U. S. ARMY

CORPS OF ENGINEERS

CIVIL WORKS INVESTIGATION

FLOW SLIDE PHENOMENA

REPORT
ON
INVESTIGATION OF FAILURE OF SHEFFIELD DAM
SANTA BARBARA, CALIFORNIA

OFFICE OF THE DISTRICT ENGINEER

LOS ANGELES, CALIFORNIA

JUNE 1949

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FLOW SLIDE PHENOMENA
INVESTIGATION OF FAILURE OF SHEFFIELD DAM
SANTA BARBARA, CALIFORNIA

I - INTRODUCTION

1. Authorization.-Presented herein are the results of a combined field and laboratory study of the failure of Sheffield dam. This report is part of the Civil Works Investigation as authorized by the Office, Chief of Engineers, in December 1947 and performed by the Los Angeles District in accordance with instructions outlined in a letter from the Office, Chief of Engineers, subject: "Flow Slide Phenomena," dated 28 January 1948.

2. Purpose of the study.-The specific purpose of this investigation is to determine if the failure of Sheffield dam during an earthquake in 1925 was the result of a flow slide. The investigation of the probable reason for the failure of the dam is the basis of this report.

3. History of Sheffield Dam.-The Sheffield reservoir, a distribution reservoir of the Santa Barbara Municipal Water Department, was constructed in the winter of 1917 in a ravine north of the city of Santa Barbara. The plans and specifications were prepared in the engineering offices of Mr. J. B. Lippincott of Los Angeles, California.

Figure 1 shows a general plan of the reservoir and the dam. Figure 2 is a sketch of a typical design section. The dam was an earth embankment 25 feet high and 720 feet long. The embankment was constructed of pit-run material from the reservoir excavation, and was compacted by routing the light construction equipment over the fill. The upstream slope was designed with a 4-foot clay blanket which was extended into the ground 10 feet to serve as a cut-off wall. The clay blanket was to be overlain with a 3-inch concrete facing. It is fairly well established that no formal stripping of the top soil under the embankment was done prior to construction of the dam. No drainage provisions at the downstream toe were included in the design. Due to lack of construction records and field control, the composition of the embankment as built and the degree of compaction obtained is not known.

At 6:00 A.M. on June 29, 1925, a severe earthquake, rated by Dr. Bailey Willis of Stanford University between VIII and IX on the Rossi-Forel scale, occurred at Santa Barbara which caused the central portion of the main dam to move out as though it were hinged at the lower left corner. Of interest is the fact that the plant growth on the downstream side of the displaced mass was undisturbed after the slip. Before the failure, seepage which was percolating either through the foundation or embankment was noted by observers near the downstream toe.

In 1925, a temporary structure was constructed to elevation 647. The foundation was removed to bedrock and replaced with a well-compacted fill. As shown in various photographs[1] the stock piles at the ends of the dam were not removed. Figure 3 shows a section of the reconstructed dam.

In 1936, P.W.A. funds were used to raise the structure to elevation 658. The 1925 embankment was left intact but all other sections were removed and replaced by a compacted fill. The reservoir was dredged and the bottom was paved. Considerable rock was found in the reservoir

[1] Memorandum Re Failure of Sheffield Reservoir in Earthquake of 1925. L. P. Lippincott, Engineering Offices, Los Angeles, California

excavation. All rock under 6 inches in diameter was placed on the earth fill. All rock larger than 6 inches was placed in the drain at the downstream slope. Figures 4, 5, and 6 show the details of the present structure.

II - EXPLORATORY PROGRAM

4. Purpose.-The purpose of the field exploratory work was to obtain data from which field conditions, similar to those existing immediately prior to the earthquake in 1925, could be reconstructed. In planning the detailed exploration it was necessary to place much reliance on information furnished by persons who were associated with the original project. As most of the information was recalled from memory, there are conflicting views on a number of points. A major point on which opinion differs is the exact methods used in the reconstruction of the dam. As has been pointed out in paragraph 2, a temporary structure was built in 1925 to elevation 647, approximately 11 feet below the top of the original embankment. It is fairly well established that the major portion of the original dam, and a considerable portion of the original foundation, were removed and wasted at that time. Assuming this to be the case and supported by data obtained by drilling test holes in the present structure, it was decided that no original undisturbed material could be found in the present structure. Therefore, most of the exploratory work was conducted in areas adjacent to and downstream of the site to obtain, if possible, samples of material similar to that which composed the original foundation and embankment.

