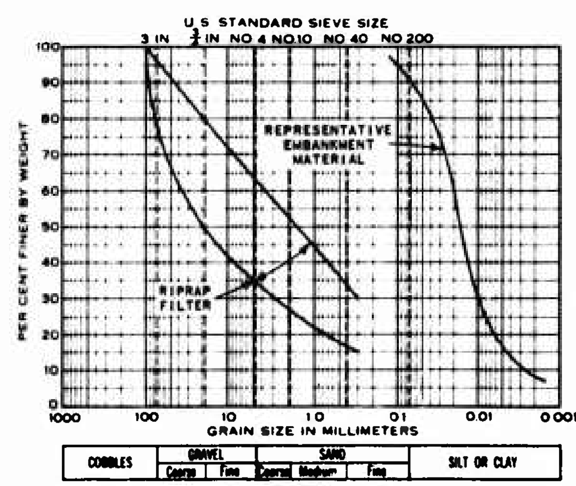


Fig. 28. Gradation of riprap filter, Island Creek Dam

69. In the design, the upstream landside slope of the dam was to be protected by 12 in. of crushed stone from the base to el 275; above el 275 the slope was to be mulched and seeded. The reservoir side slope was to be protected by 18 in. of riprap (well graded from a minimum size of 25 lb to a maximum size of 400 lb) over a 6-in. crushed stone filter between el 265 and the top of the dam. No protection was to be provided below el 265. The proposed gradation range of the filter material is shown in fig. 28.

70. The report, Investigation of Filter Requirements for Underdrains,* was used to determine the gradation of the crushed stone filter. However, a criterion factor of 5 was used instead of 4 to establish the "15 per cent passing" limits. It was felt that crusher screenings blended with commercial rock could be used for riprap bedding and upstream slope protection.

Outlet works

71. As the foundation for the outlet works consists of rock, it is not discussed in this report.

Construction

Borrow materials

72. Soils for the embankment were obtained from three borrow areas, two located on the left bank about 3/4 mile downstream, and the third on the right bank about 1/2 mile downstream from the dam. Soils in the borrow areas were residual deposits ranging from micaceous clay in the upper zone to micaceous silt in the lower zone. Significant quantities of clay in the upper zone of the borrow areas were found and this material was reserved for the impervious central core. Less impervious soils, consisting mostly of silts and a combination of clays and silts, were used for the upstream and downstream shell sections. Drainage in the borrow areas was provided by selective excavation.

73. While excavating in the right bank borrow area, zones of white micaceous silt, derived from sericite, and red and gray clay were disclosed. These materials had not been located during earlier explorations and their suitability for use as embankment material was uncertain. Samples of these materials were tested in the laboratory (for results, see table 5) and it was determined that they would be suitable for the core section if blended in the proportion of 25 per cent silt and 75 per cent clay.

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* Corps of Engineers, Waterways Experiment Station, TM No. 183-1, Vicksburg, Miss., December 1941.

74. Riprap was obtained from the Buggs Island Dam quarry. Although gradation tolerances for the riprap were reasonable, a number of loads were rejected because of excessive fines or oversize stones.

Embankment

75. Foundation preparation. The foundation for the embankment was cleared and stripped of all undesirable overburden soils. The creek area was cleaned to the rock surface and a single-line grout curtain placed along the dam center line from sta 95+00 to 99+05. Type II portland-cement grout was used at pressures of 20 psi for depths to 20 ft and 40 psi for depths to 50 ft. Grout holes were spaced on 5-ft centers; depths of alternate holes were 20 ft and 50-75 ft. Approximately 2982 linear ft of EX grout holes (1-1/2-in. diameter) were drilled and 1028 bags of cement were used.

76. Deposits of saturated alluvial sand were found along both banks during removal of material from the creek area. This material was removed to eliminate a possible underseepage path. To remove the sand from the right bank, a trench approximately 12 ft wide was cut into the bank a distance of 40 ft along the dam center line.

77. Stream diversion. As soon as clearing operations permitted, cofferdams were constructed both upstream and downstream, and the creek was diverted through a channel at sta 102+00 and into the creek bed 175 ft below the downstream cofferdam. The concrete conduit was then constructed at sta 96+62. In the second stage of stream diversion, a cofferdam was used to cut off the diversion channel; water was passed through the upstream cofferdam and into the conduit by means of a 48-in.-diam steel pipe (see fig. 3). The water was carried by a ditch from the conduit outlet to the creek.

78. Earthwork. Embankment construction was begun between sta 102+00 and 107+00 in October 1950. In November 1950, subsequent to the completion of foundation grouting, embankment construction was started in the creek bed area and the area between the creek bed and diversion channel. By the end of the first construction season (November) the embankment to the right of the diversion channel (sta 102+00 to sta 107+00) had reached el 280, and the upstream and downstream portions of

the embankment to the left of the diversion channel had reached el 248 and el 241, respectively. The next spring, the diversion channel at sta 102+00 was filled and the conduit covered (top el 275), thereby permitting placement of soil the full width of the dam.

