REVIEW OF SOILS DESIGN CONSTRUCTION, AND PERFORMANCE OBSERVATIONS, LOOKOUT POINT DAM OREGON

TECHNICAL REPORT NO. 3-502

May 1959

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

ASSOCIATED REPORTS

Report Designation	Date	Title
TM 3-409	Oct 1955	Review of Soils Design and Field Observations, Enid Dam Yocona River, Mississippi
TR 3-439	Oct 1956	Review of Soils Design, Construction, and Prototype Analysis, Blakely Mountain Dam, Arkansas
TR 3-452	Mar 1957	Review of Soils Design, Construction, and Performance Observations, Benbrook Dam, Texas
TR 3-464	Aug 1957	Review of Soils Design, Construction, and Performance Observations, John H. Kerr Project, Virginia
TR 3-474	Mar 1958	Review of Soils Design, Construction, and Performance Observations, Tom Jenkins Dam, Ohio
TR 3-484	Sept 1958	Review of Soils Design, Construction, and Prototype Observations, Texakana Dam, Tex.
TR 3-501	April 1959	Review of Soils Design, Construction, and Performance Observations, Harlan County Dam, Nebraska

U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss. REVIEW OF SOILS DESIGN, CONSTRUCTION, AND PERFORMANCE OBSERVATIONS, LOOKOUT POINT DAM, OREGON, by R. C. Sloan. May 1959, 44 pp - illus. (Technical Report No. 3-502)

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ARMY-MRC VICKSBURG, MISS.

PREFACE

This report is one of a number of similar reports on the construction and behavior, from a foundation and soil mechanics standpoint, of earth dams completed by the Corps of Engineers since 1950. These reports are prepared by the U. S. Army Engineer Waterways Experiment Station as part of the Civil Works Investigations program of the Office, Chief of Engineers, under CWI Item 505, "Prototype Analysis (Soil)." The purpose of these reports is to compare the performance of the completed structure with design predictions, and from these comparisons gain information that may be valuable in the design and construction of future projects by the Corps of Engineers.

Lookout Point Dam was designed by and built under the supervision of the Portland District, Corps of Engineers, and data on the design, construction, and performance observations were furnished by that district. No evaluation or analysis of data has been made by the Waterways Experiment Station. Design details not included in this report can be found in Analysis of Design for Lookout Point Dam (Meridian Site), Middle Fork of Willamette River (January 1950), Supplementary Analysis of Design of Left Abutment for Lookout Point Dam (Meridian Site), Middle Fork Willamette River (May 1951), and Construction of Test Fills, Heavy Pneumatic-tired Roller, Lookout Point Dam (April 1950), all of which were published by the Portland District.

This report was prepared by Mr. R. C. Sloan under the direction of Messrs. W. J. Turnbull, W. G. Shockley, and R. W. Cunny of the Soils Division, Waterways Experiment Station. Prior to publication, this report was reviewed by personnel of the Portland District and North Pacific Division, and was reviewed and approved by the Office, Chief of Engineers.

Directors of the Waterways Experiment Station during the preparation of this report were Col. A. P. Rollins, Jr., CE, and Col. Edmund H. Lang, CE. Mr. J. B. Tiffany was Technical Director.

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LIST OF NOTATIONS

c	Cohesion, tons per sq ft
C_v	Coefficient of consolidation
CD-D	Consolidated-drained direct shear test
CD-T	Consolidated-drained triaxial shear test
e	Void ratio
e_o	Initial void ratio
k	Coefficient of permeability
LL	Atterberg liquid limit
PI	Atterberg plasticity index (LL-PL)
PL	Atterberg plastic limit
s	Shear strength, tons per sq ft
S	Degree of saturation
UU-T	Unconsolidated-undrained triaxial shear test
w	Water content, per cent of dry weight
γ_d	Unit dry weight, lb per cu ft
γ_m	Unit moist weight, lb per cu ft
γ_s	Unit saturated weight, lb per cu ft
σ_1	Major principal stress, tons per sq ft
σ_3	Minor principal stress, tons per sq ft
ϕ	Angle of internal friction, degrees

SUMMARY

Lookout Point Dam, on the Middle Fork of the Willamette River, is a rolled-fill earth core and gravel shell embankment with a concrete gravity spillway and nonoverflow section, and an over-all length of 3381 ft. The embankment has a maximum height of 273 ft and is 2110 ft long along the crest from the left abutment to the point of contact with the concrete section; it contains about 8,000,000 cu yd of fill. The gated spillway has a net opening of 212.5 ft and a maximum discharge capacity of 268,000 cfs. The flood-control outlet works consists of four 12-ft-diam conduits located in the spillway section, with a maximum discharge capacity of 24,000 cfs with the reservoir at minimum flood-control pool. Reservoir elevations at the spillway crest and at maximum pool will create net heads of about 190 and 235 ft, respectively.

Foundation

The bedrock consists of tuff- and andesite-breccia overlying basalt porphyry. The overburden is Recent clay talus as much as 90 ft thick on the left abutment slope, Recent alluvium ranging from 5 to 40 ft thick on the valley floor, and Recent heterogeneous deposits at the toe of each abutment slope and high on the right abutment slope; the steep right abutment slope is exposed bedrock. The selected design shear strengths for unconsolidated clay talus, based on the lowest value of the combined direct and triaxial shear envelopes, were $c = 0.26$ ton per sq ft and $\phi = 12.9°$ for embankment loads of less than 1.15 tons per sq ft, and $c = 0.50$ ton per sq ft and $\phi = 1.7°$ for embankment loads of more than 1.15 tons per sq ft; the selected design shear strength for partially consolidated and consolidated clay talus was $c = 0.26$ ton per sq ft and $\phi = 12.9°$. Analyses of induced slides in an exploration trench prior to construction indicated strengths similar to the selected design strengths. The valley alluvium (gravels) was determined to have a strength of $c = 0.0$ ton per sq ft and $\phi = 40.4°$ by laboratory tests. Seepage through the pervious valley gravels is controlled by an impervious cutoff core, which extends down to bedrock. The foundation beneath the concrete sections is firm rock and consequently is not discussed in this report.

Embankment

The embankment consists of an impervious clay core and relatively pervious random gravel shells. It was designed to have a minimum factor of safety of about 1.3 for the very severe assumption of a sudden reservoir drawdown from maximum to minimum power pool, and about 1.5 for steady seepage with a maximum reservoir pool. The selected design shear strengths were $c = 0.30$ ton per sq ft and $\phi = 24.2^{\circ}$ and $c = 0.0$ ton per sq ft and $\phi = 40.4^{\circ}$ for the impervious core and random shell materials, respectively. The impervious embankment core material was placed in 12-in. loose lifts and compacted with four passes of pneumatic-tired rollers weighing 85,000 lb and having tire pressures of 85 psi; the random gravel shell material was placed in 18-in. loose lifts and compacted with two passes of the same rollers. Field control tests performed on the embankment impervious core materials during construction indicated that an average density, 96% that of standard density, was obtained and that the impervious core was compacted about 1.5% wet of standard optimum water content. Shear test results on record samples of the impervious core compared favorably with the selected design strengths.

The upstream slope is protected by riprap; the downstream random gravel slope has no special protection.

Performance Observations

Piezometer observations indicate that the foundation clay talus is adequately drained for all expected operating conditions. Settlement plate observations indicate that the actual embankment consolidation will be much less than the computed amounts of 11.0 and 26.8 ft at sta 8+00 and 20+00, respectively. The embankment slopes have not required excessive maintenance and none is anticipated.

REVIEW OF SOILS DESIGN, CONSTRUCTION, AND PERFORMANCE OBSERVATIONS, LOOKOUT POINT DAM, OREGON

PART I: INTRODUCTION

Purpose of Study

1. This study is one of a series of similar studies of the foundation and soil mechanics features of earth dams recently constructed by the Corps of Engineers.* In these studies original design data are reviewed, construction experience is summarized, and performance of the completed structures is compared with design predictions to provide information for use in the design and construction of future similar projects.

General Features of the Dam

2. Lookout Point Dam (fig. 1) is located on the Middle Fork of the Willamette River approximately 23 miles southeast of the city of Eugene, Oregon (see vicinity map in plate 1). The dam is one unit of a coordinated system of authorized reservoirs in the Willamette River Basin. The primary benefit to be derived from the dam is flood control; however,

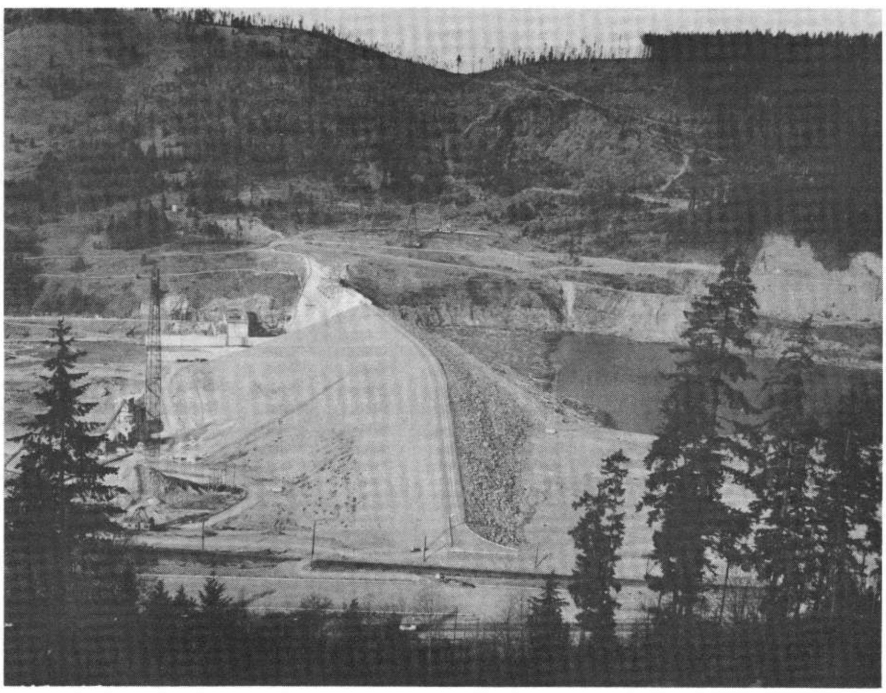

Fig. 1. Dam site

* A list of similar reports is shown on the inside of the front cover of this volume.

it also is operated in the interest of irrigation, navigation, hydroelectric power, and stream-pollution control.

3. The dam is a rolled earth and gravel embankment with a gate-controlled concrete spillway and a concrete nonoverflow section adjacent to the right abutment. A plan of the dam is shown in plate 1. The over-all length is 3381 ft; the earth and gravel embankment is 2110 ft long at crown elevation of 941.0,* and has a maximum height of 273 ft. The embankment crown width is 24 ft and the upstream slopes are 1 on 2 and 1 on 2.5; the downstream slopes are 1 on 1.75, 1 on 2, and 1 on 2.5. The total volume of the embankment is 7,943,000 cu yd. The upstream slope is protected with riprap. The downstream section, which is composed of random gravel, has no special slope protection other than a large rock toe that extends up to el 725.0.

4. The concrete spillway and outlet works are founded on rock, as is the adjacent concrete nonoverflow section that ties into the right abutment. The spillway section contains five 42.5- by 42.0-ft electrically operated tainter gates and four 12-ft-diam flood-control conduits. The tops of the gates in the closed position are at el 929.0, the top of the concrete ogee weir is at el 887.5, and the inverts of the flood-control conduits are at el 724.0. The net opening of the gated section is 212.5 ft, with a maximum discharge of 268,000 cfs at the maximum reservoir pool elevation of 934.0. The maximum discharge capacity of the flood-control conduits is 24,000 cfs with the pool at el 825.0. The concrete nonoverflow section has a crown elevation of 941.0 and contains three 18-ft-diam penstocks for the hydroelectric plant located immediately below the dam. Pertinent dam and reservoir data are summarized in table 1.

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* All elevations are in feet above mean sea level.

Table 1

Pertinent Dam and Reservoir Data

Drainage area, sq mi	991
Minimum power pool el	811.0
Minimum flood-control pool el	825.0
Maximum pool el	934.0
Embankment	
Crown el	941.0
Maximum height, ft	273
Length, ft	2110
Volume, cu yd	7,943,000
Spillway	
Total length, ft	248.5
Net length, ft	212.5
Type gates	Tainter
Number of gates	5
Size of gates, ft	42.5 by 42.0
Crest el of concrete weir	887.5
El of top of gate	929.0
Stilling basin floor el	644.0
End sill el	680.0
Maximum discharge capacity (reservoir pool at el 934.0), cfs	268,000
Outlet conduits	
Number	4
Diameter, ft	12
Type gates	Tainter
Size gates, ft	6.75 by 12
Invert el of conduits	724.0
Maximum discharge capacity (reservoir pool at el 825.0), cfs	24,000
Nonoverflow concrete section	
Length, ft	1022
Number of penstocks	3
Diameter of penstocks, ft	18
Top el	941.0

Construction History

5. Construction of the embankment was accomplished under three major contracts. The first contract extended from June to October 1948 and provided for the excavation of a large exploration trench in the left abutment to assist in the determination of the foundation clay talus characteristics. The second contract extended from June 1949 to May 1950 and provided for the diversion of the river and construction of a small section of the embankment in the valley. The third contract extended from May 1950 until June 1954 and provided for the completion of the remaining larger portion of the embankment and the concrete spillway and nonoverflow section.

6. Water was first impounded in the reservoir in June 1954 and the highest stage to date, el 927.0, occurred in June 1956.

PART II: INVESTIGATIONS AND DESIGN

Foundation

Geology

7. The dam and reservoir are in the Western Cascade Mountains physiographic subdivision of the Cascade Mountains. The rocks found in the vicinity of the dam range in age from Eocene to Miocene, and consist of tilted sediments, pyroclastic beds, and lava flows, for the most part andesitic. The topography is characterized by narrow valleys and sharp ridges that bear little obvious relation to the underlying bedrock structure. The foundation bedrock along the center line of the dam consists of tuff-breccia from the left abutment to sta 13+50, andesite-breccia from sta 13+50 to sta 19+80, tuff-breccia from sta 19+50 to sta 21+90, and andesite-breccia from sta 21+90 to the right abutment. All these are underlain by basalt porphyry. Overburden deposits in the dam area consist of Recent clay talus on the left abutment slope and Recent heterogeneous deposits at the foot of both abutment slopes and above el 900.0 on the right abutment slope; the steep slope of the right abutment has little or no overburden. The valley floor is covered with Recent alluvium. Evidence of landslides exists at various points throughout the area.

8. Structurally, the bedrock masses have been tilted but not excessively folded, and the angle of dip is relatively small. Major faulting was not observed in the region, although smaller faults are known to exist. At least one small fault crosses the dam site at sta 19+80.

9. The bedrock of the left abutment is covered by Recent clay talus overburden deposits that range in thickness from 90 ft at the base of the abutment slope to less than 20 ft up the slope. Tuff-breccia rock underlies the left abutment but does not outcrop.

10. Overburden is negligible or nonexistent on the steeper portions of the right abutment, but on the more gentle slopes above el 900.0 Recent heterogeneous deposits as much as 40 ft thick occur in places. These deposits are unconsolidated mixtures of clay talus, slope wash, and residual clay derived from weathering of the andesite.

11. The overburden of the valley floor, or flood plain, consists of Recent alluvium of unconsolidated river gravels and boulders ranging in depth from 5 to 40 ft which, in turn, are overlain by a thin, 5- to 10-ft cover of silty soil. The uppermost 20 ft of the gravel overburden is usually quite pervious; the underlying gravel contains more silt and is much less permeable. These gravels are essentially composed of andesitic and basaltic materials, although some rhyolitic lava and tuff are present.

12. Geologic profiles along the center line of the embankment foundation are shown in plate 2.

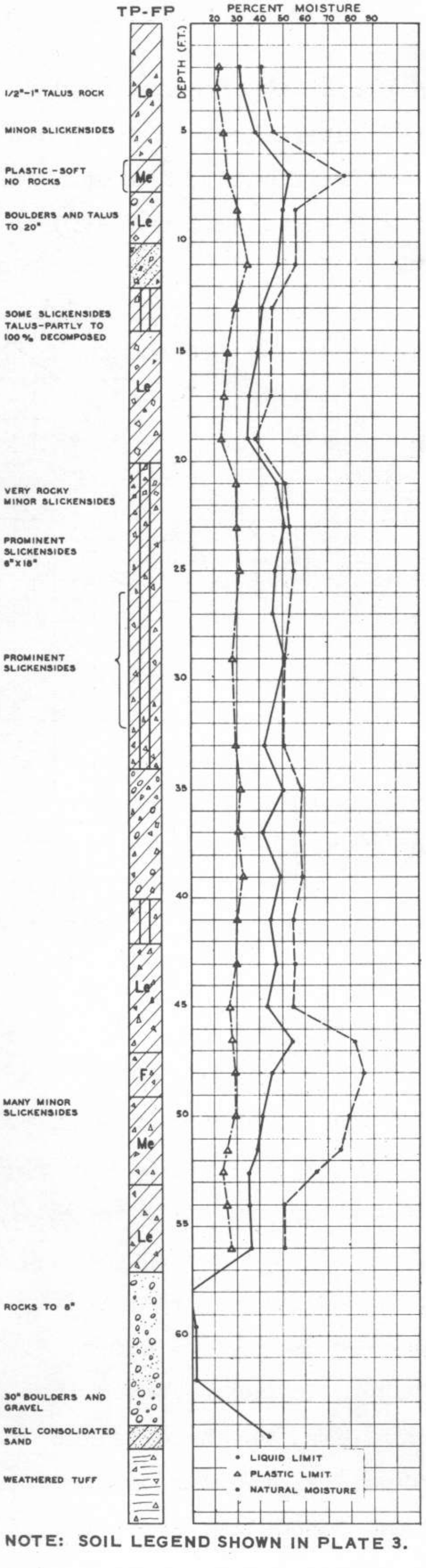
Field exploration

13. Field explorations of the embankment foundation consisted of 82 core-drill holes, 47 churn-drill holes, one 36-in.-diam calyx hole, 4 tunnels, 15 test pits, and 2 trenches. A plan of the foundation exploration is shown in plate 3, except for a few borings that were performed at an alternate dam site located 1700 ft upstream from the selected site. The core drilling was accomplished chiefly by 1-5/8-in. (BX) and 2-1/8-in. (NX) bits, but one boring (DH-110) was advanced with a 6-in. bit.

Laboratory tests on soils

14. Laboratory tests were performed on undisturbed foundation clay talus soils of the left abutment area. The other portions of the embankment foundation were to be excavated down to gravel or to bedrock. The tests performed on the clay talus materials consisted of 111 mechanical analyses, 110 Atterberg limits, 73 unconsolidated-undrained triaxial shear (UU-T), 36 consolidated-drained direct shear (CD-D), 46 consolidation, and 55 permeability tests. Samples for these tests were obtained from churn-drill holes, test pits, and trenches.

15. Mechanical analyses and Atterberg limits. Results of mechanical analysis tests on 11 typical clay talus foundation samples are shown in the table in fig. 2. The classification data shown are based on the U. S. Bureau of Soils Classification System. The average of the grain-size values given indicates about 67% of the material to be clay size (<0.005 mm), 25% to be silt size (>0.005 and <0.05 mm), 6% sand size (>0.05 and <1.0 mm), and 2% gravel (>1.0 mm). The natural water content of these samples ranged from 34.5 to 58.1% of dry weight and averaged about

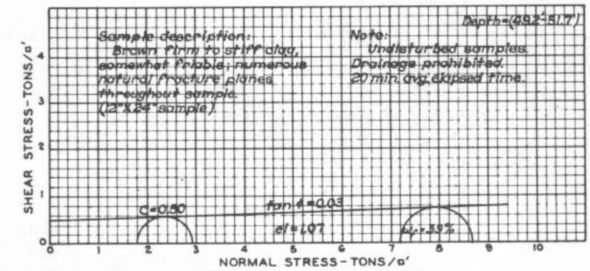


NOTE: SOIL LEGEND SHOWN IN PLATE 3.