5. Extent of exploration.-In February 1949, five auger holes 16 inches in diameter were drilled. The location of all holes is shown on Figure 7. Holes 1 and 2 are located on the center line of the

present dam. They were drilled to bedrock. Holes 3, 4, and 5 were drilled in the valley below the dam where the soil is believed to be similar to that which composed a portion of the foundation of the original structure.

6. Sampling.--Bag, moisture, and jar samples were taken at 2-foot intervals and when changes in materials were encountered. Push-tube samples were taken in soft material encountered in holes 2, 3, 4, and 5. These samples were taken by pushing a thin-walled steel tube into the layer at a uniform rate of penetration. The sample tubes had an inside diameter of 5 inches and a cutting edge of 4.96 inches in diameter.

7. Material encountered.--The materials encountered in nearly all of the auger holes consist predominantly of silt, sand, and sandstone with cobbles varying from 3 to 6 inches in diameter. Holes 1 and 2 consist predominantly of clayey sand interspersed with silty sand layers. Sandstones up to 6 inches in diameter were encountered in these holes. A layer of clay was found from 15.6 to 16.0 feet in hole 2. Sandstone which is believed to be bedrock was encountered at a depth of 14 feet in hole 1, and at a depth of 22 feet in hole 2. Holes 3, 4, and 5 consist predominantly of silty sand. Sandstone which is believed to be bedrock was encountered at 6 feet in hole 3, at 4 feet in hole 4, and at 6.5 feet in hole 5. Detailed logs of the holes are shown in figure 8.

8. Ground water.--No ground water was found in holes 1 and 2. Ground water was encountered at 6 feet in hole 3, at 0.6 foot in hole 4, and at 6.5 feet in hole 5. The source of this water was a small spring located about 25 feet upstream from hole 4.

III - LABORATORY TESTING

9. Inspection and visual classification.-In the South Pacific Division Laboratory, the undisturbed samples were extruded from the tube, classified, and representative samples were selected for detailed testing.

10. Classification tests.-Atterberg limits and specific gravity were determined by the usual methods. The mechanical analyses were made on the plus 200-mesh material, obtained by washing the soil over the 200-mesh sieve.

11. Field density and moisture.-The waxed chunk sample method was used on specimens from the undisturbed samples to obtain field density and moisture values.

12. Triaxial shear.-Two samples, one from hole 3, depth 1.0'-1.4' (silty sand); and one from hole 5, depth 0.4'-1.4' (sandy silt) were tested in a consolidated-undrained condition. One clay sample, depth 15'-16', from hole 2 was tested in an unconsolidated-undrained condition. In the consolidated-undrained test, a cylindrical specimen 1.4 inches in diameter and approximately 3¼ inches in height was allowed access to water and then consolidated fully under one of two lateral pressures; one kg./cm.^{2} or 2 kg./cm.^{2}. Under conditions of no drainage, the specimen was then rapidly loaded to failure. The average time of loading for the two samples was approximately 16 minutes. In the unconsolidated-undrained test, the same size specimen as above was used. The specimen was mounted in the apparatus on an impervious base and the lateral-pressure chamber filled with water under gravity flow. The lateral pressure was then applied as rapidly as possible and the specimen loaded to failure, within approximately 18 minutes. In this test three lateral pressures, 1, 2, and 4 kg./cm.^{2} were used; also, when the testing of each undisturbed

specimen was completed, the specimen was remolded in the apparatus and again tested under unconsolidated-undrained conditions. The detailed shear test data are given on figures 9, 10, and 11.

13. Consolidation.-A consolidation test was made on a sandy silt foundation material from hole 5, depth 0.4'-1.4'. The cylindrical specimen was about 4.4 inches in diameter and one inch in height. With moist cotton batting covering its exposed surface, the soil was allowed to consolidate under a seating load for 24 hours and then consolidated in increments of load starting with 0.1 kg./cm.^{2}. After final dial reading under a consolidating load of 0.2 kg./cm.^{2}, the moist cotton batting was removed and the specimen saturated. Loads were doubled until the characterestics under a load of 3.2 kg./cm.^{2} were established. A vertical dial reading to 0.0001 inch, measured the settlement under each load. A reading of 0.3000 inch on the dial represents the height of one inch. The detailed consolidation test data are given on figures 12, 13, and 14.