79. Although it had been recommended in design that the zonation of the embankment be obtained by extra compaction of the core section, adequate quantities of clay for a central impervious core were found in the upper zones of the borrow areas, and during construction it was decided that the zonation of the embankment would be obtained by placement of these clays in the central core. Both shells and core were compacted with a minimum of 6 passes of 500-psi rollers. Additional passes were made, as necessary, on the central core to obtain a relatively more impervious central section. All soils were compacted to a minimum of 95 per cent standard density. Moisture content, which was selected as the basic control, was specified to be between 2 per cent dry to 4 per cent wet of optimum.

80. Soils were placed in 8-in. loose lifts, bladed smooth, and then compacted with sheepsfoot rollers providing a contact foot pressure of 500 psi on L-shaped feet having a cross-sectional area of 7 sq in. Soils placed within 4 ft of the conduit and concrete walls were spread in 4-in. layers and hand-tamped. Lumpy and wet materials were disked before rolling. When too dry, the lift was sprinkled before rolling. At the end of each work day and in anticipation of rain, the embankment surface was bladed to facilitate drainage.

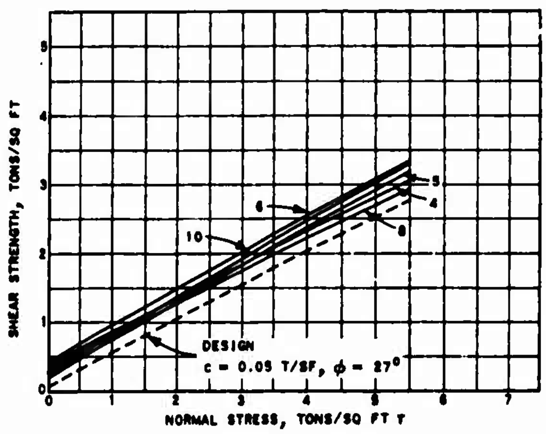
81. A total of 239 field density tests were made for compaction control; this was an average of one test for about 2400 yd of compacted material. Field control methods were similar to those for Buggs Island Dam (see paragraph 35). Results of the field tests indicated that water contents for 76 per cent of the specimens were within the specified 2 per cent dry to 4 per cent wet of optimum, and densities for 97 per cent of the specimens were at least 95 per cent of standard density. Density tests indicated that little additional rolling was required to obtain adequate densities in the core, and although provisions for 400 hr of additional rolling were included in the contract, only 80 hr were required.

82. Record samples. Ten undisturbed box samples were taken from the embankment for record purposes. Three of the samples contained rock fragments or were otherwise disturbed to such an extent that preparation of undisturbed specimens for laboratory testing was not possible. A summary of record sample test results is presented in table 6. Mechanical analyses were performed on all record sample materials. A comparison of typical record sample gradations with the ranges of borrow material gradation is shown in fig. 24. The as-placed shell materials are slightly coarser than the design gradations of borrow materials.

83. Densities of the record samples are shown in table 6; a comparison of these densities with the ranges of laboratory compaction curves is shown in fig. 25.

84. Consolidated-drained direct shear tests were performed on saturated undisturbed specimens from five record samples, and similar shear tests were performed on remolded specimens from two other record samples. Shear strengths of undisturbed record samples varied from c = 0.20 ton per sq ft to 0.40 ton per sq ft, $\phi = 25^{\circ}$ to 29°; the design shear strength was c = 0.05 ton per sq ft, $\phi = 27^{\circ}$. A comparison of record sample shear strength curves with the design strength curve is shown in fig. 29. It can be seen that strengths of record samples were somewhat greater than the design strength.

85. The coefficient of horizontal permeability was determined for undisturbed specimens from four record samples taken in the core and from two record samples taken in shell materials. The permeability of the samples from the core ranged from 0.002×10^{-4} cm per sec to 0.200×10^{-4} cm per sec with an average value of 0.056×10^{-4} cm



SAMPLE NO.	SHEAR STRENGTH c, T/SF	SHEAR STRENGTH ϕ°	AVERAGE γ_D	AVERAGE w_f
4	0.35	26.2	93.2	28.1
5	0.20	28.8	91.8	31.8
6	0.40	26.2	93.7	31.7
8	0.32	25.4	95.0	25.8
10	0.25	29.0	90.7	22.8
DESIGN	0.05	26.6	—	—