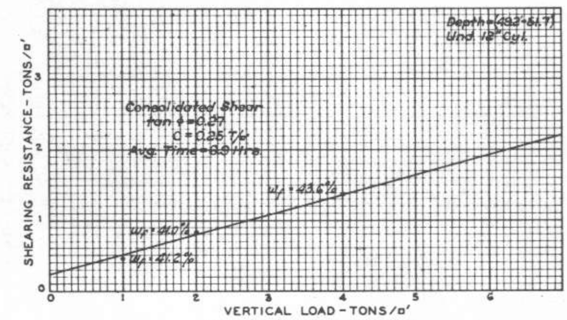
REF NO.	DEPTH (ft.)	CLASS. DATA * Clay %	Silt %	Sand %	Grav. %	ATT. LIM. L.L. %	P.L. %	Nat. e %	Nat. w %	SHEAR DATA Tan. φ	C T/□'	CONSOL. & PERM. t_{50} min.	Preconsol. Ld.-T/□'	k x 10^{-8} ft./min.
						(TRIAX. RESULTS)								
1	7-8	60	26	7	7	84	28.8	1.19	44.4	0.05	0.37	17.5	4.0	9
2	29.2-30.2	38	41	19	2	55.2	29.3	1.44	47.6	0.12	0.46	4.0	6.5	290
3	46.5-49.0	78	19	3	0	81.7	30.6	1.27	46.8	0.05	0.69	–	6.1	29
4	47.4-48.4	73	20	0	7	87.5	33.3	1.31	48.8	0.05	0.70	250		8
5	49.2-51.7	68	25	7	0.	73.5	26.8	1.07	39.9	0.03	0.50	16.0	5.4	21
6	51.0-54.0	67	26	7	0	72.9	27.7	0.935	34.5	0.02	0.65	–		
						(DIRECT SHEAR RESULTS)								
7	46.4-47.4	70	26	4	0	94.1	34.8	1.437	58.1	0.25	0.22	19.0	3.6	12
8	46.5-49.0	69	28	3	0	91.5	31.6	1.433	51.4	0.21	0.20	–		
9	49.2-50.2	74	22	4	0	83.5	30.3	1.189	44.3	0.28	0.17	27.0	6.8	4.4
10	49.2-51.7	75	18	7	0	73.5	26.8	1.200	43.6	0.27	0.25	–		
11	51.0-54.0	65	28	7	0	72.9	27.7	1.001	34.8	0.29	0.20	–		

* BASED ON U. S. BUREAU OF SOILS CLASSIFICATION

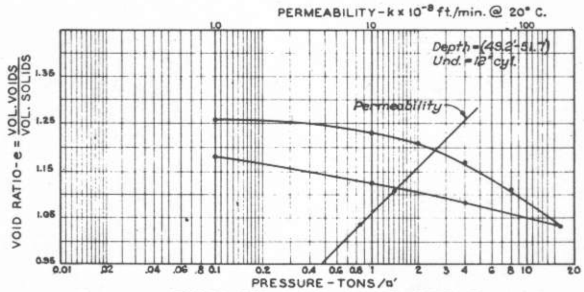
TYPICAL DATA



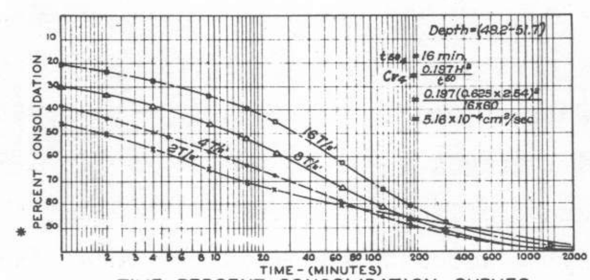
TRIAXIAL SHEAR TEST



DIRECT SHEAR TEST



PRESSURE- VOID RATIO CURVE

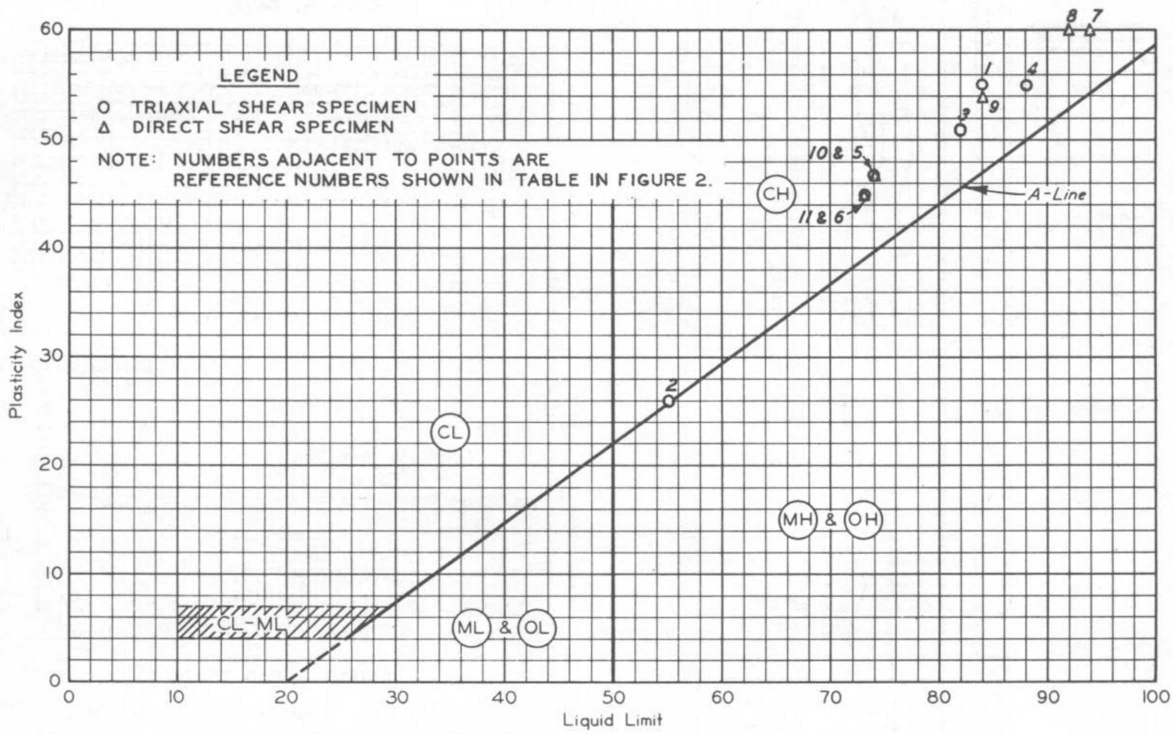


TIME-PERCENT CONSOLIDATION CURVES

* 0 PER CENT CONSOLIDATION CORRESPONDS TO VOID RATIO AT START OF LOAD INCREMENT.

Fig. 2. Typical test data for foundation soils

45% of dry weight. The liquid limit (LL) and plasticity indexes (PI) are shown plotted in fig. 3 and reveal that the materials were fat clays (CH) based on the currently used Unified Soil Classification System. The liquid limit and plasticity indexes ranged from 55.2 to 94.1 and 25.9 to 59.9, respectively, and averaged 79.1 and 49.4.



PLASTICITY CHART

Fig. 3. Plasticity chart for typical foundation soils

16. Shear tests. UU-T shear tests were performed on sixty-six 2.8- by 6.0-in., undisturbed specimens and on seven 12- by 24-in., undisturbed specimens. The shear strength values for the smaller specimens ranged from c = 0.10 to 1.55 tons per sq ft and $\phi = 0.0$ to 24.7° and averaged c = 0.83 ton per sq ft and $\phi = 6.1^{\circ}$; shear strength values for the larger test specimens ranged from c = 0.50 to 1.37 tons per sq ft and $\phi = 0.6$ to 3.4° and averaged c = 0.92 ton per sq ft and $\phi = 2.3^{\circ}$. It was believed that because of the structure of the clay talus materials, which contained varying amounts and sizes of gravel and slickensides, the larger specimens gave values that were more representative of the foundation strength. All the samples for the large specimens were obtained from test pit F-P (see plate 3). Check tests were performed using regular 2.8- by 6.0-in.

specimens cut from undisturbed cubic foot samples obtained from the same location as the large specimens, and also using small specimens taken from undisturbed portions of the large specimens after they were tested. It was found that the tests on the larger specimens indicated strength values slightly lower than those on the smaller specimens of similar or same materials. The shear strength determined on eleven smaller specimens taken from undisturbed portions of the seven larger specimens ranged from $c = 0.20$ to 1.40 tons per sq ft and $\phi = 2.9$ to 15.0° and averaged $c = 0.76$ ton per sq ft and $\phi = 7.2^{\circ}$. A typical triaxial shear strength curve for the clay talus materials is shown in fig. 2.

17. Thirty-six CD-D shear tests performed on 4- by 4- by 1-in. specimens prepared from undisturbed 12-in.-diam samples indicated strengths ranging from $c = 0.10$ to 0.79 ton per sq ft and $\phi = 11.3$ to 38.0° and averaged $c = 0.27$ ton per sq ft and $\phi = 24.0^{\circ}$. A typical shear strength curve for these tests is also shown in fig. 2.

18. Consolidation. Forty-six consolidation tests were performed on the clay talus materials. A typical consolidation vs time plot is shown in fig. 2, together with a pressure vs void ratio curve. For the typical data shown, it may be seen that the time for 50% laboratory consolidation (T_{50}) is 16 minutes for the load of 4 tons, and the coefficient of consolidation (C_v) for the 4-ton load is 5.16×10^{-4} sq cm per sec.

19. Permeability. Permeability tests were performed on undisturbed foundation clay talus materials in an adapted consolidometer. Results of 55 tests indicated coefficients of permeability ranging from 0.03 to 4750×10^{-8} cm per sec, with an average value of 560×10^{-8} cm per sec. Results of a test on a specimen considered typical of the more impermeable foundation clay talus is shown in the pressure-void ratio plot in fig. 2. Falling-head-type field permeability tests on the foundation gravels were performed in the exploration trench on the left abutment. Values of 15 typical tests ranged from 0.04 to 15.42×10^{-4} cm per sec, and averaged 4.76×10^{-4} cm per sec.

Laboratory tests on rock

20. Laboratory tests were performed on representative samples of the foundation rock from core boring DH-110 in the valley section near the right abutment and included unconfined compression, specific gravity, and

absorption tests. The modulus of elasticity and shearing resistance were determined using data obtained from the compression tests.

21. The compressive strength of the foundation rock as determined by five unconfined compression tests on 4-7/8- by 12-in. cylinder specimens from boring DH-110 averaged 5800 psi. These tests also indicated the modulus of elasticity to be 3,500,000 psi. It was recognized that the unconfined compression tests were performed on sound unjointed rock cores, and that the actual strength of the foundation rock mass would be substantially less than that indicated by these tests.

22. Relative density and absorption tests were performed on fragments of approximately 2-cu-in. volume. The specific gravity of six specimens of the rock averaged 2.39, with a corresponding computed average weight of 149.0 lb per cu ft. Absorption tests were performed for three periods, viz., 24, 96, and 336 hours, and showed percentages of absorption ranging from 2.3 to 4.2, 3.0 to 5.0, and 3.6 to 6.2%, respectively.

Embankment

Materials

23. Sufficient quantities of materials suitable for construction of the embankment were located within the reservoir area by field exploration. A plan of the borrow areas is shown in plates 4 and 5, together with the plan of borings. In general, silts were found to exist in scattered, shallow deposits in the flood plain. Suitable clays were found as a terrace deposit on the left slope of the valley about two miles upstream from the dam site. Random gravel deposits were found exposed in the river bed and beneath the silt overburden in the flood plain.

24. Filter materials would be available by processing the 1-in. minus fraction from select gravel deposits or by crushing rock from the spillway and tailwater-channel excavation. Rock from the spillway and channel excavation would also be suitable for the dumped-stone revetment and the downstream rock toe.

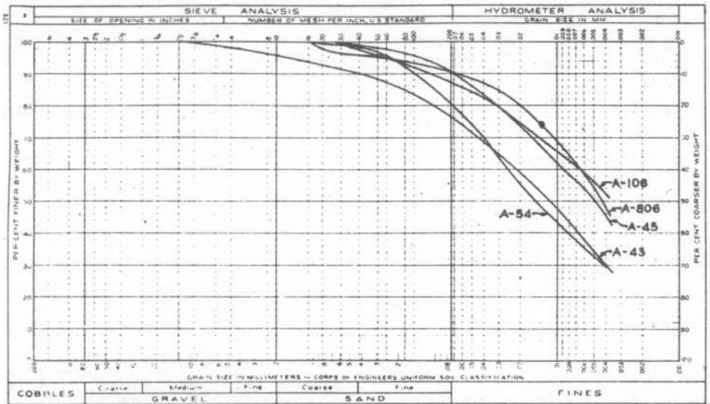
Tests on laboratory-prepared specimens

25. Laboratory tests performed on embankment materials consisted of mechanical analysis, Atterberg limits, water content, compaction,

unconsolidated-undrained triaxial shear, single-phase triaxial shear, consolidated-drained triaxial shear, consolidated-drained direct shear, consolidation, and permeability. These tests were performed on representative samples and mixtures of the silts, clays, and random gravel materials.

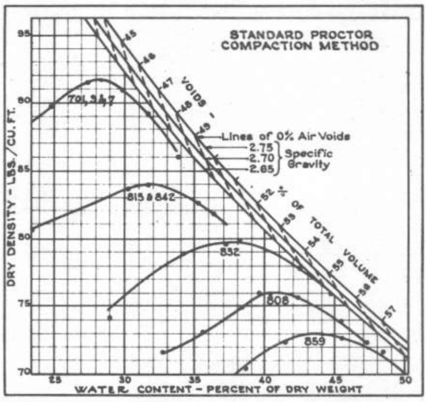
26. Compaction tests. Standard (Proctor) compaction tests were performed on the impervious clay and silt borrow material. Results of tests on nine typical samples are shown plotted in fig. 4. It may be seen that

TERRACE CLAY



BORROW AREA	HOLE NO.	DEPTH-FT.	CLASSIFICATION	NAT. W.C.	L.L.	P.L.	P.I.
1	A-45	2.5-6.5	Me. Clay	46.2	74.0	3.0	41.0
2	A-43	3.0-4.0	Le. Clay	17.7	38.0	24.0	14.0
3	A-806	2.5-5.0	Me. Clay	41.2	116.6	49.7	66.9
4	A-106	2.0-3.0	Me. Clay	33.4	85.0	26.1	58.9
5	A-54	0.5-4.5	Le. Clay	48.5	60.0	35.0	25.0

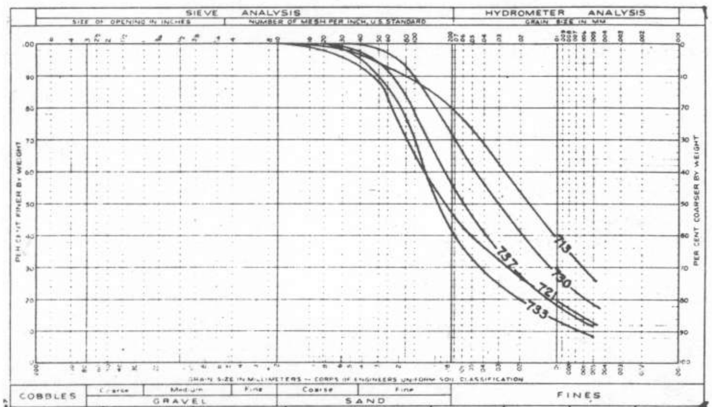
TYPICAL M. A. CURVES



Field Number	701,703,707	815 & 842	832	808	859
Classification	Le. Clay	Me. Clay	Me. Clay	Le. Clay	Le. Clay
Maximum Dry Density P.C.F.	91.5	84.0	79.8	76.0	73.0
Optimum Water Content - %	28.5	32.0	37.8	40.6	44.0
Specific Gravity	2.77	2.72	2.79	2.78	2.77
Liquid Limit	58.9	94.0	85.2	91.7	71.2
Plastic Index	33.3	61.6	46.4	46.7	21.4

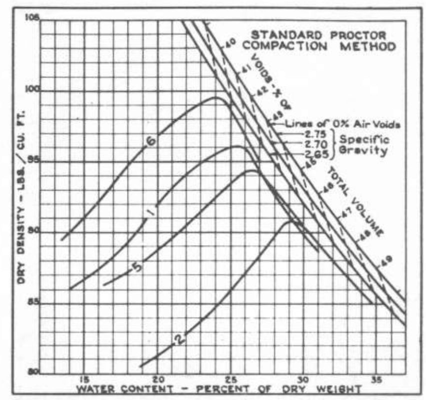
TYPICAL COMPACTION CURVES

FLOOD - PLAIN SILT



BORROW AREA	HOLE NO.	DEPTH-FT.	CLASSIFICATION	NAT. W.C.	L.L.	P.L.	P.I.
5	A-713	0.5-5.6	Clayey Silt	26.8	43.3	27.8	15.5
5	A-721	0.5-3.5	Clayey Sand	7.6	30.9	28.5	2.4
5	A-730	0.5-7.0	Clayey Silt	23.3	39.5	28.3	11.2
5	A-733	0.5-2.0	Clayey Sand		N.P.	N.P.	N.P.
5	A-737	0.5-6.5	Clayey Sand	22.6	N.P.	N.P.	N.P.

TYPICAL M. A. CURVES



Field Number	CP-1	CP-2	CP-5	CP-6
Classification	Cl Sa	Cl Si	Cl Si	Cl Si
Maximum Dry Density P.C.F.	96.2	90.8	94.6	99.6
Optimum Water Content	25.8	29.2	26.5	24.0
Specific Gravity	2.73	2.73	2.73	2.76
Liquid Limit	35.1	44.2	41.4	31.0
Plastic Index	4.5	14.4	12.7	9.3

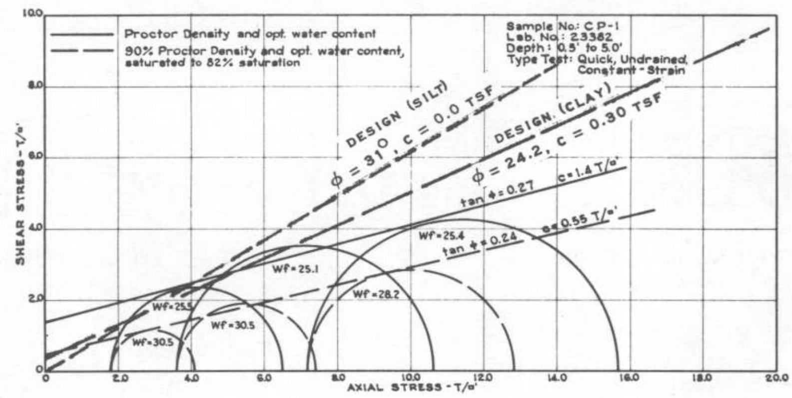
TYPICAL COMPACTION CURVES

Fig. 4. Typical classification data, mechanical analysis, and compaction curves for laboratory-compacted impervious borrow materials

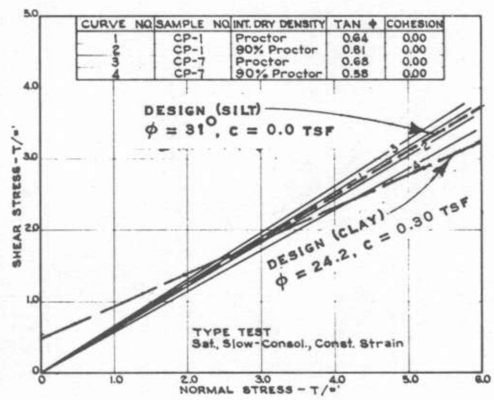
the maximum dry densities ranged from about 73 to 92 lb per cu ft for the clay, and from about 90 to 100 lb per cu ft for the silt. The optimum water contents ranged from about 28 to 44 and from about 24 to 29% dry weight for the clay and silt, respectively.

27. Standard compaction tests were not performed on the random borrow materials. Compaction characteristics were determined by constructing test fills that were compacted by sheepsfoot and pneumatic-tired rollers, and obtaining and testing samples from these test fills; results of these tests will be discussed later.

28. Shear tests. Shear tests on laboratory-compacted impervious core materials included UU-T and CD-D tests. The results of two UU-T and four CD-D shear tests performed on representative silt specimens are shown in fig. 5. One of the UU-T tests was performed on material compacted to



TRIAXIAL SHEAR TESTS



CONSOLIDATED DIRECT SHEAR TESTS

Fig. 5. Results of shear tests on laboratory-compacted flood-plain silt borrow material

90% of standard density and the other on material compacted to standard density. Two of the CD-D tests were performed on materials compacted to 90% of standard density and two on materials compacted to standard density. Results of the UU-T tests indicated shear strength values of c = 0.55 ton per sq ft and $\phi = 13.5^{\circ}$ for the material compacted to 90% of standard density, and c = 1.4 tons per sq ft and $\phi = 15.1^{\circ}$ for the material compacted to standard density. Results of the CD-D shear tests indicated average shear strength values of c = 0.0 ton per sq ft and $\phi = 31.0^{\circ}$ for the materials compacted to 90% of standard density, and c = 0.0 ton per sq ft and $\phi = 33.4^{\circ}$ for the materials compacted to standard density.

29. Shear strengths of impervious clay borrow material were also determined by a series of 12 constant-strain, single-phase UU-T (one-point UU test where $s = \frac{\sigma_1 - \sigma_3}{2}$) shear tests. These tests were performed on four 2.8- by 6-in. specimens of a typical sample compacted to 83 to 100% of standard density with water contents varying from about 4% dry to 6% wet of optimum. The relations of shear resistance to various densities and water contents are shown in fig. 6.

30. CD-D shear tests were performed on specimens of clay borrow material to determine the maximum shear strength. Results of only one of

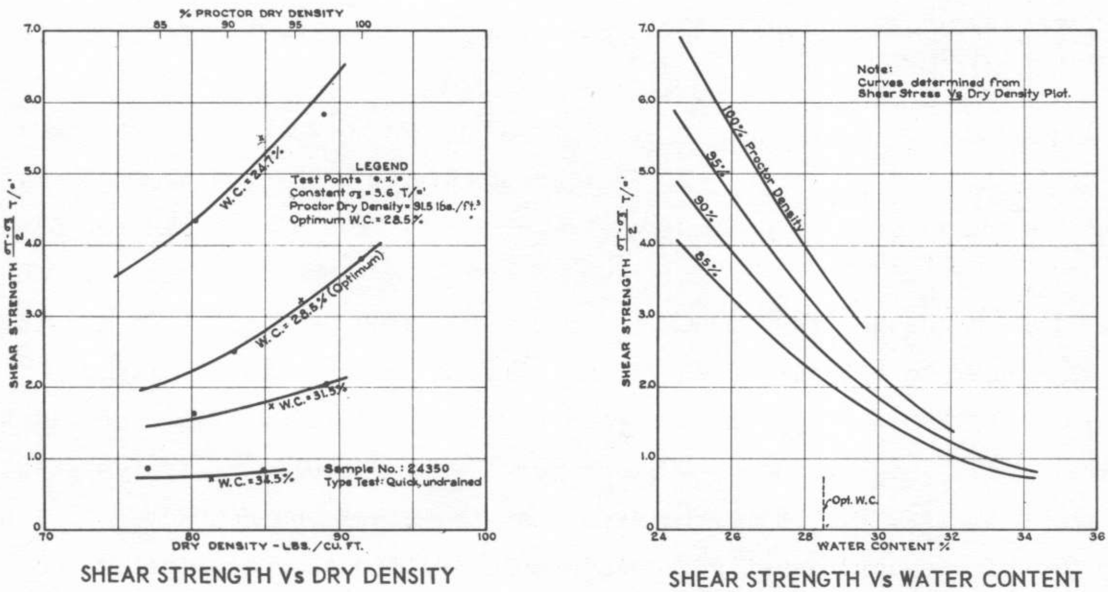
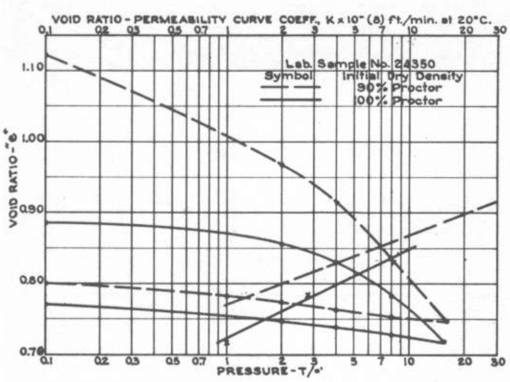


Fig. 6. Shear strength of clay borrow material vs density and water content

these tests were available. This test indicated a shear strength of c = 0.63 ton per sq ft and $\phi = 21.8^{\circ}$ for a material compacted to 95% of standard density at optimum water content.

31. Consolidated-drained triaxial (CD-T) shear tests on the random gravel borrow material were performed on 12- by 24-in. specimens compacted by rodding and by vibration and sheared under saturated and drained conditions. The large specimens were used rather than specimens of more conventional size because material up to 3 in. in size was included in the specimens in order to obtain a shear strength as representative of that of the prototype embankment as possible. The densities of the specimens are tabulated in fig. 8, which also shows typical data obtained in other tests of the random gravel borrow materials. For the 18 test results available for this review, the shear strength averaged c = 0.0 ton per sq ft and $\phi = 43.5^{\circ}$.

32. Consolidation. Consolidation tests were performed on the impervious clay core materials. Two typical pressure vs void ratio curves are shown in fig. 7, one for material compacted to 90% of standard density and the other to standard density.