Fig. 29. Record sample shear strength curves, Island Creek Dam

Table 6

Summary of Test Data, Record Samples, Island Creek Dam

BORING	SAMPLE	SAMPLE EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS (1) GVL %	SAND %	FINES %	D_{10} MM	ATTERBERG LIMITS LL	PI	SP GR	NAT w %	NAT DRY DENSITY LB/CU FT	COMPACTION OPT w %	MAX DRY DENSITY LB/CU FT	SHEAR DATA INITIAL e	DRY DENSITY LB/CU FT	w_i %	w_f %	S_i %	TYPE TEST	SPECIMEN SIZE IN.	NO.	σ_{iii} TONS/SQ FT	σ_i TONS/SQ FT	c TONS/SQ FT	ϕ DEG	PERMEABILITY e	k_H CM/SEC X 10^{-4}	CONSOLIDATION DATA P_o TONS/SQ FT	P_c TONS/SQ FT	c_v(2) SQ FT/MIN X 10^{-4}	r(3)	LOCATION
	1	272.3	ML	7	10	83		47	17	2.80	20.8	105.3			0.65*	100.2	23.3	24.1		DS-CD					0.37	28.7	0.65	0.002			18.7	0.30	98+00 - ℄
	2T	270.5	ML	23	15	62		42	11	-	20.8	106.1		Rock fragments; sample broken on receipt in laboratory																			98+00 - 85 US
	2B	271.5	MH	10	10	80		52	12	-	29.5	90.9		Rock fragments; sample broken on receipt in laboratory																			98+00 - 85 US
	3	280.5	MH	0	23	77		56	8	2.80	34.0	80.1			1.13*	79.8	33.3	36.8		DS-CD					0.12	25.4	1.13	0.20			8.9	0.40	104+00 - ℄
	4	280.0	MH	0	15	85		48	9	2.75	26.1	93.2			0.82			28.1		DS-CD					0.35	26.2	0.82	0.040			13.8	0.44	104+00 - 80 US
	5	285.0	MH	2	11	87		59	23	2.77	27.4	91.8			0.89			31.8		DS-CD					0.20	28.8	0.89	0.018			10.5	0.50	98+00 - ℄
	6	285.0	MH	15	10	75		54	14	2.77	26.5	92.[illegible]			0.86			31.7		DS-CD					0.40	28.2	-	-			8.8	0.41	98+00 - 80 DS
	7	301.0	MH	3	22	75		51	14	2.71	29.3	88.7			0.84		Very hard rock fragments										0.84	0.003			12.5	0.43	104+00 - ℄
	8	300.0	ML	2	25	73		39	13	2.71	20.6	95.0			0.82			25.9		DS-CD					0.33	25.4	0.82	0.030			16.2	0.52	104+00 - 40 DS
	9	301.0	ML	4	22	74		49	11	-	24.2	93.6			-	Unblended heterogeneous material in sample											-	-					98+00 - ℄
	10	301.0	ML	2	22	76		36	9	2.70	20.1	99.7			0.70			22.9		DS-CD					0.25	29.0	-	-			8.3	0.45	98+00 - 40 DS

NOTES: (1) Soils were classified according to Unified Soil Classification System of Corps of Engineers in use at time tests were made. Sand size range was 2.0 to 0.074 mm; fines include silt and clay sizes, <0.074 mm.
(2) C_v = Coefficient of consolidation at 1 T/sq ft.
(3) r = Primary compression ratio at 1 T/sq ft.
* Shear test specimens remolded; test performed on material finer than No. 4 sieve.

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DS—Direct Shear
CD—Consolidated Drained

per sec. The average permeability of the samples from the shell was 0.035×10^{-4} cm per sec. The apparent lack of attainment of the desired permeability ratio between shell and core might be the result of an inadequate number of tests, excess compaction of the shell, inadequate compaction of the core, or ineffective selection of materials for placement in the shell and core.

86. Slope protection. Riprap placement on the reservoir slope, accomplished under a separate contract, was begun when the embankment reached el 280. Filter materials were placed and hand-raked to a 6-in. thickness. Riprap was dumped into a skip pan which was lowered into position and dumped by a crane operating on top of the embankment. Final positioning of the riprap was done by hand. Riprap was placed from el 265 to el 330. No record samples of riprap and filter material were obtained.

87. On the upstream or landside slope a 12-in. layer of crushed stone (4 in. to sizes passing No. 50 screen) was placed from the base to el 275. Heavy rains in the summer of 1951 eroded the protective stone layer in some places. It is believed that the washouts were caused by excessive fines in the blanket. The breached areas were repaired with crushed stone of maximum 6-in. size. Finally an 8-in. layer of 1-1/2- to 6-in. stone was placed over the entire protected area to prevent recurrence of the erosion.

Prototype Observations

88. With the exception of the erosion of the downstream slope protection during the summer of 1951 while construction was still in progress, the embankment is apparently performing satisfactorily. Embankment shear strengths appear to be generally as great or greater than the design strengths. Provisions for observation of settlement and seepage were not made.