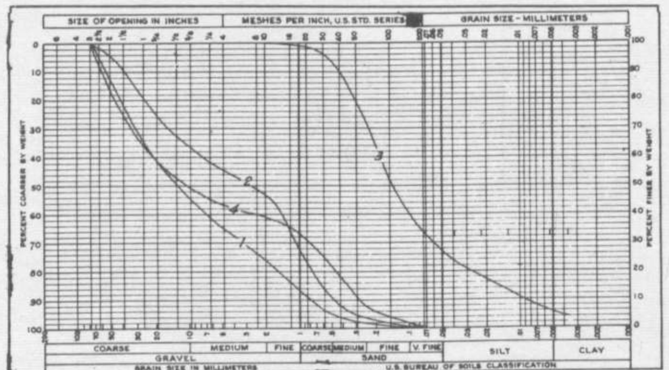


PRESSURE - VOID RATIO CURVES

Fig. 7. Typical pressure-void ratio curves for clay borrow materials

33. A series of special tests was performed on the impervious silty core materials to determine the minimum density and water content at which these materials could be placed so that excessive consolidation within the embankment would not occur after saturation. These tests were conducted in two stages. The specimens were first loaded in a normal manner to 8 tons per sq ft and allowed to consolidate without saturation; then the specimens were saturated, and allowed to consolidate further while still loaded at 8 tons per sq ft. Results of typical tests are shown in fig. 9. It may be seen that for specimens compacted to less than 95% of standard density, considerable additional consolidation occurred after saturation.

GRAVEL BORROW

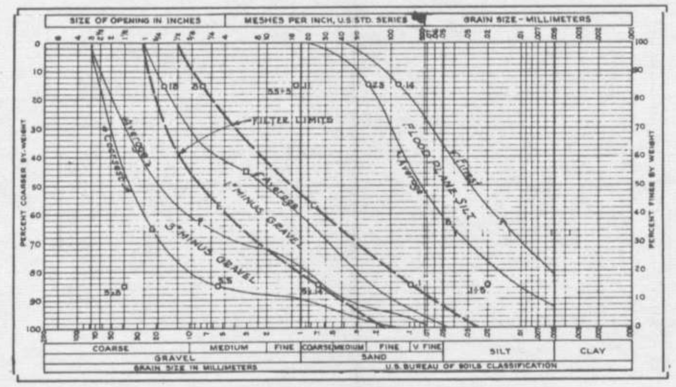


FIELD NO.	LAB. NO.	DEPTH	CLASS	L.L.	P.L.	REMARKS +3" % TOTAL SAMPLE
1	15867	0.0 - 2.5	Co. Gravel	Non-Plastic		30
2	15868	2.5 - 5.3	Gr. & Sa.	"	"	10
3	15869	5.3 - 7.0	Si. Sand	"	"	0
4	15870	7.0-10.0	Gr. & Sa.	"	"	30

TYPICAL M. A. CURVES

TEST PIT B-Q

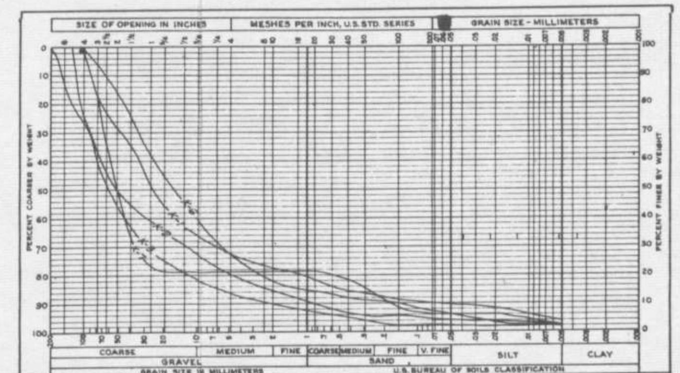
FILTER REQUIREMENTS



* Coarsest material tested with +3" material removed.

FILTER LIMITS

FOUNDATION GRAVELS LEFT ABUTMENT



FIELD NO.	LAB. NO.	ELEV.	CLASS	REMARKS
K-1	22040	771.5	Gr. & Sa.	
K-2	22041	768.5	Gravel	Cemented
K-3	22042	765.5	Gravel	Cemented
K-6	22045	758.0	Gravel	Well Cemented Max. 10"
K-7	22046	759.0	Gr. & Sa.	Poorly Cemented

TYPICAL M. A. CURVES

EXPLORATION TRENCH

TYPICAL SHEAR TEST NO.	PIT NO.	DEPTH OF SAMPLE	LAB. NO.	DENSITY LBS/CU.FT.	PRINCIPAL STRESSES AT FAILURE T/□' MINOR	MAJOR	SHEAR STRENGTH C=0 TAN. ø	PERMEABILITY 10^{-4} CM/SEC
1	B-P	2.0-11.5	15853-7	130.3	2.0	11.35	0.98	25
					5.0	23.50	0.86	
2	B-P	2.0-11.5	15853-7	129.4	2.0	12.42	1.04	26
					5.0	25.45	0.91	
3	B-P	2.0-11.5	15853-7	134.4	5.0	26.20	0.92	
4	B-P	2.0-11.5	15853-7	134.1	5.0	24.70	0.88	
	B-S, B-T, B-U	0.0- 7.5	Comp.#1	135.4	2.0	8.99	0.85	
					5.0	25.04	0.91	
	B-S, B-T, B-U	0.0- 7.5	Comp.#1	141.2	5.0	32.30	1.05	
5	B-W, B-X	2.0-12.4	Comp.#2	141.0	2.0	14.62	1.12	9
					5.0	27.55	0.95	
	B-W, B-X	2.0-12.4	Comp.#2	140.9	5.0	34.40	1.12	
	B-Z, B-AA, B-AC		Comp.#3	141.5	2.0	15.64	1.21	29
				140.1	5.0	28.05	0.98	
	B-W, B-Y, B-AC		Comp.#4	132.6	2.0	12.06	1.10	25
				131.1	5.0	27.55	0.95	
	B-Q, B-AB		Comp.#5	132.9	2.0	9.61	0.92	1.4
				131.1	5.0	23.60	0.87	2.2
6	B-R-B-AD	0.0-11.6	Comp.#6	133.2	2.0	10.64	0.92	0.33
7				134.0	5.0	22.38	0.83	1.64

Note:
Constant strain Triaxial Tests on remolded 12" x 24" specimens. Samples saturated, drainage allowed during test.

SHEAR DATA

CRITICAL FILTER RATIOS (CASAGRANDE)

1. To prevent erosion of adjacent material

$$\frac{\text{15\% Size of coarser material}}{\text{85\% Size of finer material}} = <5$$

A. Core Vs Filter

$$\frac{\text{15\% Size of filter material}}{\text{85\% Size of core material}} = \frac{D}{A} = \frac{23}{.14} = 1.6$$

15 % filter limit = .14 x 5 = .70 mm.

B. Filter Vs Gravel Shell

$$\frac{\text{15\% Size of gravel}}{\text{85\% Size of filter}} = \frac{G}{D} = \frac{5.5}{18} = 0.30$$

85% filter limit = 5.5 ÷ 5 = .11 mm. or
15% Size of gravel could be = 8 x 5 = 40 mm.

2. To insure adequate permeability in filter layer

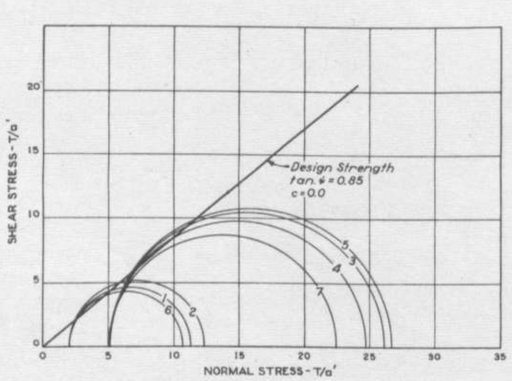
$$\frac{\text{15\% Size of filter material}}{\text{15\% Size of core material}} = >5$$

$$\frac{C}{B} = \frac{0.1}{0.012} = 8.3$$

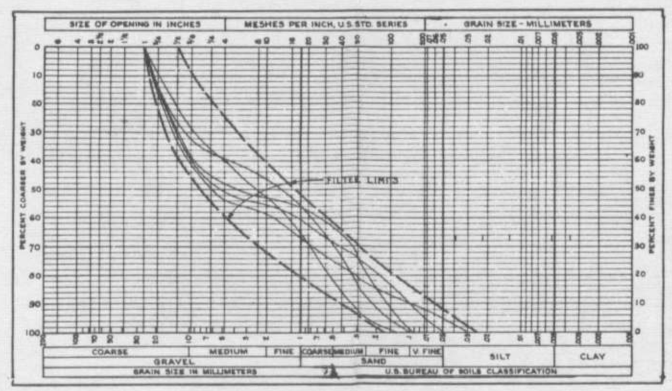
TEST NO.	STATION	ELEV.	LAB. CLASS	AVE. PERM. COEFF. 10^{-4} Feet/Min.	REMARKS
K-1	14+00-60' Lt.	771.5	Gr. & Sa.	7.35	Silty, sandy gravel.
K-2	14+00-60' Lt.	769.0	Gravel	1.20	Cemented sandy gravel, short on fines.
K-3	14+00-60' Lt.	765.5	Gravel	4.82	Cemented sandy gravel, short on fines.
K-4	14+00-60' Lt.	763.0	Gravel	3.68	Cemented sandy gravel, well blended.
K-5	14+00-55' Lt.	760.0	Gravel	14.10	Cemented sandy gravel, short Me. Sa. excess, Fi. Sa.
K-6	14+00-50' Lt.	758.0	Gravel	21.30	Well cemented sandy gravel, max. size 10".
K-7	14+00-15' Rt.	759.0	Gr. & Sa.	16.70	Medium to fine sand poorly cemented, sand well distributed.
K-9	9+60-10' Lt.	777.0	Gravel	0.88	Well cemented sandy gravel, short on fine sand.
K-10	9+60-℄	775.0	Gravel	4.50	Well to moderate cemented medium sandy gravel.
K-11	9+60- 5' Rt.	773.5	Gravel	2.57	Well to moderate cemented medium sandy gravel.
K-12	10+00-25' Lt.	775.0	Gravel	0.08	Well cemented sandy gravel.
K-13	10+00-24' Lt.	773.0	Gravel	10.70	Well cemented medium sandy gravel.
K-14	10+25-28' Rt.	777.0	Gravel	14.10	Well to moderately well cemented sandy gravel.
K-15	14+00-44' Lt.	757.0	Gravel	8.45	Well cemented medium sandy gravel.
K-16	14+00-42' Lt.	755.0	Gravel	30.40	Well cemented sandy gravel.

Note:
1. Tests were run in the exploration trench after excavation.
2. Average permeability values determined from a minimum of two tests.
3. See M. A. curves for gradation.

FIELD PERMEABILITY DATA



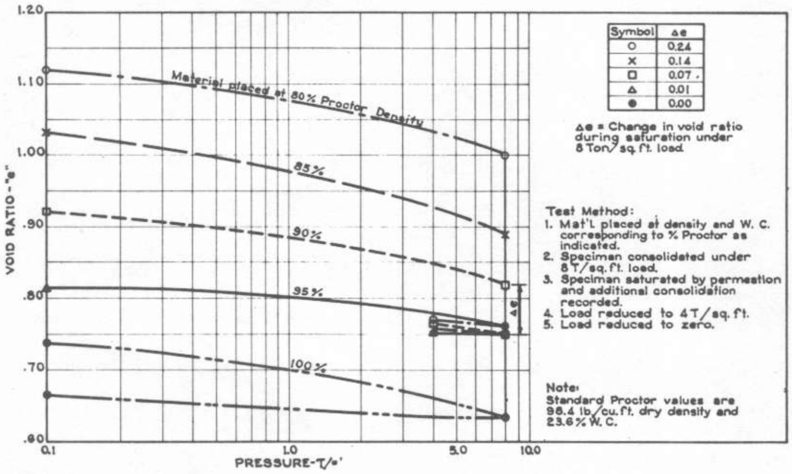
TYPICAL SHEAR CURVES



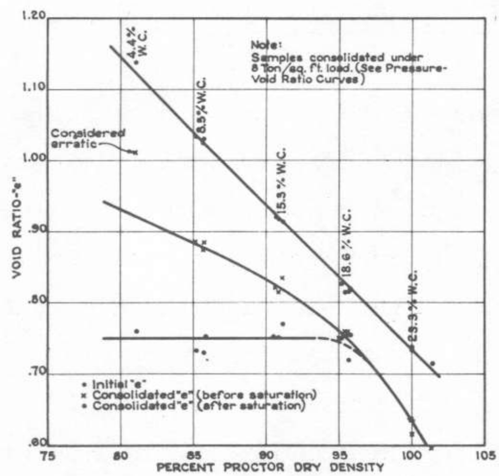
TYPICAL M. A. CURVES

1" MINUS GRAVEL BORROW

Fig. 8. Typical test data for laboratory-compacted random gravel borrow materials



PRESSURE - VOID RATIO CURVES



PERCENT PROCTOR DENSITY - VOID RATIO CURVES

Fig. 9. Typical pressure- and density-void ratio curves for silt borrow material

34. Permeability. Permeability tests were made on compacted impervious silty core material specimens after completion of the consolidation tests. Results of three tests available for this study indicate a range of coefficients from 11 to 152 x 10^{-8} cm per sec, with an average value of 52 x 10^{-8} cm per sec.

35. Results of permeability tests performed on two typical compacted impervious core clay materials are shown in fig. 7 as the relation between void ratio and coefficient of permeability.

36. Permeability tests performed on nine typical 12- by 24-in. specimens of random gravel borrow materials gave values that ranged from 0.33 to 29 x 10^{-4} cm per sec, and averaged 13.2 x 10^{-4} cm per sec.

Test fills

37. Prior to construction of the embankment, two test-fill programs were conducted to determine the best method of obtaining the desired density for the proposed embankment. The first test-fill program consisted of compaction with a sheepsfoot roller with foot areas approximating 5.75 sq in. each and with foot pressures as high as 650 psi. The second test-fill program consisted of compaction with pneumatic-tired rollers loaded to a total weight of 69,000 lb and having tire pressures of 70 psi. Respective details of these two test-fill programs may be found in the Definite Project Report on Willamette Basin Project, Oregon, Lookout Point Dam (Meridian Site), vol I (June 1946), and Construction of Test Fills, Heavy Pneumatic-tired Roller, Lookout Point Dam (April 1950), both of which were published by the Portland District.

38. The general conclusions from the test-fill programs were that a greater percentage of standard density could be achieved with fewer passes, and that thicker lifts could be compacted with the pneumatic-tired roller. More specific conclusions derived from the tests conducted with the pneumatic-tired roller were that the random gravel materials could be satisfactorily compacted in 18-in. lifts with two passes, and the silt materials could be compacted to 95% of standard density in 12-in. lifts with four passes. The medium terrace clays could be compacted to 95% of standard density in 6- to 12-in. lifts with four passes; the stiff moist clays could not be satisfactorily compacted with either the sheepsfoot or pneumatic-tired rollers. The samples obtained from test fills for density tests were apparently taken from the center of the lifts, and no data are available to determine the differences in density that may have existed between the upper and lower portions of the thicker lifts.

39. A number of undisturbed samples were taken from each test-fill strip, but results of laboratory tests performed on these samples are not available.

Analyses

40. Locally available borrow materials dictated that the embankment be designed as a zoned embankment as shown in plate 6. The random gravel material was to be used for the upstream and downstream shells, with the

coarser material being placed toward the outside and the finer materials toward the center. The impervious core was to consist of silt and clay, with a clay barrier having a minimum thickness of 8 ft on the upstream face. It was believed that this clay barrier would reduce the quantity of seepage and thereby prevent a high escape gradient and eliminate the need for an upstream filter. A downstream filter with a minimum thickness of 8 ft was to be placed adjacent to the downstream face of the core.

Selection of design values

41. The values tabulated below were adopted for design of the embankment and foundation:

Table 2

Design Strengths and Soils Constants

Material	ϕ^{o}	c tons/sq ft	k cm/sec x 10^{-8}	Unit Weight γ_m lb/cu ft	Unit Weight γ_s lb/cu ft
Foundation					
Clay talus	12.9*	0.26*	**	120	125
	1.7†	0.50†	**	120	125
Gravel	40.4	0	**	145	148
Random fill	40.4	0	**	145	148
Core section					
Clay barrier	24.2	0.30	0.50	---	---
Silt and clay	31.0	0	5.0-50.0	128	130
Berm	0	0.35	**	100	110

* CD-D shear strength for normal loads < 1.15 tons per sq ft with no consolidation and for all loads when assuming partial or total consolidation.

** Considered nonfree-draining relative to sudden drawdown for stability analyses.

† UU-T shear strength for normal loads > 1.15 tons per sq ft with no consolidation.

42. In the selection of the foundation shear strengths shown above, the minimum shear strength defined by the combined CD-D and UU-T shear curves was used for the unconsolidated foundation. The ultimate shear strength that could be developed and the strength values to be used for conditions of partial consolidation were determined by CD-D shear tests. For the condition of partial foundation consolidation, the indicated

strength was adjusted for partial consolidation. To check the validity of the selected shear strength values, an exploration trench with side slopes of 1 on 1 was excavated along the axis of the dam on the left abutment. From slides that developed, the average shear strength was determined. The results of this study, which will be discussed later, substantiated the selected values.

43. The design shear strength for the core section was selected on the basis that the core would be constructed predominantly of silt that would consolidate rapidly. CD-D shear tests for the silt were considered the applicable tests and the design value for the core section was based on these tests.

44. The shear strength of the impervious clays to be used for the vertical 8-ft-thick upstream barrier was based on results of the UU-T tests performed at various water contents and densities in order to determine the effect of placement water content on shear strength (see fig. 6). CD-D shear tests were also performed to determine the maximum ultimate strength that could be expected after sufficient time elapsed for these materials to consolidate. However, in the stability computations, the impervious core was treated as if it was composed entirely of the silt materials and, therefore, the design shear strength for the silt was also used for the clay barrier.

45. The design shear strength of $c = 0.0$ ton per sq ft, $\phi = 40.4^{\circ}$ for the relatively free-draining random materials was based on CD-T shear tests performed on compacted specimens 12 in. in diameter and 24 in. high.

46. The berm material was to be uncompacted random fill and rock. A design shear strength value of $c = 0.35$ ton per sq ft, $\phi = 0.0^{\circ}$ was selected and used in the stability analyses.

Field determination of shear strengths

47. As stated earlier, before the final design of the embankment, the shear strength of the clay talus materials of the left abutment was determined by a study of slides that occurred in the exploration-trench excavation. The sides of this excavation were cut to 1-on-1 slopes so this could be accomplished. The slides were analyzed and an over-all shear strength was determined that agreed with the selected design shear strength

values shown in table 2, which were based on results of laboratory tests.

Stability analyses

48. The stability of the embankment and foundation was determined by the Swedish circular-arc method. The foundation between the left abutment (sta 5+25) and sta 14+50 consists of clay talus deposits; the remaining portion of the foundation consists principally of gravels in the flood plain and exposed bedrock on the right abutment. Typical sections analyzed for the left abutment and valley sections are shown in plates 7 and 8, respectively, together with the conditions for which the sections were analyzed and the failure arcs for which stability was computed. Stability for conditions immediately after construction, sudden drawdown, and steady seepage was determined for the left abutment section, and stability for sudden drawdown and steady seepage was determined for the valley section.

49. Left abutment. For the immediately-after-construction condition for the left abutment section, it was assumed that no consolidation had occurred in the clay talus foundation material. The minimum computed factors of safety for sections at sta 7+00 and 8+00 were 1.53 and 1.37, respectively, with the failure arcs penetrating approximately two-thirds of the clay talus material.

50. The minimum factor of safety for the sudden-drawdown condition at sta 8+00 was based on the assumption that sudden drawdown occurred before the clay talus materials had fully consolidated and the reservoir was drawn down from normal pool (el 929.0) to minimum power pool (el 811.0). The per cent the clay talus foundation had consolidated along the failure arc was computed at selected points along the arc according to their relative positions within the clay talus stratum. The effective embankment weight used in the analysis was the total weight of saturated soil less neutral "standpipe curve" pressures (see plate 7) determined from flow nets. The force developing effective intergranular stresses along the failure arc in the foundation was assumed to be equal to the effective embankment weight above a given point times the per cent consolidation at that point. The tangential, or unbalancing, forces were assumed to be produced by the saturated weight of the soil. It may be seen in plate 7 that the critical failure arc is located in the embankment and clay talus material and does not penetrate into the foundation gravel. The minimum factor of safety for

the case of sudden drawdown and partially consolidated foundation was 1.25, which was considered adequate for the severe conditions assumed.

51. The steady-seepage analysis for the left abutment section (sta 7+00) was conducted assuming foundation conditions similar to those described in the preceding paragraph. The reservoir was assumed to be at normal pool (el 929.0) and the tailwater at ground-surface elevation. The random gravel-filled trench downstream from the impervious core contributed materially to the stability of the downstream slope. The factor of safety, assuming steady seepage with the foundation partially consolidated, was 1.64; with the foundation fully consolidated, the factor of safety was 1.74.

52. Valley section. The stability analyses for the valley section are shown in plate 8. The embankment in the valley is on a gravel foundation, as previously mentioned, and pore pressures in the foundation were not of concern in the stability analyses; therefore, the valley section of the embankment was not analyzed for the immediately-after-construction condition as was the left abutment section.

53. Stability analyses for a condition of sudden drawdown from maximum to minimum power pool indicated a minimum factor of safety of 1.32. The flow net developed for this condition and for the condition of steady seepage is shown in plate 9. It was believed that the factor of safety of 1.32 would decrease slightly for very shallow arcs paralleling the steepest slope. However, the stability of the embankment was considered to be adequate for this sudden-drawdown condition. The minimum factor of safety for a condition of steady seepage was computed using the assumptions described for analyses of the left abutment section and was 1.96 for a circular-arc failure surface. The factor of safety decreased to 1.48 for a shallow failure surface paralleling the steepest slope. These factors of safety were considered adequate.

Foundation and embankment consolidation

54. The settlement of the foundation was not computed because in the left abutment area, the embankment was relatively low and it was expected that the foundation materials would consolidate slowly, and in the valley section, the embankment was to be founded on gravel and little or no foundation settlements were expected to occur.

55. Embankment consolidation was computed for the closure section at sta 20+00 and the left abutment section at sta 8+00. The consolidation was initially computed assuming a silt core since it was originally anticipated that the predominant material to be used for the core construction would be silt. Therefore, initial computations for the closure section indicated that approximately 3.8 ft of consolidation would occur after completion of construction. Settlement gages, as shown in fig. 10, were to be installed in the embankment, and the settlement estimates were to be revised during the final construction season. Revised computations, based on the clay core actually constructed, indicated total embankment consolidation of 11.0 and 26.8 ft at sta 8+00 and 20+00, respectively, with 4.6 and 9.4 ft to occur after completion of construction. The elevation of the dam crown was to be raised an amount equivalent to the predicted amount of settlement by slightly steepening the upstream and downstream slopes. The amount of consolidation of the random materials was expected to be negligible.

Seepage considerations

56. It was believed that the impervious core would be of such low permeability that the seepage loss would be negligible; therefore, computations to determine the quantity of seepage were not made. An 8-ft-thick (minimum) clay facing was to be placed on the upstream side of the impervious core to eliminate the possibility of any serious piping of the impervious zone. At the transition from the earth embankment to the concrete spillway, the ratio of the seepage path of impervious material in contact with the concrete face to the head of water from normal pool elevation is 2.5. This was considered adequate to prevent piping along the contact plane.

57. The gravel strata underlying the clay deposit on the left abutment were considered fairly impervious; however, piezometers were to be installed in these gravels to provide information on seepage pressures. The need for a drainage-collector tunnel would be determined from observations of these piezometers after water was impounded in the reservoir.

Drainage facilities

58. The natural free-draining foundation gravels beneath the valley section provided a means of drainage for the downstream portion of the embankment. An 8-ft-thick pervious filter zone was to be placed on the

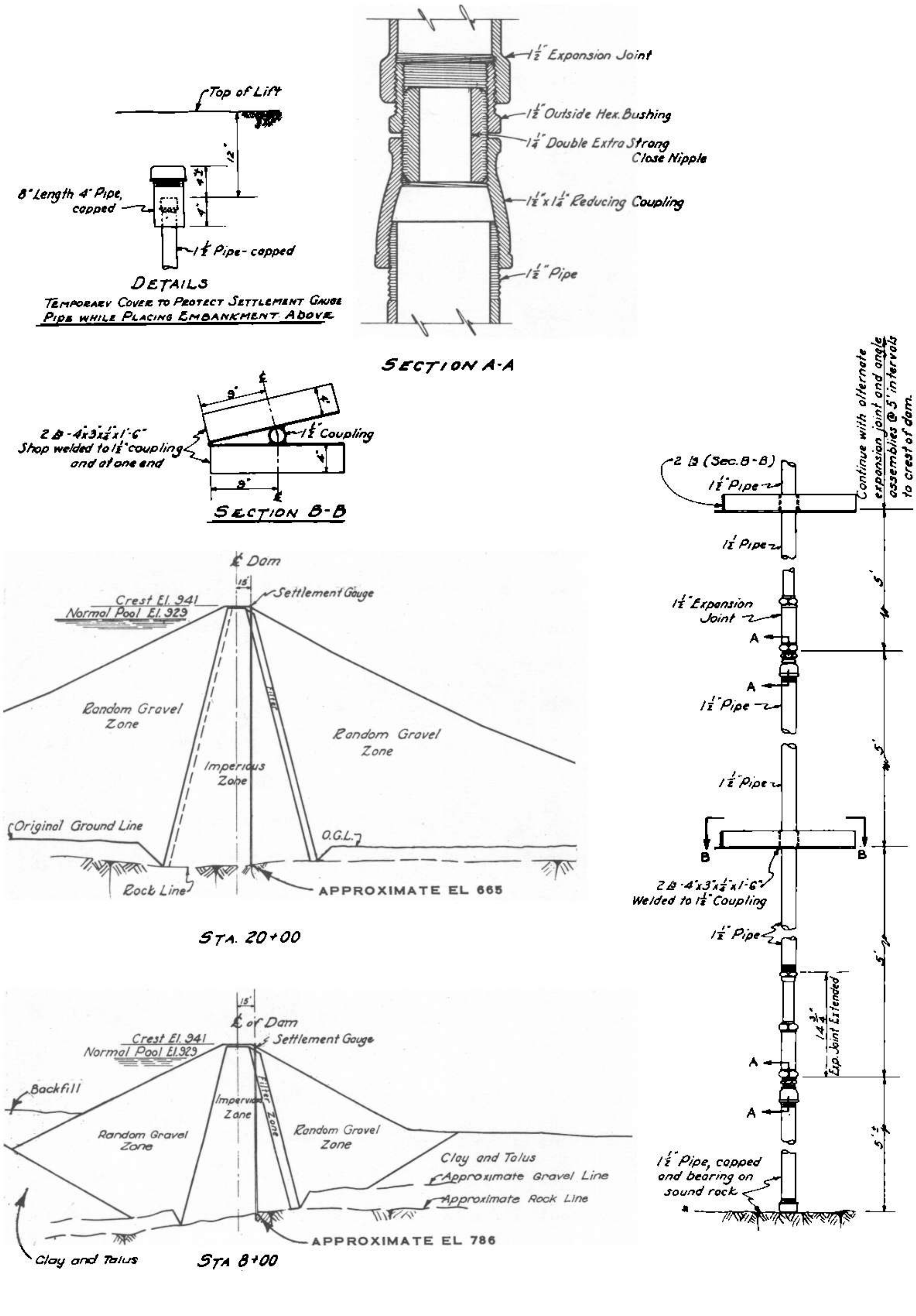


Fig. 10. Details and locations of settlement gages

downstream face of the impervious core to intercept any seepage coming through the core and to further insure against piping of the core materials.

59. In the left abutment section where the impervious core was not to be extended down to rock, a trench was to be excavated in the foundation immediately downstream of the impervious core and backfilled with free-draining gravels to intercept and carry the seepage down to the tailwater elevation (see plate 7). However, as will be discussed later in this report, a slide that occurred during excavation operations necessitated deepening the excavation for the impervious core to bedrock, and the proposed gravel-filled trench was not constructed (see plate 6).

Slope protection

60. The upstream slope was to be covered with dumped stone excavated from the spillway and appurtenances areas. A filter blanket beneath the dumped stone on the random gravel material was thought unnecessary. The stones were to be dumped on the slope in such a manner as to obtain a uniform depth of about 2 ft over the entire slope. The larger stones were to be uniformly distributed, with the smaller stones and spalls filling the spaces between the large stones. The stone weight was not to exceed 500 lb and the least dimension of each stone was to be not less than one-third the greatest dimension of that stone. The stones were to be uniformly graded from the largest to the smallest.

61. The downstream slope was to be provided with a 10-ft-thick rock toe below el 725.0. No size limit was specified for these stones. The slope above the rock toe to the top of the dam was to be constructed of random gravel and was to have no special protection.

Spillway and Outlet Works

62. The foundation for the concrete gravity-type spillway containing the outlet conduits is firm rock and, therefore, is not discussed in this report.

PART III: CONSTRUCTION

Borrow Areas

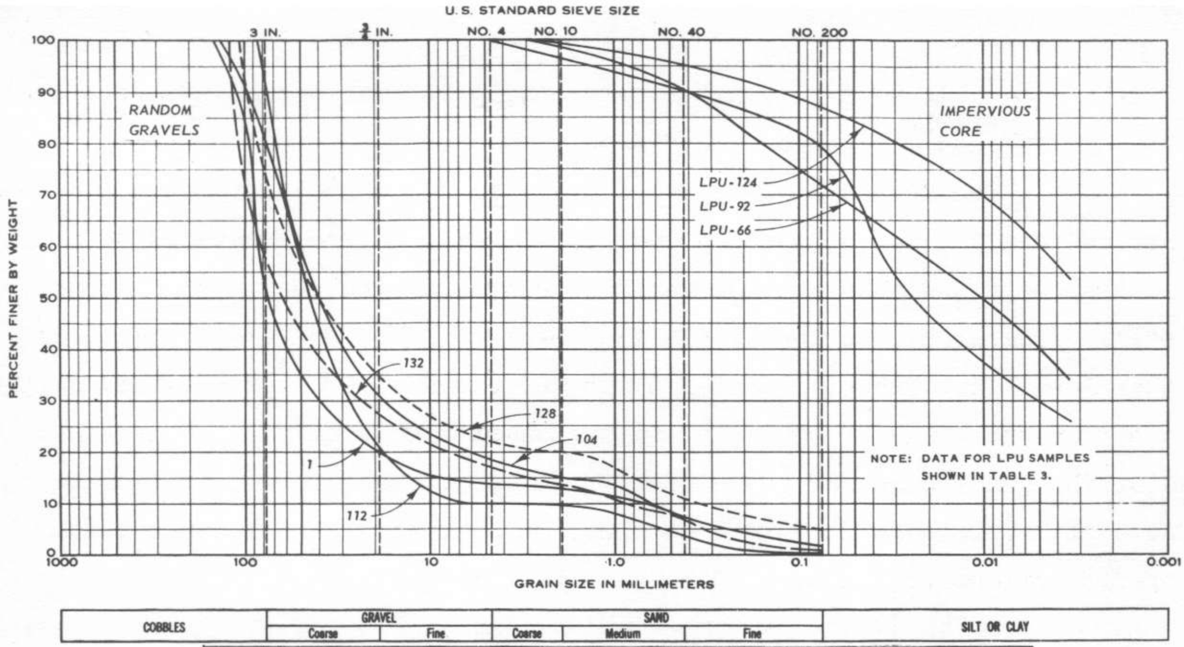
Impervious materials

63. The impervious material borrow areas that were available for use were established by field exploration prior to construction of the embankment and are shown in plates 4 and 5. Based on the embankment design, specifications required that the impervious core be constructed of silt and clay or clay alone. As may be seen in plates 4 and 5, the principal designated impervious silt borrow areas were located at distances ranging from about 2 to 7-1/2 miles upstream from the dam site. The clay borrow area (area 10) was about 1-3/4 miles upstream from the dam site. Because of the natural high water content and the shorter construction season for the clay, it was thought that the use of silts might be more desirable for construction of the impervious core. However, the contractor elected to use the closer clay materials, and a total of 1,157,600 cu yd was used in the core. Construction notes indicate that aeration and large borrow area surfaces were the principal means of reducing clay water contents to within satisfactory workable limits. Typical gradation curves for the impervious clay core material are shown in fig. 11.

Random gravel, filter, and stones

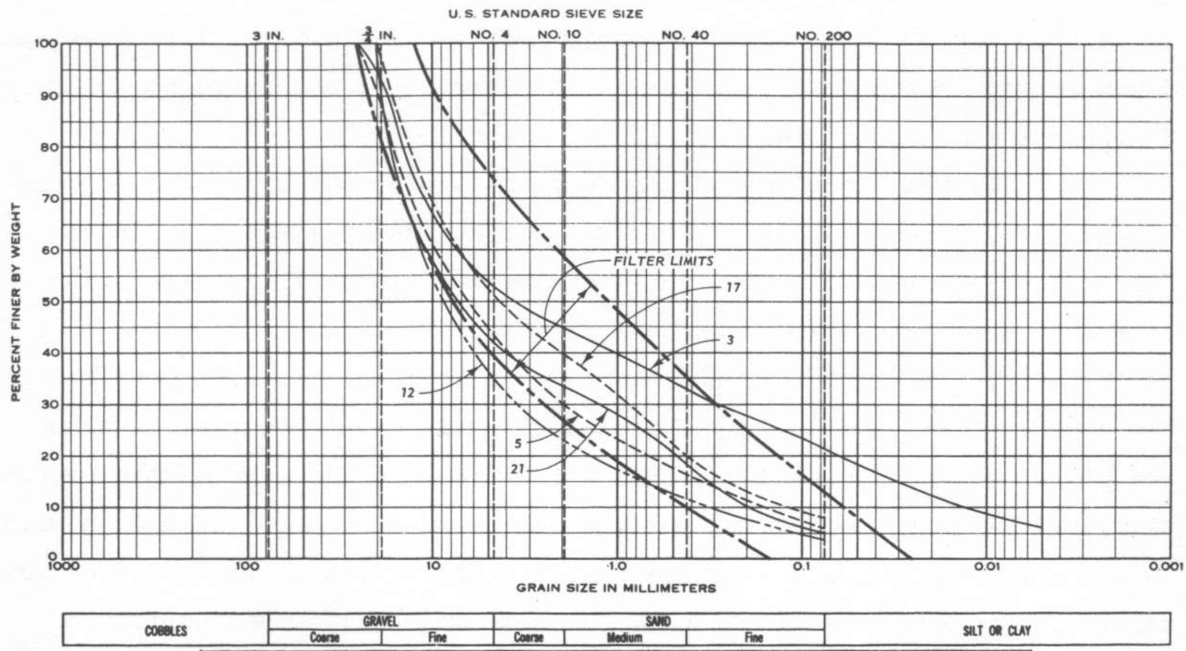
64. The random gravel material was obtained from within the reservoir area upstream from the axis of the dam below el 929.0, and from excess rock from required excavations. The random rock-fill materials contained rock not larger than 18 in., no vegetation, not more than 1% wood fragments or other combustible materials, and not more than 12% material passing the 200-mesh sieve. Typical gradation curves for the random gravel material placed in the embankment are also shown in fig. 11. A total of 6,529,000 cu yd of this material was used in the construction of the embankment.

65. The filter material to be placed on the downstream face of the core was specified to be clean, hard, well-graded crushed rock, crushed gravel, or naturally occurring granular material consisting of clean gravel and sand. The gradation limits of the filter material are shown in fig. 12, together with some typical gradation curves of the material used.



Sample No.	Elev	Classification	NatWC	LL	PL	PI	Sta	Offset
LPG 1	710	GRAVEL (GP)	—	—	—	—	17+00	400' DS
LPG 104	825	SANDY GRAVEL (GP)	—	—	—	—	26+00	100' US
LPG 112	795	GRAVEL (GP)	—	—	—	—	23+00	240' DS
LPG 128	845	SANDY GRAVEL (GP)	—	—	—	—	20+00	100' DS
LPG 132	880	SANDY GRAVEL (GP)	—	—	—	—	14+00	50' US

Fig. 11. Typical gradation of impervious core and random gravel materials



Sample No.	Elev	Classification	NatWC	LL	PL	PI	Sta	Offset
LPF 3	772	SILTY SANDY GRAVEL (GM)					15+00	FILTER ZONE
LPF 5	675	SANDY GRAVEL (GW)					23+00	FILTER ZONE
LPF 12	826	SANDY GRAVEL (GW)					19+00	FILTER ZONE
LPF 17	877	SANDY GRAVEL (GW)					23+00	FILTER ZONE
LPF 21	925	SANDY GRAVEL (GW)					19+00	FILTER ZONE

Fig. 12. Typical gradation of filter material, and specified filter limits

Filter materials were obtained by processing materials from upstream gravel borrow sources; a total of 128,550 cu yd was used.

66. The dumped stone for the upstream slope protection and the downstream rock toe was obtained principally from required excavations for the powerhouse foundation and the tailrace channel. The quantity of stone obtained from other sources within the reservoir area and from commercial quarries is not known. A total of 48,000 cu yd of stone was placed on the upstream slope and 28,900 cu yd in the downstream rock toe. Size-distribution records of the slope-protection stones are not available; however, tests were performed at the time of placement to assure that specifications were met.

Embankment

Foundation excavation

67. The exploration trench dug during October 1948 for investigation purposes was incorporated as part of the required foundation excavation. The trench was 40 ft wide at the bottom with side slopes of 1 on 1; the depth ranged from 0 to 35 ft from sta 6+75 to 9+50, and averaged about 30 ft from sta 9+50 to 14+50.

68. Typical sections of the originally proposed excavation and embankment are shown in plates 7 and 8. During August 1950, a major wedge-type slope failure occurred under the upstream slope of the excavation between sta 10+00 and 14+00, which necessitated revision of the design of the embankment in this area. The quantity of material involved in the slide is not known. The slide was thought to be largely if not completely caused by the piling of the excavated materials too close to the edge of the excavation, and although the contractor was warned of the danger of such a slide, no precautionary measures were taken until the slopes showed signs of movement. The movement was first noticed on 14 August 1950 and continued through the remainder of that month. The excavated materials were moved upstream away from the edge of the excavation when the movement was first noted, but the slide continued during this time and a substantial total movement occurred.

69. A detailed description and analysis of the failure and redesign

of the left abutment embankment was published in <u>Supplementary Analysis of Design of the Left Abutment for Lookout Point Dam (Meridian Site), Middle Fork Willamette River</u> (May 1951), and will not be repeated in this report. It was believed that had the method of disposal of the excavated material been prescribed in the contract specifications, the slide might not have occurred. As a result of the failure and subsequent analyses, an additional section of clay talus was removed and a large upstream berm was constructed of random gravel.

<u>Foundation preparation</u>

70. The core trench for the valley section was excavated to firm bedrock as shown by a typical section in plate 6. Specifications required that the bedrock be thoroughly cleaned prior to placement of the impervious core material, and in the event that the bedrock was fractured or irregular, a layer of concrete was to be poured in such a manner as to effectively seal all seams and cracks. However, no concrete coverage was required for this purpose, and none was used. The rest of the embankment foundation area was stripped of all vegetation, roots, and other undesirable materials, scarified to a depth of 6 in., and then thoroughly compacted before placement of fill materials.

<u>Construction procedure</u>

71. <u>Impervious core.</u> As previously stated, the contractor elected to construct the impervious core with clays from the terrace clay borrow areas which were nearer the dam than the silt borrow areas. The clay was spread in 12-in. loose lifts and compacted with four passes of pneumatic-tired rollers loaded to about 85,000 lb and having tire pressures of 85 psi. It was feared that use of the clay material might unduly delay construction because the working season for the clay was relatively shorter than that for the silts. Specifications provided that if the clay was used, the contractor would be required to stockpile this material in such a manner that a ready supply at the proper water content would be available after the time that wet weather ordinarily would have halted construction had these materials not been stockpiled. However, the contractor scheduled the core-placement operation so that stockpiling of the clay material was not required.

72. Specifications required only that the clay be placed and

compacted within such water-content limits that would assure a homogeneous fill without visible voids. The upper water-content limit was that which would permit excavation, traffic, placement, and compaction with the pneumatic-tired rollers without excessive deformation of the embankment material. The lower limit was that point above which a homogeneous fill without visible voids could be obtained. The relation between these specified limiting placement water contents and optimum water content is not known. A minimum per cent of standard density was not specified but field inspection personnel were instructed to obtain at least 90 per cent of standard density and provisions for additional compaction were included in the contract. As will be discussed later, field control samples of the embankment were obtained during construction and records of the water content and density were maintained.

73. Random gravel and filter materials. The random gravel and filter materials were placed in 18-in. loose lifts and compacted with two passes of the pneumatic-tired roller described in paragraph 71. The materials were specified to contain at least the minimum amount of moisture required to obtain the desired compaction as determined by the contracting officer. There were no specified maximum water contents except that the water content would be less than that which would result in an excessive deformation of the fill material from traffic of construction equipment.

74. Field construction control for embankment. A field control program was conducted for the impervious clay core material throughout construction of the embankment to determine the water content and density that were being obtained. A total of 971 samples, or approximately 1 for each 1190 cu yd of the clay core material placed, were obtained and tested. Available records do not indicate that any of the 971 field tests discussed in the following paragraphs were performed on materials that were subsequently removed and replaced, and personnel associated with construction of the embankment believe that none were.

75. One phase of this program was conducted on the clay core material at sta 20+00, where a total of 333 samples were obtained and tested. Individual test results for this group of samples were made available for this review. The water contents in per cent dry weight were determined for all 333 samples; the field density, together with the standard density

determined from compaction tests on sack samples of material obtained from the area immediately adjacent to the field density samples, was determined for 53 of the samples. Results of these tests are shown in the following tabulation.

	Group of 53 Tests	Group of 280 Tests
Field w in % dry weight		
Range	21.8 to 51.3	25.4 to 56.6
Average	36.8	38.7
Standard optimum w in % dry weight		
Range	27.8 to 44.3	--
Average	35.4	--
Field γ_d in lb/cu ft		
Range	67.2 to 93.4	--
Average	80.8	--
Standard γ_d in lb/cu ft		
Range	72.1 to 94.6	--
Average	83.4	--

It appears from the tabulated data that at sta 20+00 the average field water content (36.8%) was 1.4% wetter than optimum (35.4%) and the average field density (80.8 lb per cu ft) was 97% of the average standard density (83.4 lb per cu ft). The average optimum water content (35.4% of dry weight) determined during this phase of the field program compares favorably with that (36.6% of dry weight) determined by laboratory compaction tests performed prior to construction on five typical specimens of the clay.

76. Tests of the other 280 samples obtained at sta 20+00 indicated an average placement water content of 38.7% of dry weight, which is 3.3% wetter than the average optimum water content indicated by the 53 field compaction tests discussed above.

77. The remaining 638 field construction control samples were obtained from the impervious core throughout the entire length of the embankment. Results of individual tests of this group of samples are not available, but a relation between the per cent of standard density attained versus the total number of samples is shown in the following tabulation and is plotted in fig. 13.

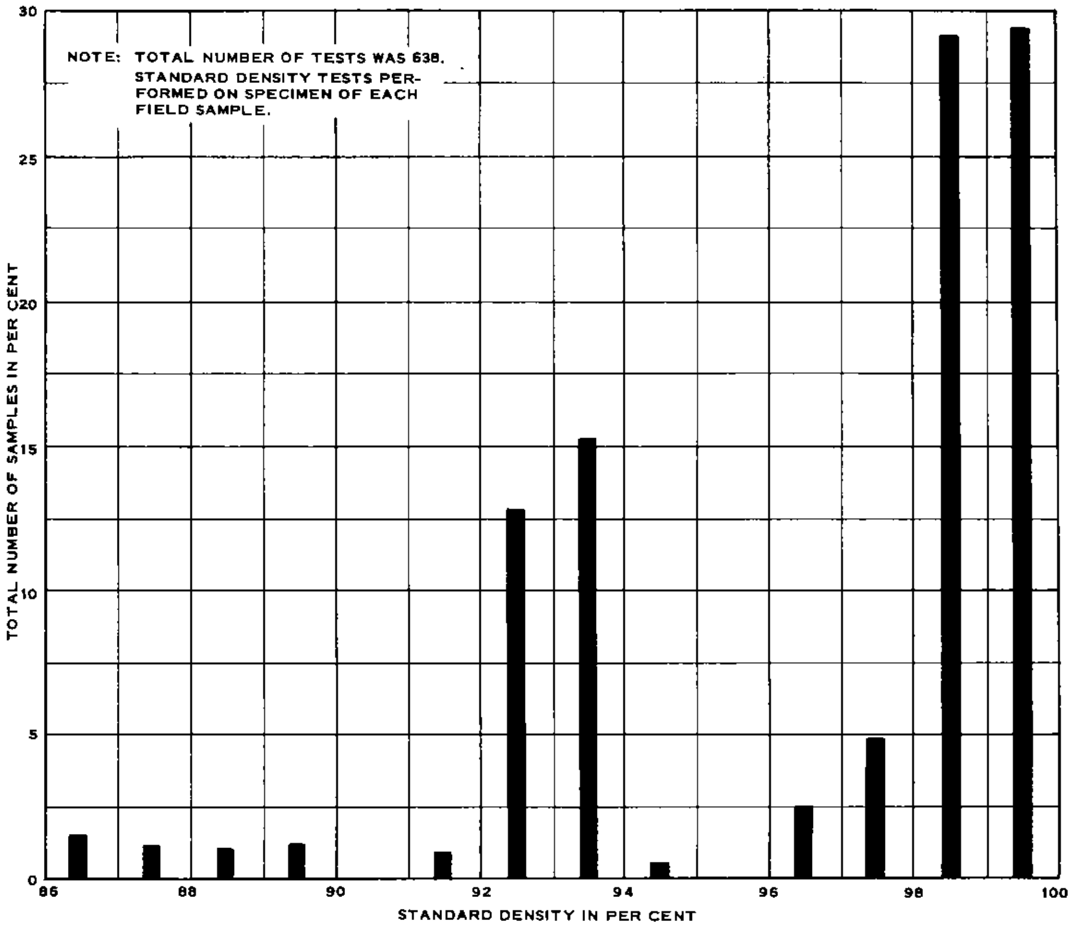


Fig. 13. Field density tests on impervious core materials

Number of tests	638
Per cent of samples having densities greater than 95% of standard density	66
Per cent of samples having densities greater than 90% of standard density	95
Average per cent of standard density for all samples	96

78. Record samples. A total of 368 record samples were taken from the embankment during construction for testing. Of this total, 160 were disturbed sack samples from the random gravel shells, 24 were disturbed sack samples from the filter, 21 were undisturbed core samples of the filter adjacent to the impervious core, 21 were sack samples from the random gravel adjacent to the filter, 108 were undisturbed 6- by 5-in.

core samples from the impervious core, and 34 were undisturbed cubic-foot samples from the impervious core. Tests performed on selected record samples consisted of mechanical analysis, Atterberg limits, void ratio, specific gravity, water content and density, unconfined compression, unconsolidated-undrained triaxial shear, consolidated-drained direct shear, consolidation, and permeability. Some of these tests are discussed in the following paragraphs.

79. Only mechanical analysis tests were performed on the 160 random gravel and 24 filter material samples. Typical grain-size gradation curves for these materials are shown in figs. 11 and 12, respectively. Mechanical analysis tests were also performed on the 21 undisturbed core samples of the filter material adjacent to the impervious core and the 21 disturbed random gravel samples taken immediately adjacent to the filter zone material. Grain-size gradation curves are not shown for these samples, but they are very similar to the curves shown for the impervious core materials and random gravel materials, respectively. Grain-size gradation curves for typical impervious material are also shown in fig. 11.

80. Results of 326 water-content and density tests performed on the undisturbed impervious core record samples are summarized as follows:

Number of tests	326
Water content, % dry weight	
Range	21.5 to 54.2
Average	36.2
Dry density, lb/cu ft	
Range	65.7 to 101.5
Average	81.2

The relations of water contents and densities to the per cent of the total number of samples are shown plotted in fig. 14. The relation of these values to standard optimum water content and density cannot be established since compaction tests were not performed on the record samples; however, it may be noted that the average water content of the record samples was 0.6% less and the average density was 0.4 lb per cu ft greater than that reported as the average for the group of 53 field construction control samples (paragraph 75).

81. Unconfined compression tests were performed on 108 undisturbed record samples of the impervious core materials. Results indicated that

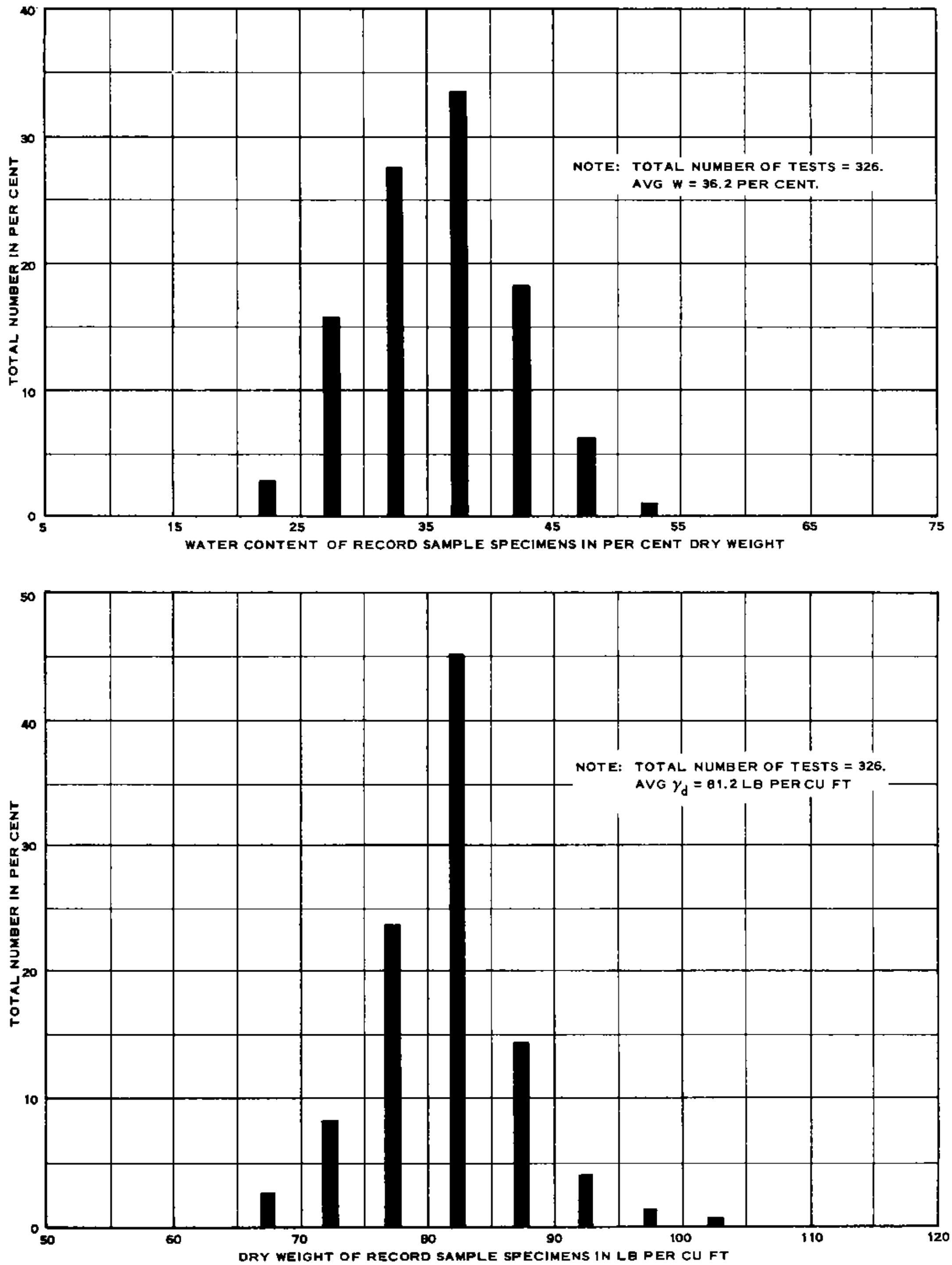
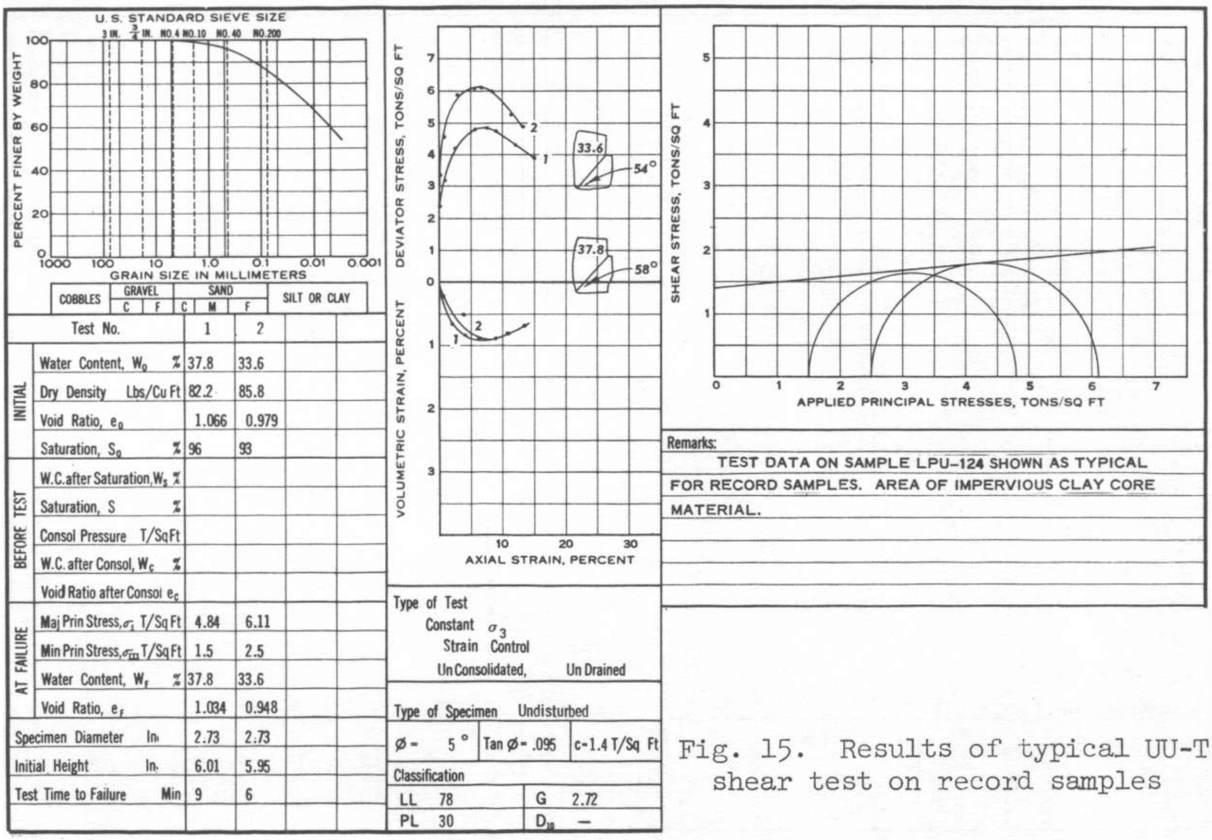


Fig. 14. Water contents and densities of record samples

unconfined shear strengths ranged from 0.40 to 2.88 tons per sq ft, with an average value of 1.11. Some of these tests were run in conjunction with UU-T tests which are discussed in the following paragraph.



Test No.		1	2		
INITIAL	Water Content, W_0 %	37.8	33.6		
	Dry Density Lbs/Cu Ft	82.2	85.8		
	Void Ratio, e_0	1.066	0.979		
	Saturation, S_0 %	96	93		
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/Sq Ft				
	W.C. after Consol, W_c %				
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	4.84	6.11		
	Min Prin Stress, σ_{III} T/Sq Ft	1.5	2.5		
	Water Content, W_f %	37.8	33.6		
	Void Ratio, e_f	1.034	0.948		
Specimen Diameter In.		2.73	2.73		
Initial Height In.		6.01	5.95		
Test Time to Failure Min		9	6		

Type of Test
Constant σ_3
Strain Control
UnConsolidated, Un Drained

Type of Specimen Undisturbed		
ϕ = 5°	Tan ϕ = .095	c = 1.4 T/Sq Ft
Classification		
LL 78		G 2.72
PL 30		D_{10} —

Remarks:
TEST DATA ON SAMPLE LPU-124 SHOWN AS TYPICAL FOR RECORD SAMPLES. AREA OF IMPERVIOUS CLAY CORE MATERIAL.

Fig. 15. Results of typical UU-T shear test on record samples

82. UU-T shear tests were performed on 21 undisturbed record samples of the impervious core. The shear strength indicated by these tests ranged from c = 0.50 to 2.70 tons per sq ft and ϕ = 0.0 to 20.8°, with an average value of c = 1.29 tons per sq ft and ϕ = 8.6°. Results of a typical test are shown in fig. 15; the range of shear strength curves is shown plotted, together with the selected design shear strength of c = 0.30 ton per sq ft and ϕ = 24.2° for the clay material, in fig. 16. However, it was assumed during the design that the impervious core would be constructed of silts and the shear strength of silt (c = 0.0 ton per sq ft and ϕ = 31.0°) was actually used in the analysis; this curve is also shown in fig. 16.

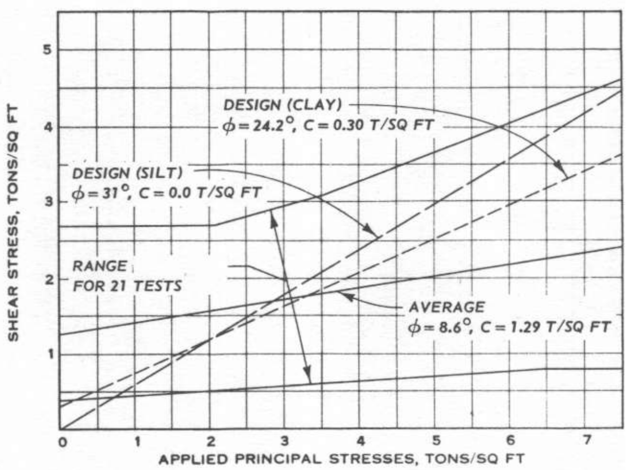
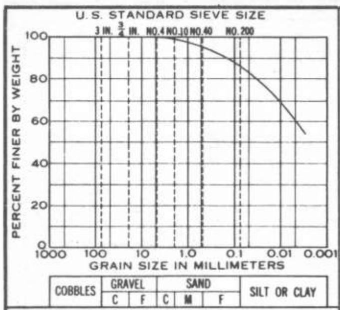
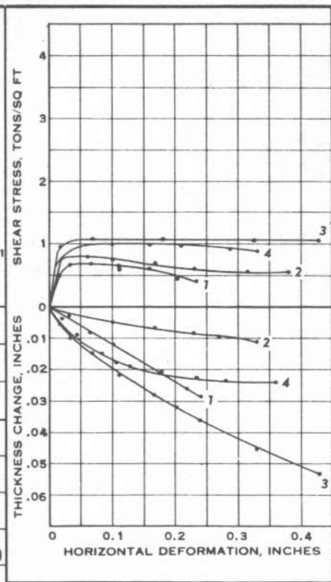
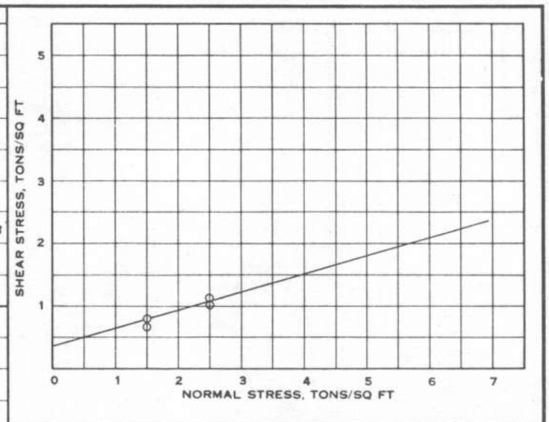


Fig. 16. Range and average of UU-T shear strength curves for record samples







Classification Clay (CH)				
LL 78	G 2.72			
PL 30	D_{10} —			
Test No.	1	2	3	4
Initial Water Content, W_0 %	36.7	33.9	36.0	37.5
Dry Density Lbs/Cu Ft	81.1	85.6	84.7	80.9
Void Ratio, e_0	1.096	0.985	1.006	1.099
Saturation, S_0 %	88	94	98	93
Duration of Test, hrs	5.25	5.00	7.50	5.25
WC after Test, W_f %	39.4	36.9	35.8	37.8
Normal Stress, σ T/Sq Ft	1.5	1.5	2.5	2.5
Max. Shear Stress, τ T/Sq Ft	0.70	0.82	1.11	1.01

Type of Test	Type of Specimen
Strain – Control	Undisturbed
Consolidated	3 & 4 in. square
Drained	3/4 & 1 in. thickness

SHEAR VALUES	ϕ	TAN ϕ	C, T/SQ FT
Maximum	16°	0.29	0.34
Ultimate	—	—	—

Remarks:
TEST DATA ON SAMPLE LPU-124 SHOWN AS TYPICAL FOR RECORD SAMPLES. AREA OF IMPERVIOUS CLAY CORE MATERIAL.

Fig. 17. Results of typical CD-D shear tests on record samples

83. CD-D shear tests were performed on 14 undisturbed record samples. The shear strength values indicated by these tests ranged from c = 0.0 to 0.80 ton per sq ft and ϕ = 10.2 to 34.2°, with an average value of c = 0.28 ton per sq ft and ϕ = 25.0°. Results of a typical test are shown in fig. 17; the range of the shear strength curves for the CD-D tests is shown in fig. 18, together with the selected design shear strength curves for both clay and silt. It may be noted that the average strength for the record samples equals the selected design shear strength for the clay.

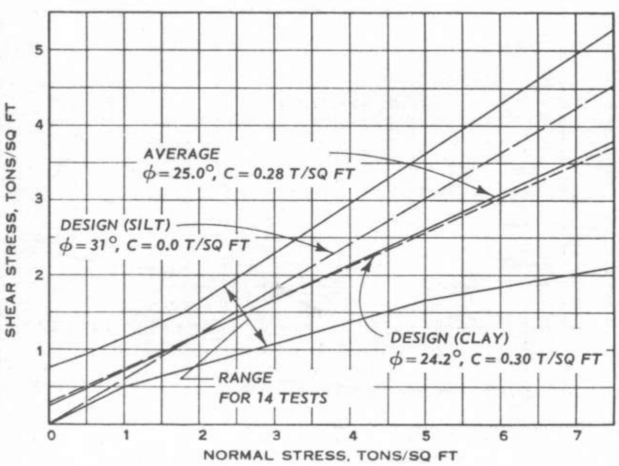


Fig. 18. Range and average of CD-D shear strength curves for record samples

84. Table 3 shows a summary of results of all tests performed on the undisturbed impervious core samples, except the unconfined

Table 3

Summary of Test Data for Record Samples of Impervious Core Material

BORING	SAMPLE	SAMPLE EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS 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 X 10^{-9}	CONSOLIDATION DATA P_0 TONS/SQ FT	P_c* TONS/SQ FT	C_c	t_{50}* MIN	LOCATION STATION	OFFSET
LPU-1		690.0	ML	0	17	83		41	28	2.75	31.5				1.19	78.4	31.5	30.9		UU-T	1.4x3.0				0.60	20.8							19+00	℄
				0	31	69					23	84.8															0.875	220		1.8		2.3		
-2		699.5	ML	1	28	71		45	28	2.76	32				1.14	80.4	32.5	32.4	79	UU-T	2.8x6.5				0.75	10.2							18+00	40 US
											27	84.2															0.860	330		3.3		4.3		
-3		709.5	ML	0	45	55		35	25	2.79	28				0.85	94.2	27.7	27.7	91	UU-T	2.8x6.5				0.60	13.0							17+00	60 DS
											29	94.0															0.745	98		4.2		5.0		
-4		721.5	ML	0	47	53		32	27	2.83	18				0.83	95.8	18.4	24.6	100	CD-D	4x4x1				0.30	33.8							16+00	℄
-5		727.5	ML	0	39	61		46	28	2.78	22				0.73	100.4	22.2	24.9	100	CD-D	3x3x0.75				0.25	33.0							17+00	55 US
-15		785.5	CH	0	12	88		67	31	2.75	26				1.14	80.1	26.4	33.2	64	CD-D	3x3x0.75				0.21	26.6							12+00	℄
											30				0.96	87.2	30.1	29.9	86	UU-T	--				2.15	12.4								
				0	10	90					28	79.5															1.010	1090		1.8		0.2		
-27		824.5	MH	w/weak tuff				73	38		28				1.07	82.6	28.4	28.4	73	UU-T	2.7x6.1				0.95	18.8							8+00	℄
											28				1.07	82.6	27.8	30.0	71	CD-D	4x4x1				0.40	30.6								
											33	80.1															1.010			1.5		0.7		
-31		864.5	MH	0	20	80		61	33	2.72	30				1.06	84.8	29.2	29.2	75	UU-T	--				2.2	14.0							8+00	20 US
												81.1															1.050	1930		4.6		2.3		
-33		882.5	MH	w/weak tuff				69	37	2.73	32				1.06	82.6	32.0	32.0	82	UU-T	2.7x6.1				1.8	20.3							8+00	11 DS
											34				0.98	86.3	34.1	35.2	95	CD-D	3x3x0.75				0.7	24.2								
												85.4															---	--		2.0		--		
-36		677.5	MH	0	18	82		81	42	2.73	53				1.51	67.7	53.3	53.3	96	UU-T	2.7x6.2				1.10	1.7							22+00	50 DS
-47		704.5	CH	w/weak rock				--	--	2.75	39				1.13	80.6	38.6	32.1	93	CD-D**	3x3x0.75				0.0	34.2							20+00	40 DS
											31	87.3															0.90	--		4.4		1.4		
-56		724.5	MH	w/weak rock				59	36	2.71	35				1.06	82.0	35.3	35.3	91	UU-T	2.9x6.2				2.7	0.6							20+00	℄
											39	76.5															1.18	--		4.8		2.5		
-62		734.5	CH	w/weak rock				84	35	2.75	36				1.08	82.5	36.2	36.2	92	UU-T	--				1.2	4.6							25+00	50 US
-66		743.5	CH	0	28	72		73	33	2.72	45				1.43	69.6	44.8	44.8	84	UU-T	--				0.50	2.3							20+00	30 DS
											45	73.4															1.13	112		1.7		1.2		
-78		764.5	CH	--	--	--		73	31	2.71	40				1.18	77.8	39.8	39.8	92	UU-T	2.7x6.1				1.2	0							20+00	℄
										2.72	40				1.16	78.8	39.2	36.4	92	CD-D	3x3x0.75				0.0	26.6								
												79.6															1.04	9.7		2.2		5.7		
-88		782.5	CH	w/weak rock				72	33	2.71	34				1.10	80.4	33.2	33.2	86	UU-T	2.8x--				0.75	11.9							20+00	℄
											31	82.9															1.00	--		3.7		1.2		
-92		799.5	CH	0	20	80		73	32	2.71	34				1.12	79.6	34.1	34.1	82	UU-T	2.7x--				1.65	7.4							20+00	℄
											34	82.3															0.99†	--		3.3†		1.7†		
-100		824.5	CH	0	13	87		67	32	2.73	30				1.11	85.6	29.8	29.8	74	UU-T	2.7x6.0				2.40	9.4							20+00	20 DS
											31				1.07	82.3	30.8	34.7	79	CD-D	--				0.20	20.8								
											--	85.7															0.93	84.6		3.0		4.2		
-110		837.5	CH	0	12	88		102	34	2.72	36				1.06	82.4	35.4	35.4	96	UU-T	2.7x6.1				1.20	4.5							20+00	℄
											36				1.05	82.9	35.7	38.2	92	CD-D	--				0.80	10.2								
											--	80.2															1.102††	5.43††		5.8††		4.2††		
-117		867.5	CH	0	15	85		70	28	2.71	34				1.00	84.5	33.4	33.4	90	UU-T	2.7x6.1				1.60	2.2							20+00	℄
											34				1.01	84.2	33.1	33.4	89	CD-D	--				0.20	20.8								
											40	79.9															1.06‡	2.16‡		3.9‡		32.5‡		

* Values of pressure of 4 tons/sq ft unless otherwise noted.
** Average of 2 or more specimens of same sample.
† Values at pressure of 8 tons/sq ft.
†† Values at pressure of 3 tons/sq ft.
‡ Values at pressure of 5 tons/sq ft.

WES FORM NO. 956
OCTOBER 1956

UC–Unconfined Compression T–Triaxial Compression DS–Direct Shear
UU–Unconsolidated Undrained CU–Consolidated Undrained CD–Consolidated Drained

Table 3 (Concluded)

BORING	SAMPLE	SAMPLE EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS 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 $X 10^{-9}$	CONSOLIDATION DATA P_0 TONS/SQ FT	P_C TONS/SQ FT	C_C	t_{50} MIN	LOCATION STATION	OFFSET
LPU		887.5	CH	0	16	84		78	30	--	36				1.02	84.0	35.7	35.7	94	UU-T	2.7x6.0				1.40	5.5							20+00	℄
-124										--	36				1.05	83.1	36.0	37.5	93	CD-D	--				0.34	16.2								
										2.72	38	82.8															0.99‡	2.14‡		3.5‡		38‡		
-132		906.5	MH	0	14	86		75	37	--	40				1.20	78.0	40.1	40.1	92	UU-T	2.7x6.1				0.80	0							20+00	℄
											37				1.13	80.5	37.0	37.5	92	CD-D	--				0.26	24.2								
											--	83.1															0.98	5.22		1.9		13		
-135		921.5	CH	0	9	91		91	31	2.73	36				1.03	83.7	35.4	35.4	93	UU-T	2.7x6.1				0.80	14.4							20+00	℄
											37				1.05	82.4	36.5	40.1	95	CD-D	--				0.05	31.8								
											--	81.7															1.03‡	7.10‡		3.3‡		85†		
-139		904.5	CH	0	7	93		106	35	2.71	40				1.11	80.3	39.8	39.8	98	UU-T	1.9x4.0				0.66	7.4							8+00	℄
											41				1.13	79.2	40.6	41.9	96	CD-D	1.9x4.0				0.20	16.7								

† Values at pressure of 8 tons/sq ft.
‡ Values at pressure of 5 tons/sq ft.

WES FORM NO. 956
OCTOBER 1956

UC – Unconfined Compression
UU – Unconsolidated Undrained
T – Triaxial Compression
CU – Consolidated Undrained
DS – Direct Shear
CD – Consolidated Drained

compression tests. The liquid limit and plasticity indexes are given in table 3 and plotted in fig. 19.

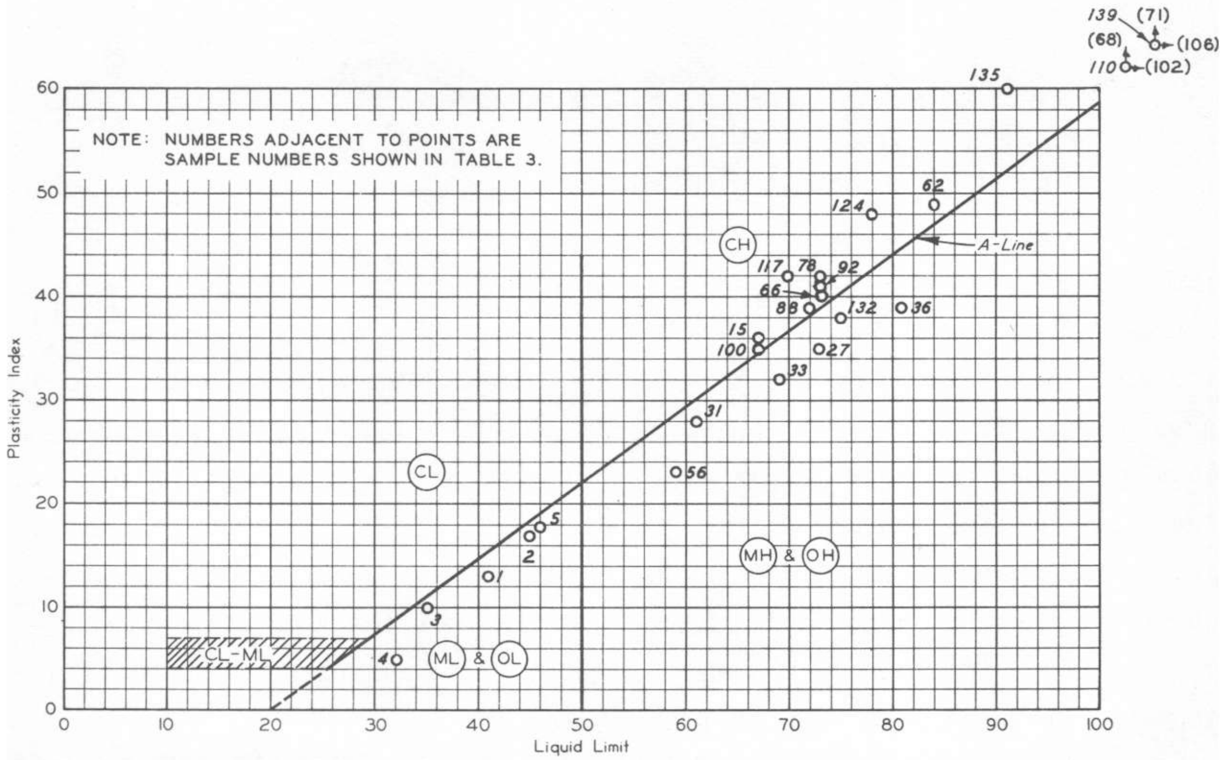


Fig. 19. Plasticity chart for record samples of impervious core materials

PART IV: PERFORMANCE OBSERVATIONS

Piezometer Readings

85. Six piezometers, three at sta 6+50 and three at sta 8+00, were installed in the clay talus materials at the toe of the left abutment embankment after construction to determine if a downstream drainage-collector tunnel would be required to relieve excessive foundation pressures. The piezometers at sta 6+50 are 140 ft downstream from the embankment center line and the ones at sta 8+00 are 185 ft downstream from the embankment center line. Details of the piezometer installations are not available. However, the tips of the piezometers installed at sta 6+50 are approximately 47, 32, and 16 ft below natural ground surface and the tips of those installed at sta 8+00 are approximately 84, 54, and 34 ft below ground surface. The deepest piezometers at both stations are about one or two feet above the foundation bedrock. Piezometer observations made after impoundment of water in the reservoir (April 1954) are shown plotted in fig. 20, together with the reservoir pool hydrograph.

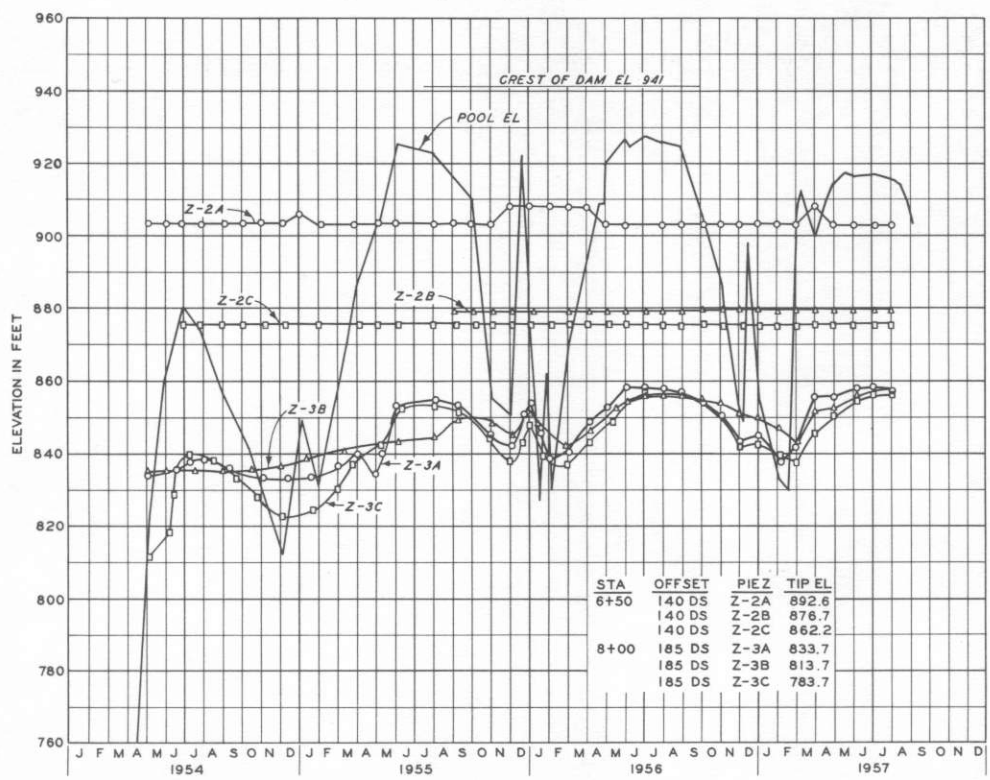


Fig. 20. Piezometer observations and reservoir pool elevations

86. It may be seen that the readings of the piezometers at sta 6+50, where the ground surface is about el 909.0, do not show a corresponding rise and fall with that of the reservoir pool. The slight rise indicated by piezometer Z-2A appears to have been influenced by surface water rather than the reservoir pool.

87. The readings of piezometers at sta 8+00, where the ground surface is at el 868.0, do show a direct relation to the reservoir pool. It may be seen from the curves in fig. 20 that, in general, the piezometer readings rise and fall with the reservoir pool. The maximum head experienced against the embankment at this station to date was about 60 ft (June 1956), at which time the corresponding hydrostatic pressure in the clay talus stratum was at about el 856.0, or 12 ft below the natural ground surface.

88. From the piezometer observations collected to date, it appears that the foundation beneath the downstream portion of the embankment is adequately drained and also that excessive pressures are not developing in the foundation downstream of the dam. Also, since no seepage has been observed at the downstream toe of the embankment, the drainage-collector tunnel is not considered necessary.

Embankment Consolidation

89. Two settlement-measuring devices, one at sta 20+00 and the other at sta 8+00, are located 15 ft downstream from the center line of the embankment. These devices consist of segmented vertical pipes capable of being contracted between welded horizontal crossarms attached at 10-ft intervals (details are shown in fig. 10). Total embankment consolidation is determined by adding the measured consolidation between each crossarm. These measurements are made by an 0.8-in.-OD mandrel, 12 in. long, lowered down the pipe to establish the elevation of each close nipple, the original elevation of which had been established during installation.

90. The time-consolidation curves obtained from observations of these devices are shown in fig. 21, together with the corresponding fill heights. These observations served to determine the amount of overbuild required for the embankment. As a result of the observations, the

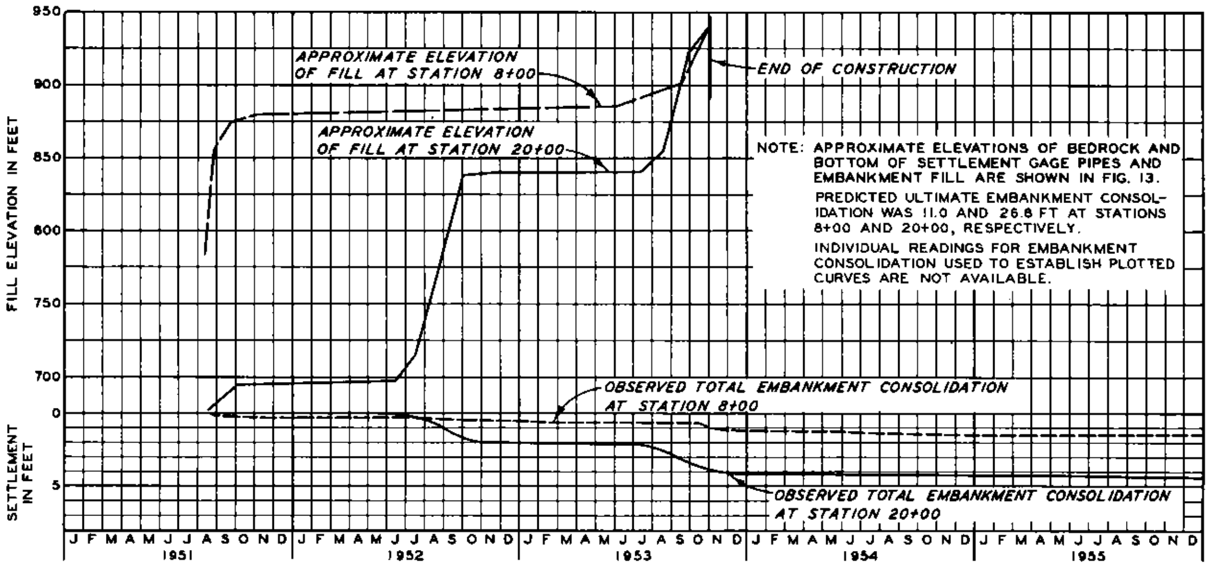


Fig. 21. Elevation of fill and observed embankment consolidation

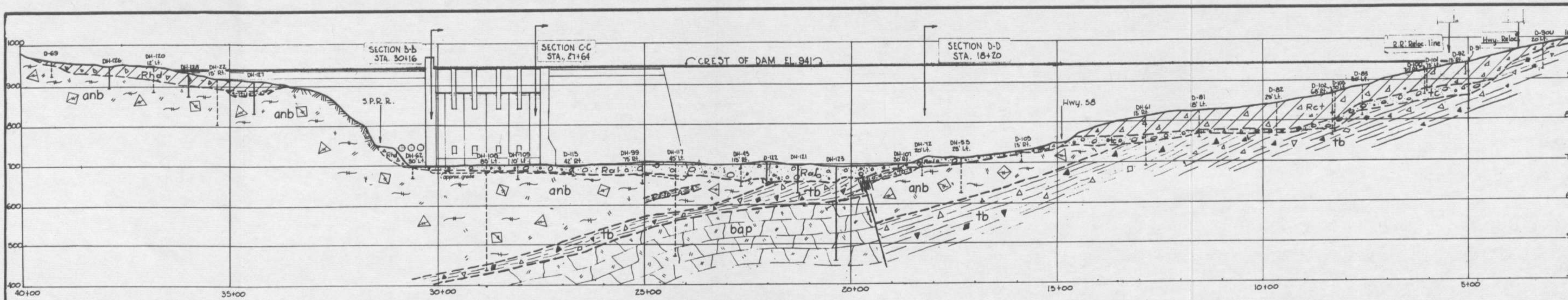
embankment was overbuilt 2 ft. The devices are cemented in the foundation bedrock and it is assumed that all settlement shown occurred within the embankment. It may be seen that at sta 20+00 a total consolidation of 3.8 ft had occurred at the end of embankment construction whereas only about 0.5 ft occurred during the two years following completion of construction. At sta 8+00, a total consolidation of about 1 ft had occurred at the end of construction and about an additional 0.5 ft has occurred since the completion of construction. The total computed amounts of settlement of 11.0 and 26.8 ft at sta 8+00 and 20+00, respectively, appear to be much larger than will actually be experienced in the embankment. However, these amounts were computed as the maximum that would occur in the clay core without consideration of the supporting action of the outside shell. It is believed from past experience that arching action in the relatively narrow core will tend to reduce the consolidation considerably from the computed amount.

Maintenance

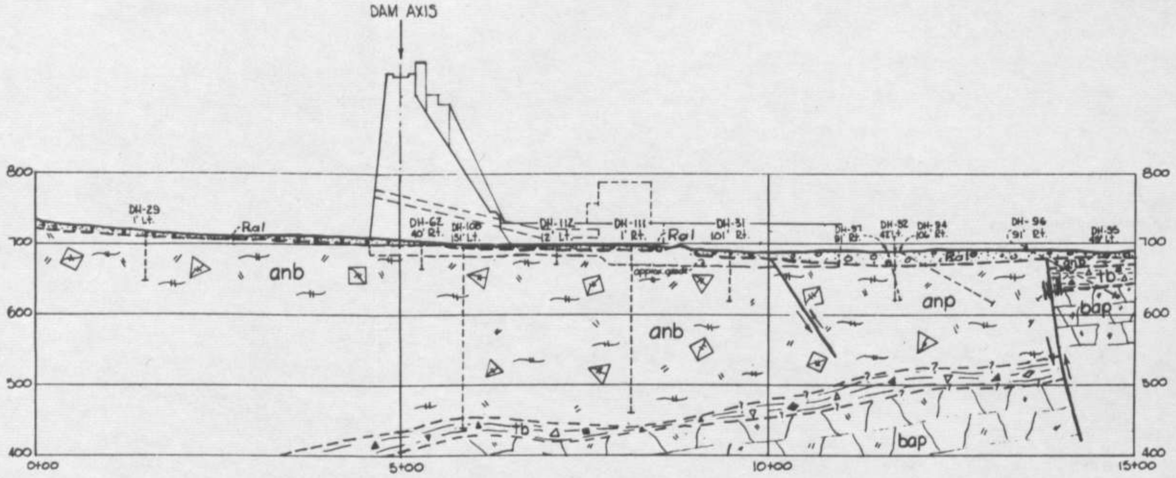
91. The slopes have required no extensive amounts of maintenance since completion of the embankment, and none is anticipated.

GENERAL PLAN OF DAM, AND VICINITY MAP

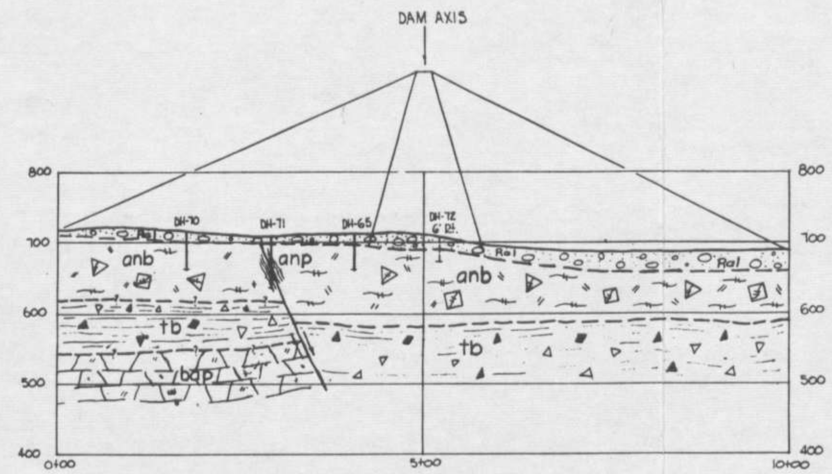
PLATE 1



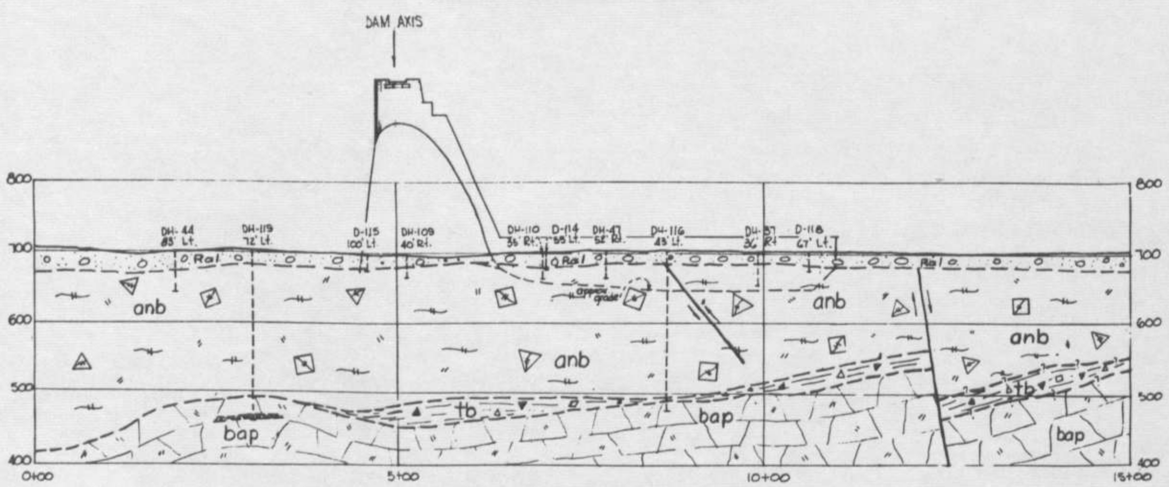
LONGITUDINAL SECTION ALONG AXIS OF DAM



SECTION B-B AT STA. 30+16



SECTION D-D AT STA. 18+20



SECTION C-C AT STA. 27+64

Core drill hole (DH); and churn drill hole (D), on line of section.

Core drill hole (DH); and churn drill hole (D), projected into line of section.

Projected angle core drill hole into line of section.

Geologic contact.

Inferred geologic contact.

Possible fault and gouge zone, showing upthrow and downthrow blocks.

LEGEND

OVERBURDEN

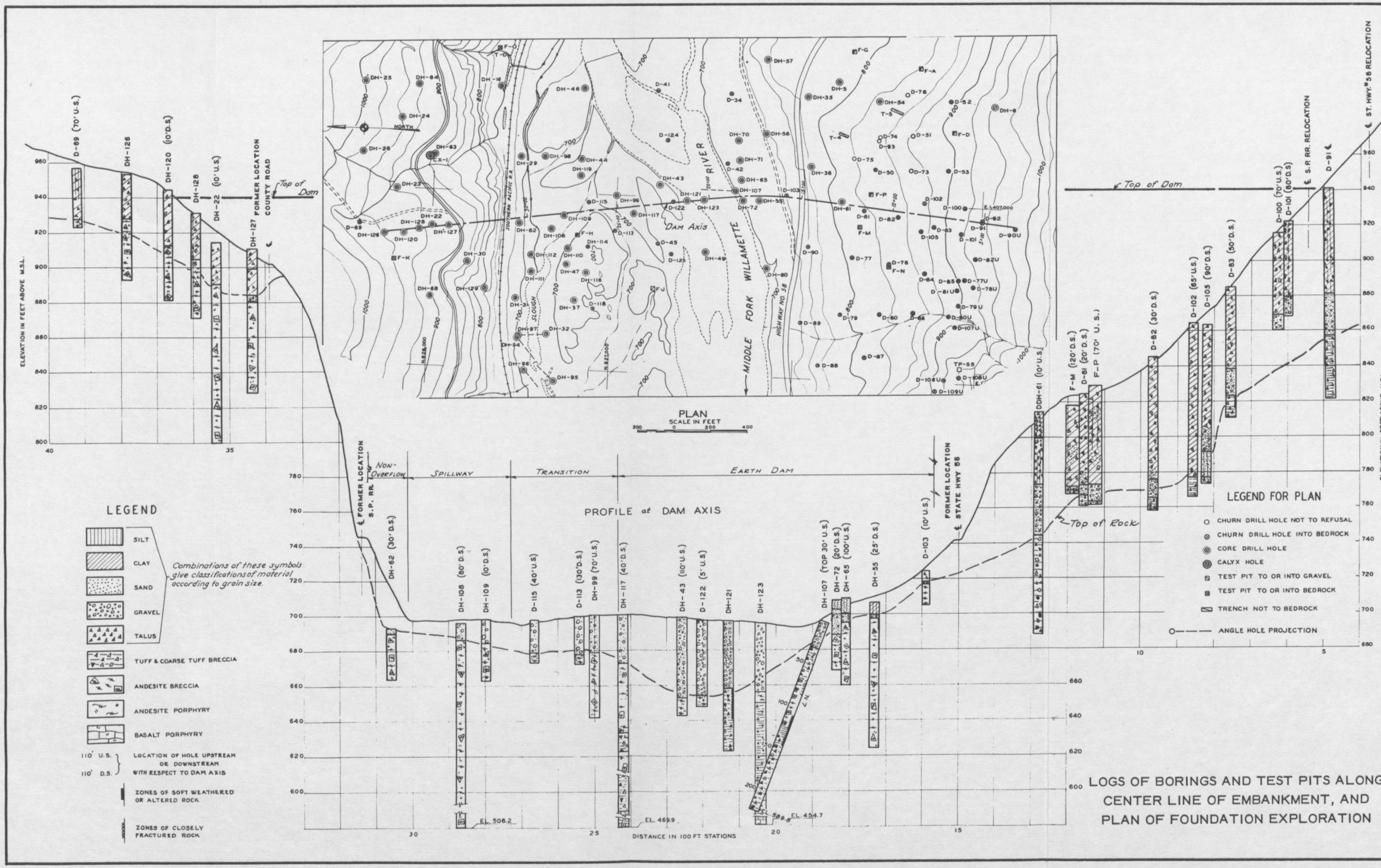
- Rhd — Recent Heterogeneous Deposits
- Rct — Recent Clay Talus
- Ral — Recent Alluvium
- tc — Terrace conglomerate

BEDROCK SERIES

- ltu — Lapilli tuff
- tb — Tuff breccia
- anb — Andesite breccia
- anp — Andesite porphyry
- bap — Basalt porphyry

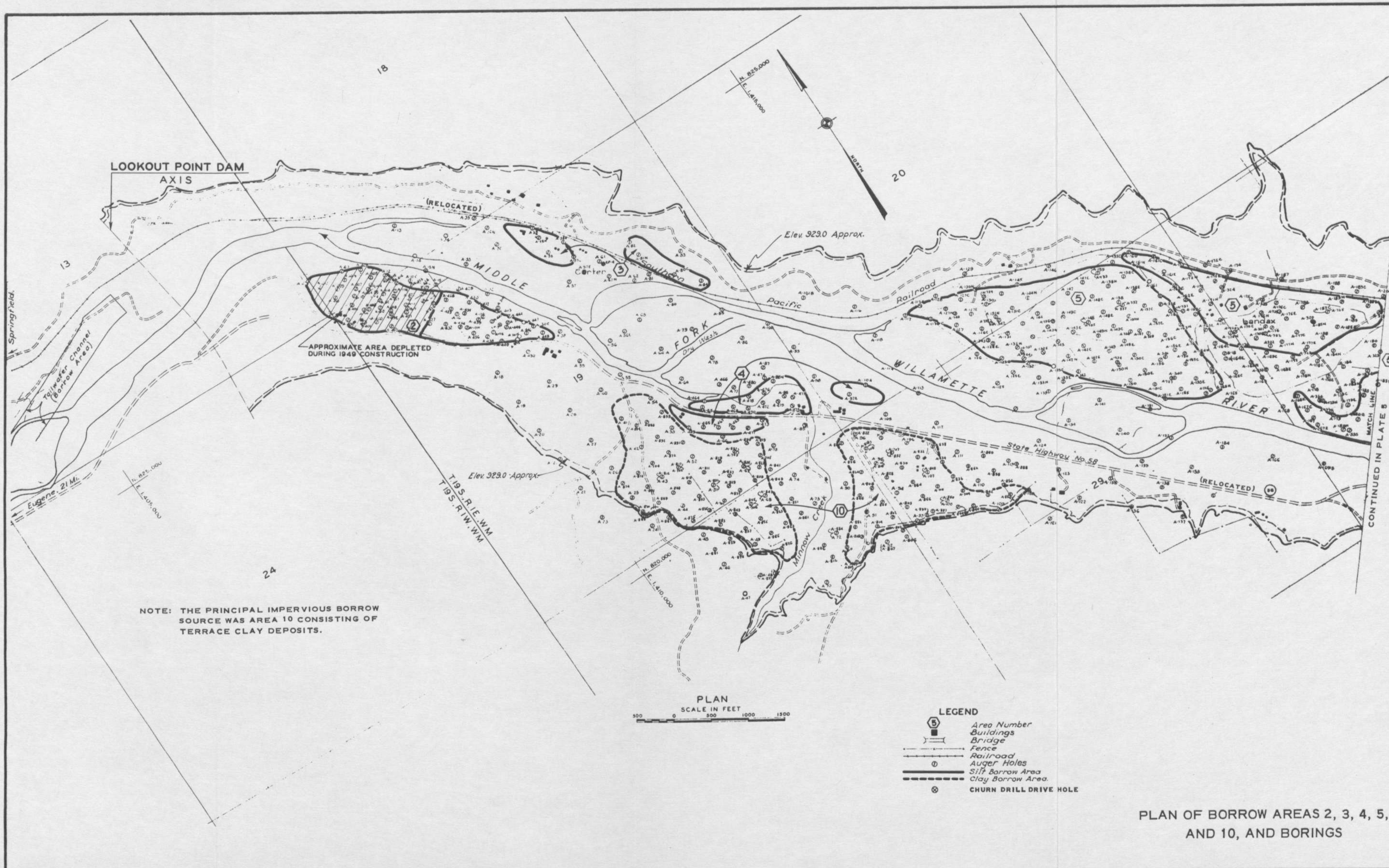
NOTE: FOR BORING LOCATIONS SEE PLATE 3.

GEOLOGIC PROFILES OF EMBANKMENT FOUNDATION

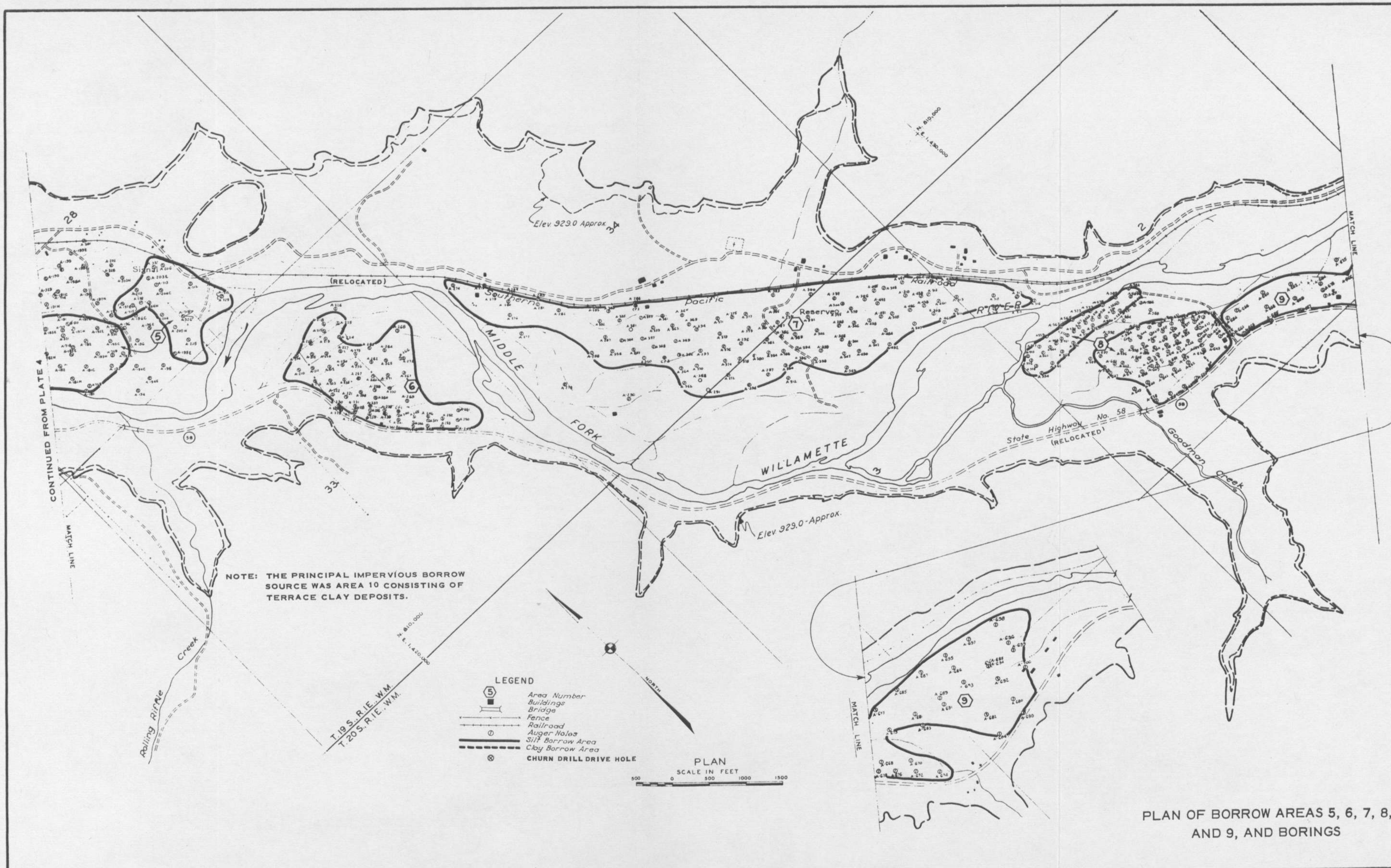


LOGS OF BORINGS AND TEST PITS ALONG CENTER LINE OF EMBANKMENT, AND PLAN OF FOUNDATION EXPLORATION

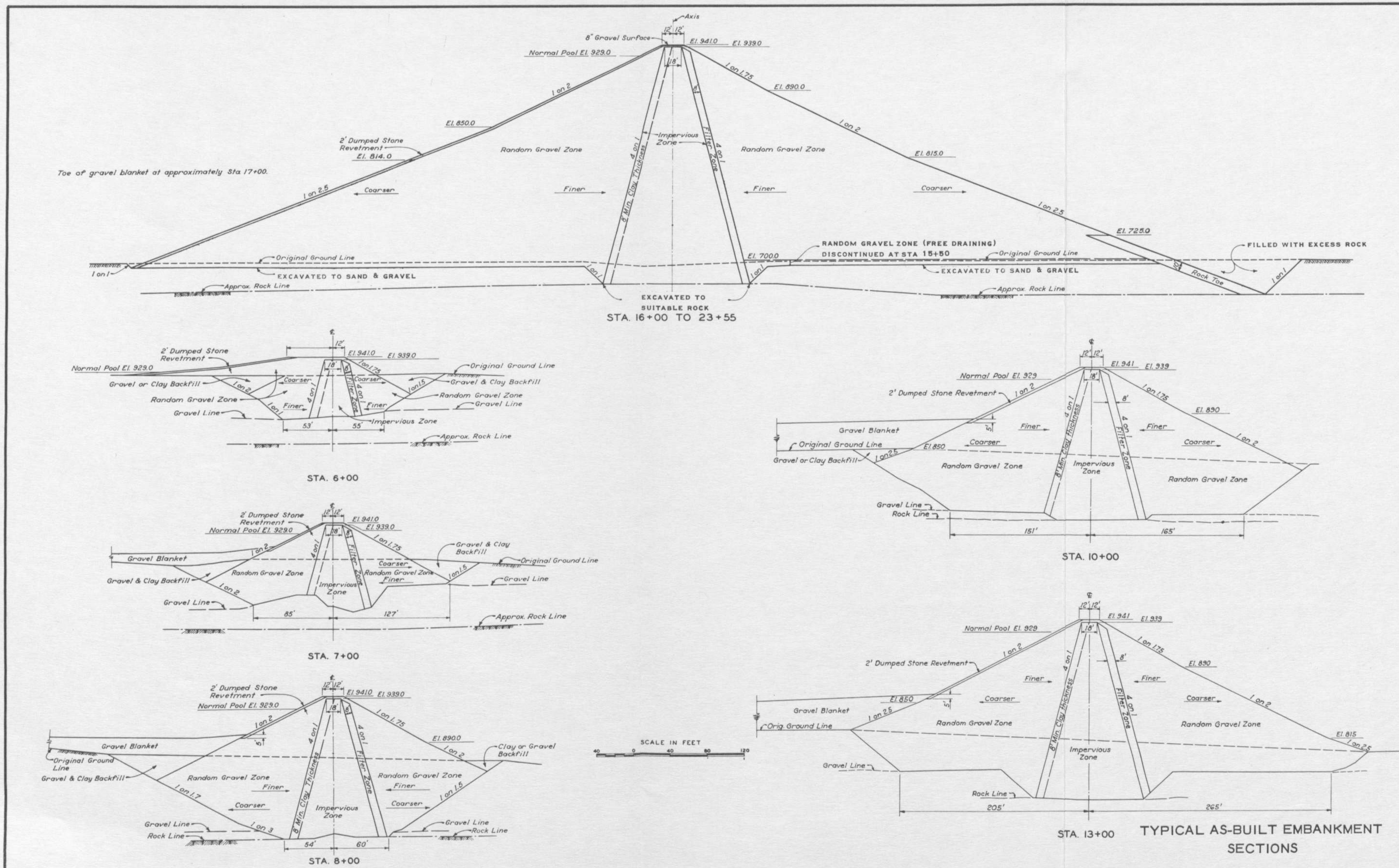
PLATE 3



PLAN OF BORROW AREAS 2, 3, 4, 5, AND 10, AND BORINGS



PLAN OF BORROW AREAS 5, 6, 7, 8, AND 9, AND BORINGS

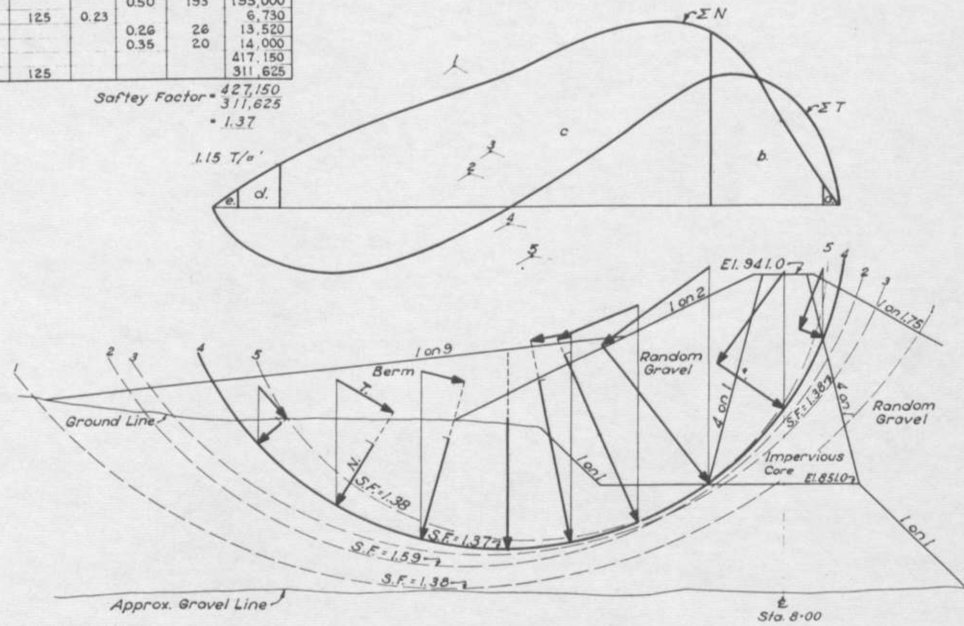


TYPICAL AS-BUILT EMBANKMENT SECTIONS

STABILITY COMPUTATION

Section	Planimeter Reading	Planimeter Const.-ft²	Rel. Wt. lb/cuft	Tan φ	C. T/ft²	Length ft.	Force lbs.
a.	0.03	30²	125	0.85			2,870
b.	2.38	30²	125	0.60			160,650
c.	10.78	30²	125	0.03			36,380
					0.50	193	193,000
d.	0.26	30²	125	0.23			6,730
					0.26	26	13,520
e.					0.35	20	14,000
ΣN+CL							417,150
ΣT	2.77	30²	125				311,625

Saftey Factor = $\frac{427,150}{311,625}$ = 1.37



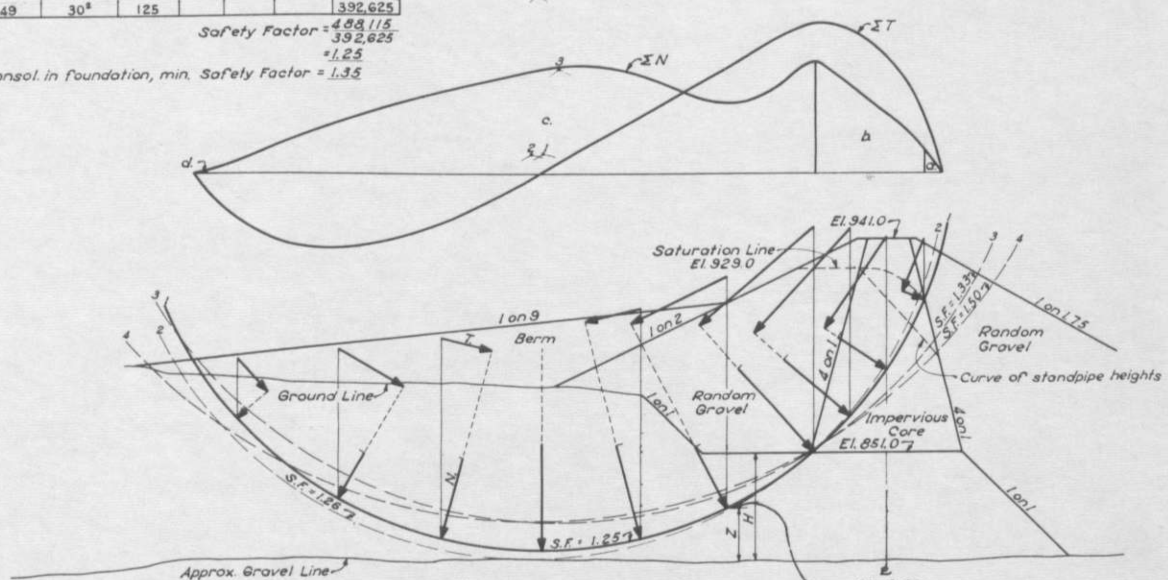
END OF CONSTRUCTION
(NO CONSOLIDATION)
SCALE IN FEET

STABILITY COMPUTATION

Section	Planimeter Reading	Planimeter Const.-ft²	Rel. Wt. lb./cu.ft.	Tan φ	C. T/ft.²	Length ft.	Force lbs.
a.	0.04	30²	125	0.85			3,825
b.	1.5	30²	125	0.60			101,250
c.	8.79	30²	125	0.23			227,440
					0.26	288	150,000
d.					0.35	8	5,600
ΣN+CL							488,115
ΣT	3.49	30²	125				392,625

Safety Factor = $\frac{488,115}{392,625}$ = 1.25

Note:
For 100% consol. in foundation, min. Safety Factor = 1.35



FIRST DRAWDOWN
(PARTIAL CONSOLIDATION)
SCALE IN FEET

NOTE: PER CENT CONSOLIDATION COMPUTED FOR EACH POINT SHOWN ON FAILURE ARC.

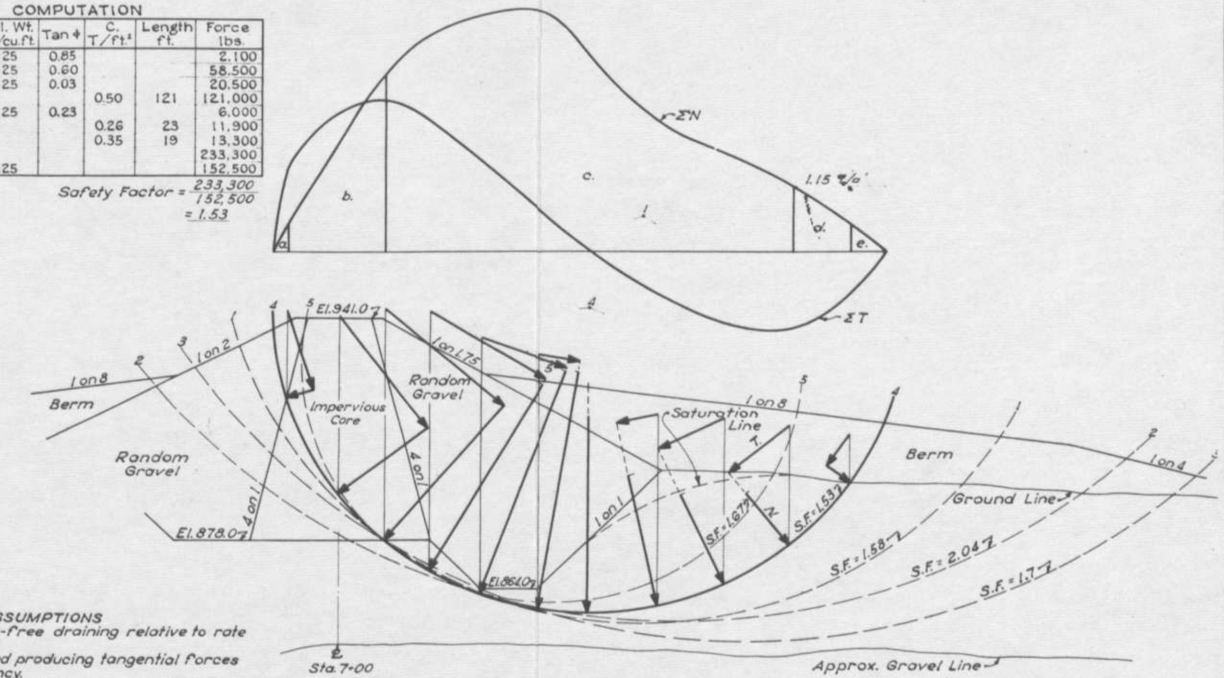
SOIL DATA

Zone	Unit Wt. lb/cuft Moist.	Unit Wt. lb/cuft Sat.	Shearing Strength Tan φ	Shearing Strength C. T/ft.²	Shearing Strength Load T/ft.²
Random	145	148	0.85	0	
Imper.	128	130	0.60	0	
Found.	120	125	0.23	0.26	< 1.15
			0.03	0.50	> 1.15
Berm.	100	110	0	0.35	

STABILITY COMPUTATION

Section	Planimeter Reading	Planimeter Const.-ft²	Rel. Wt. lb./cu.ft.	Tan φ	C. T/ft.²	Length ft.	Force lbs.
a	0.05	20²	125	0.85			2,100
b.	1.95	20²	125	0.60			58,500
c	13.7	20²	125	0.03			20,500
					0.50	121	121,000
d.	0.52	20²	125	0.23			6,000
					0.26	23	11,900
e					0.35	19	13,300
ΣN+CL							233,300
ΣT	3.05	20²	125				152,500

Safety Factor = $\frac{233,300}{152,500}$ = 1.53



BASIC DATA AND ASSUMPTIONS

1. Random gravel zones considered non-free draining relative to rate of drawdown.
2. Saturated weight of material assumed producing tangential forces but normal forces reduced for buoyancy.
3. Average time for consolidation determined from construction schedule.
4. No consolidation considered for End of Construction analysis.
5. Consolidation in foundation considered for Drawdown & Seepage analysis.

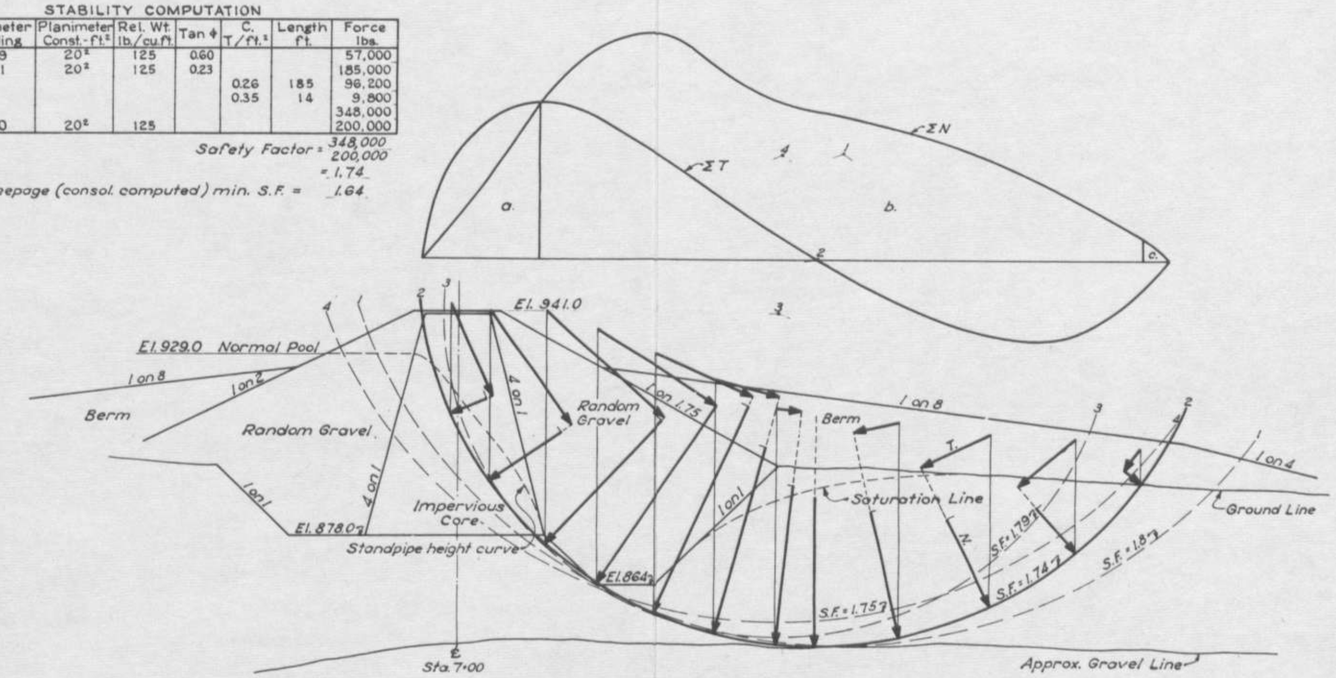
END OF CONSTRUCTION
(NO CONSOLIDATION)
SCALE IN FEET

STABILITY COMPUTATION

Section	Planimeter Reading	Planimeter Const.-ft²	Rel. Wt. lb./cu.ft.	Tan φ	C. T/ft.²	Length ft.	Force lbs.
a	1.9	20²	125	0.60			57,000
b.	16.1	20²	125	0.23			185,000
					0.26	185	96,200
c.					0.35	14	9,800
ΣN+CL							348,000
ΣT	4.0	20²	125				200,000

Safety Factor = $\frac{348,000}{200,000}$ = 1.74

Note:
First steady seepage (consol. computed) min. S.F. = 1.64



STEADY SEEPAGE
(100% CONSOLIDATION)
SCALE IN FEET

TYPICAL STABILITY ANALYSES FOR LEFT ABUTMENT SECTIONS

BASIC DATA AND ASSUMPTIONS

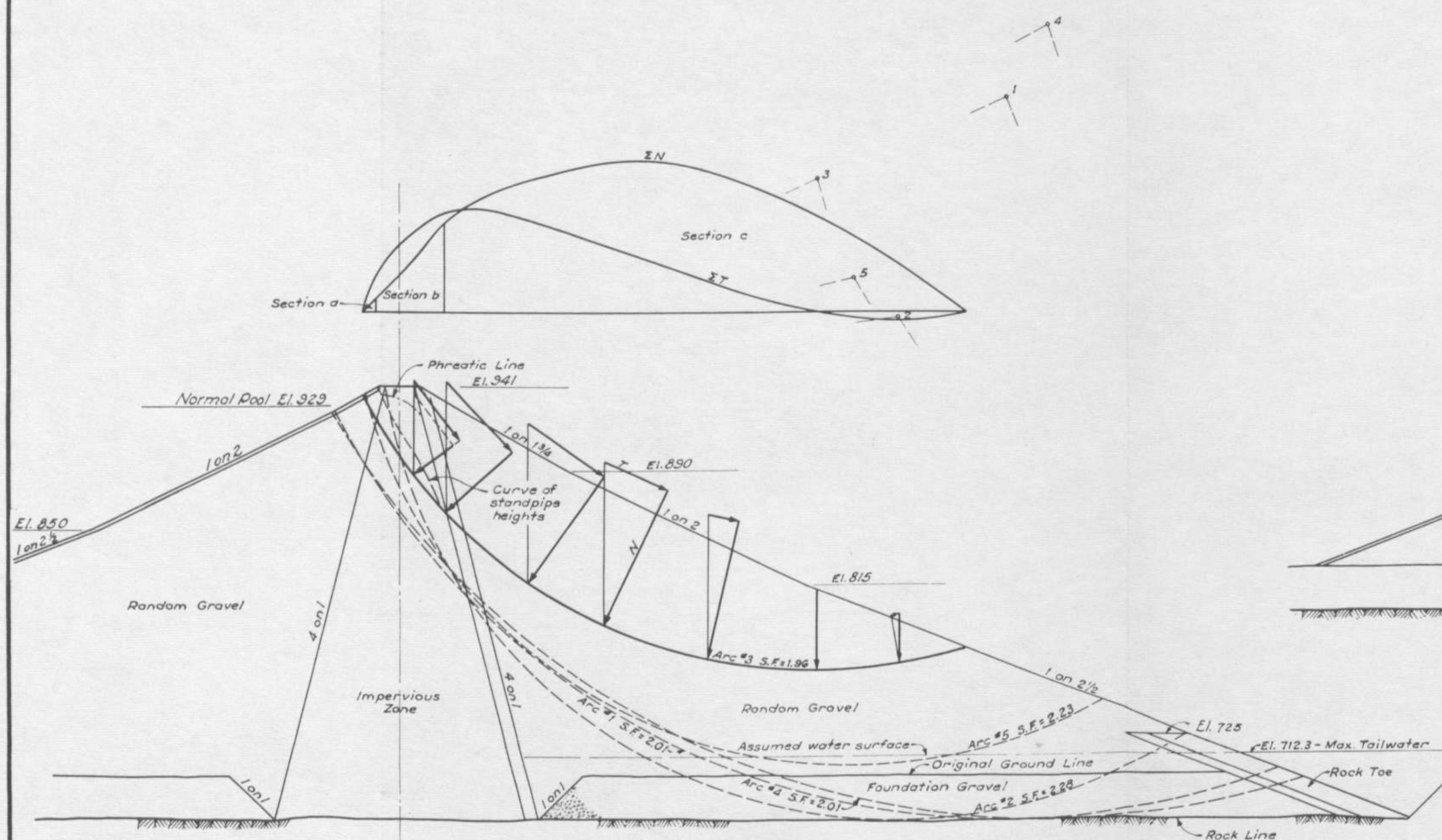
a. GENERAL ASSUMPTIONS

1. SATURATED SOIL WEIGHTS EFFECTIVE IN PRODUCING DRIVING FORCE.
2. NORMAL FORCES ON SLIDING SURFACE REDUCED BY WATER PRESSURE OBTAINED FROM CURVE OF "STANDPIPE HEIGHTS" DERIVED FROM FLOW NET (SEE PLATE 9).

b. SOIL DATA

Zone	Unit Weight		Relative Weight*		Shearing Strengths	
	Moist	Sat.	Moist	Sat.	Tan ϕ	Cohesion T/□'
1 & 3	145	148	1.208	1.233	0.85	0
2	128	130	1.067	1.083	0.60	0
Fdn.	145	148	1.208	1.233	0.85	0

*Relative weights based on 120 lbs. per cu. ft as unity.

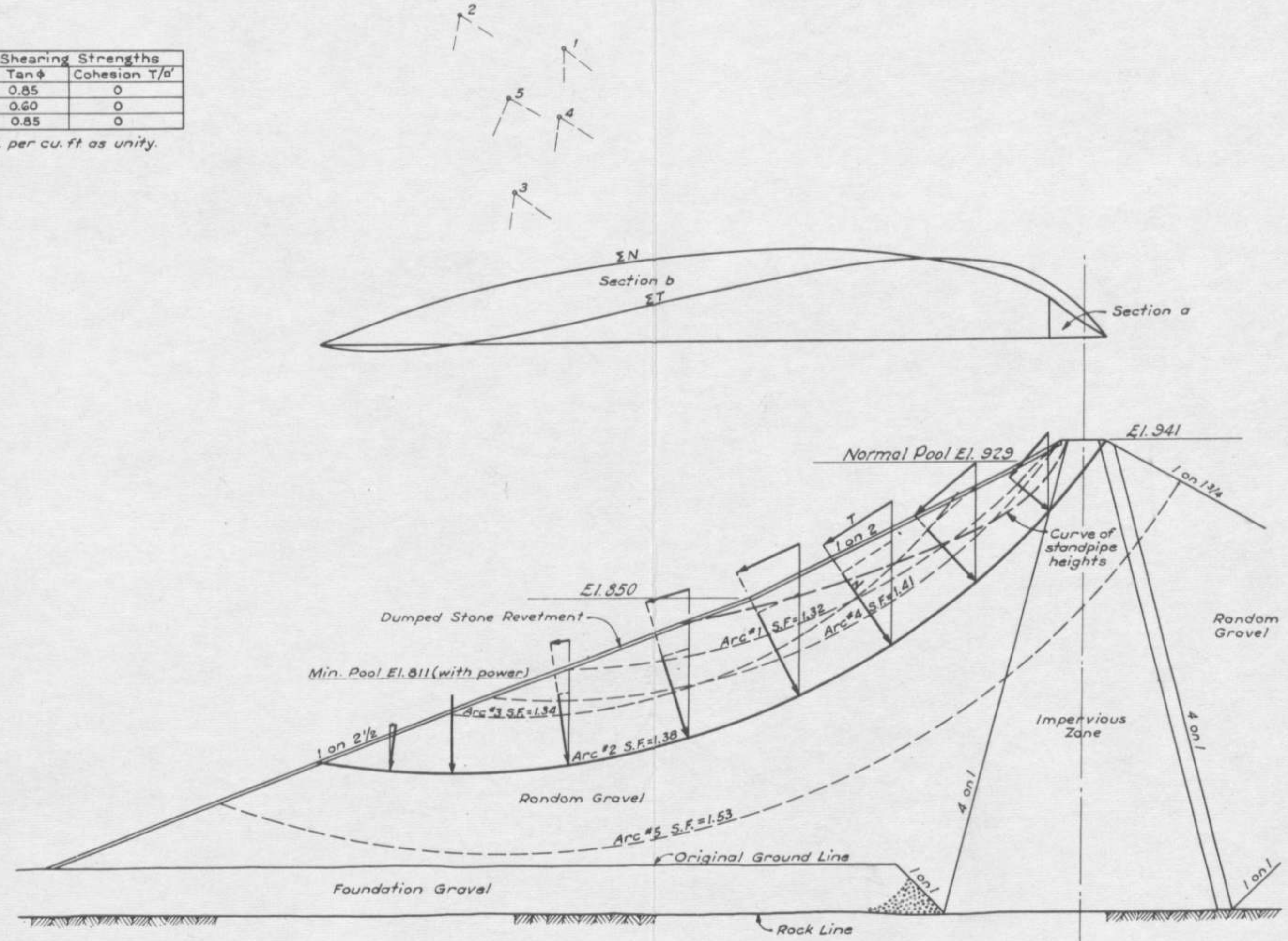


STABILITY ANALYSIS
STEADY SEEPAGE

COMPUTATIONS

ΣN Section	Area	Constant	Unit Wt.	Tan ϕ	Force-lbs.
a	0.04	40^2	120	0.85	6,528
b	0.82	40^2	120	0.60	94,464
c	14.10	40^2	120	0.85	2,301,120
					2,402,112
ΣT	6.38	40^2	120		1,224,960

$$\text{Safety Factor} = \frac{2,402,112}{1,224,960} = 1.96$$



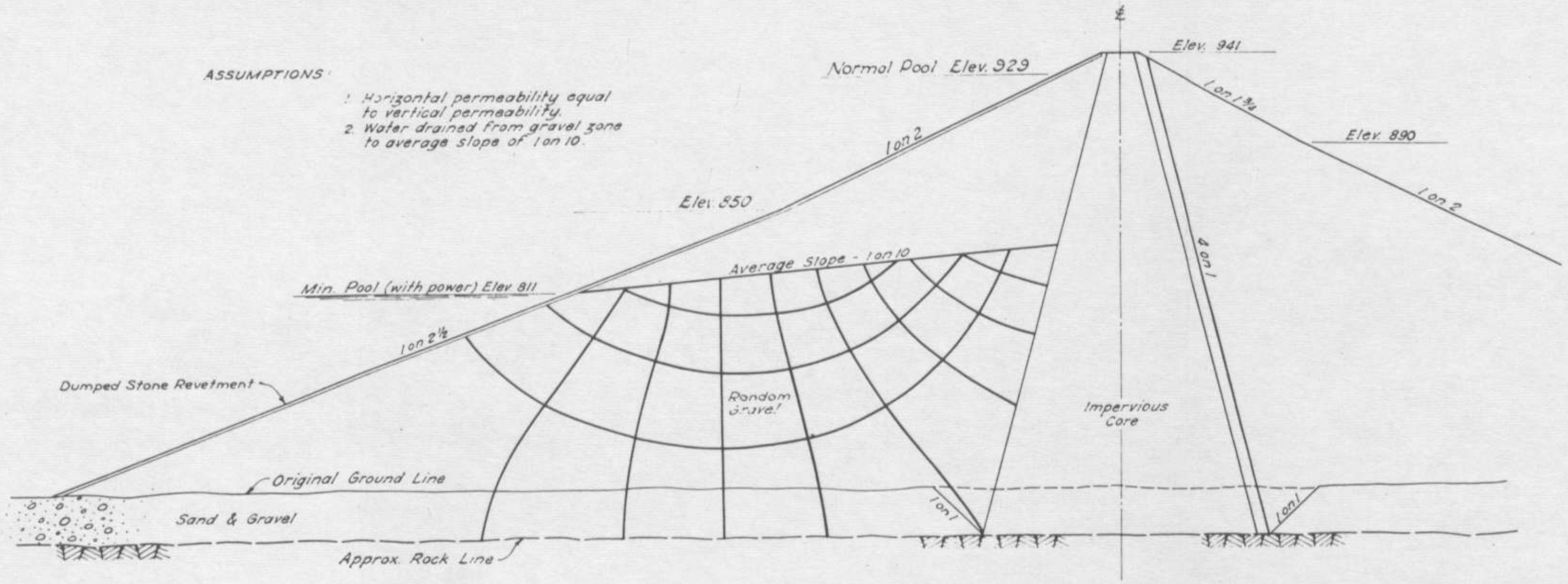
STABILITY ANALYSIS
RAPID DRAWDOWN

COMPUTATIONS

ΣN Section	Area	Constant	Unit Wt.	Tan ϕ	Force-lbs.
a	0.24	40^2	120	0.60	27,700
b	10.14	40^2	120	0.85	1,658,000
					1,685,700
ΣT	6.39	40^2	120		1,225,000

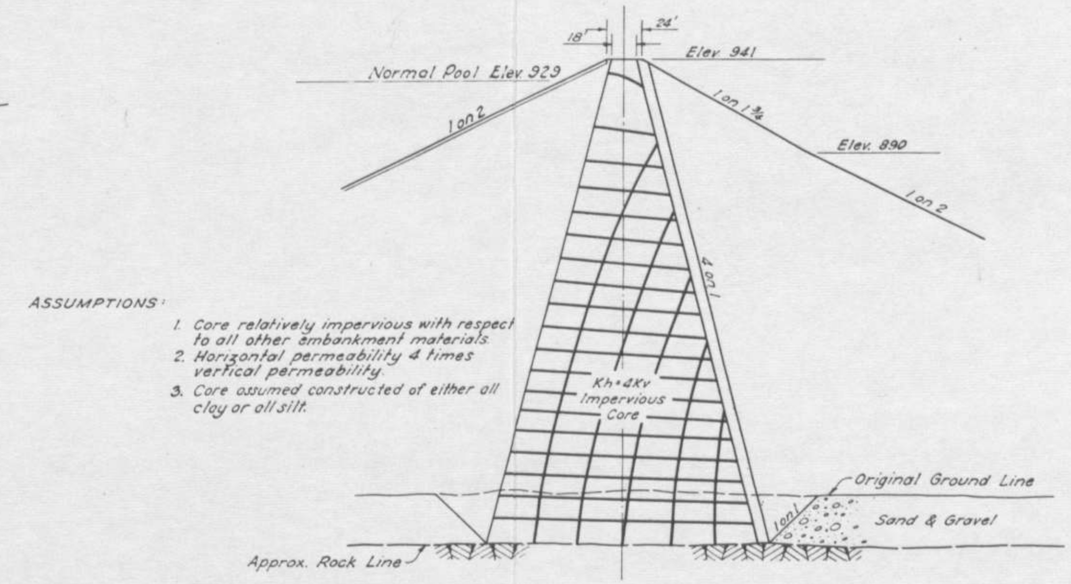
$$\text{Safety Factor} = \frac{1,685,700}{1,225,000} = 1.38$$

TYPICAL STABILITY ANALYSES FOR VALLEY SECTION



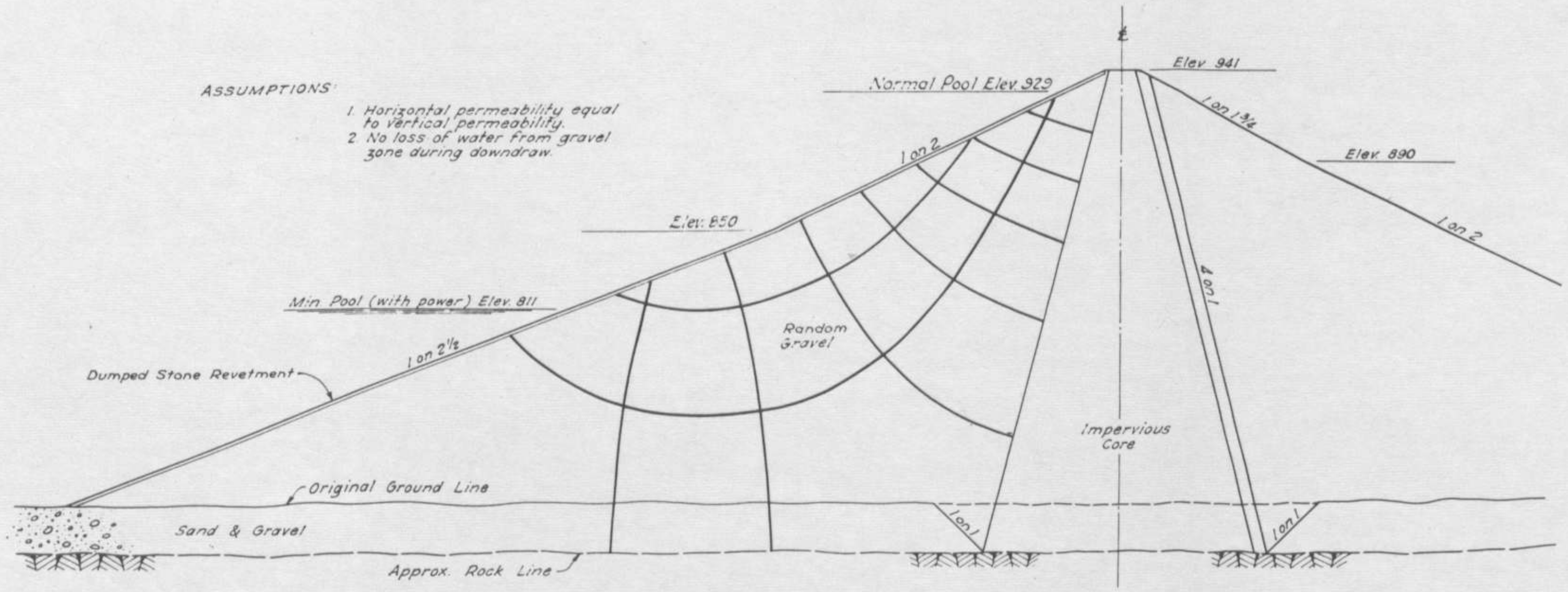
STATION 23+00

RAPID DRAWDOWN FLOW NET



STATION 23+00

STEADY SEEPAGE FLOW NET



STATION 23+00

RAPID DRAWDOWN FLOW NET

TYPICAL FLOW NETS FOR
VALLEY SECTION