14. Test results.-A summary of the test results described above is given in tables 1 and 2.

IV - ANALYSIS AND CONCLUSIONS

15. Analysis of test results.-Based on the field and laboratory investigations, it is believed that the material composing the original dam and its foundation was fairly well compacted silty sand and sandy silt. As the structure had been completed many years prior to the earthquake, it is reasonable to assume that both the embankment and foundation were fully consolidated in 1925. The consolidation test performed on a sample of sandy silt indicates that the time-compression curves do not have the typical double curvature shape exhibited by clay soils. It

is therefore doubtful if the consolidation theory is applicable for the prediction of the time-compression characteristics. However, it is evident that this type of soil undergoes fairly rapid consolidation, but it is not sufficiently free draining to consolidate during rapid application of shearing stress. The lack of any appreciable rebound, as noted on the void ratio-pressure curve, indicates that the natural structure of the material is considerably altered by loading past the preconsolidation pressure.

Based on the consolidation test data, the triaxial shear tests were performed for consolidated-undrained conditions. The results of these tests indicate that shear constants, varying between an angle of internal friction ϕ, equal to 16.5 degrees and zero cohesion, and ϕ equal to 21.5 degrees and zero cohesion, may represent the range in shear strength for the foundation material at the time of failure. A value of $\phi = 19^{\circ}$ was adopted for both the foundation and embankment. The sample of clay obtained from test hole 2, which conceivably might be representative of the original foundation material, was tested in an unconsolidated-undrained condition in both undisturbed and remolded states to determine the probable loss in strength due to remolding. The test results indicate little loss in strength.

16. Physical properties of materials.-The physical properties of ~~the~~ materials, determined for use in the stability analysis of the embankment and foundation, are based on the laboratory analysis of the undisturbed samples from holes 3 and 5, and are shown in the following table:

Soil Type	Density Lb./Cu.Ft. Dry	Field	Saturated	Ø Degree	Cohesion T./Sq.Ft.
Silty sand	86.5	99.0	115.5	19	0

17. Stability analysis of embankment.-A stability analysis of the embankment was made on the basis of the following assumptions: (1) the material composing the embankment and foundation had physical properties as shown in the above table, (2) bedrock existed at a depth of 6 feet below ground surface, (3) the motion of the foundation during the earthquake was normal to the axis, and (5) the acceleration due to the earthquake was equal to 0.1 g.

By the critical circle method, the following safety factors have been determined:

(1) Before earthquake, F.S. = 1.61
(2) During earthquake, F.S. = 0.91

Results are shown on figure 15.

18. Underseepage.-Observers indicate that considerable seepage was noted both at the toe of the downstream slope and in the area below the toe. It is concluded that, in addition to the development of seepage through the embankment, a seepage plane occurred along the surface of the bedrock or along the plane of contact between the embankment and the foundation. Because of root holes in the porous topsoil, it is more likely that a path of seepage developed along the plane of contact between the embankment and the foundation rather than along the top of the bedrock. In

the event of the latter condition, it is questionable whether there would have been a noticeable quantity of seepage below the downstream toe as the 6 feet of soil which overlies the bedrock at that point would have probably balanced out most of the pressure head transferred along the seepage plane. The differential head at the time of failure was 15 feet. If a condition of underseepage, as presented, did exist, it is probable that the embankment was on the verge of failure prior to the earthquake and that the additional shearing stresses induced by the quake resulted in sudden failure.

19. Conclusions and recommendations.-It is concluded from the investigations that the cause of failure of the Sheffield dam can not be attributed to a flow slide. The fact that the portion of the embankment which was displaced remained intact during the slide rules out the possibility of liquefaction of the embankment itself. The fact that the foundation material was probably well graded and fairly compact and that drainage of the material could occur appears to eliminate the possibility of liquefaction of the foundation. It appears more likely that failure occurred along a shear plane such as shown on figure 15. It is probable that a plane of seepage developed between the embankment and the foundation and the pressure head transmitted along this plane reduced the effective shearing strength of the foundation and embankment material to values below those used in this report. The resulting factor of safety was probably close to unity prior to failure. The additional shearing stresses induced by the earthquake would then be sufficient to cause sudden failure. The removal of the porous top soil under the embankment to provide a firm surface on which a well compacted fill could have been

constructed, and the inclusion of a downstream toe drain would probably have obviated the undesirable features of the design.

It is recommended that no further study be made of the Sheffield dam failure.

TABLES

Table 1 - Summary of Classification Tests

PART A - DISTURBED SAMPLES — Sheffield Dam

Hole No.	Depth Sampled	Soil Type	Grain Size Distribution: Max. Size	Gravel	Sand	Silt or Clay	Field Moisture	L.L.	P.I.	Field remarks
1	0-2	Clayey sand					7.2			Elev. 656 feet
	2-4	Clayey sand					9.3			
	4-6	Clayey sand					11.6			
	6-8	Clayey Silty sand					11.4			
	8-10	Clayey silty sand					14.8			
	10-12	Clayey silty sand					14.0			
	12-14	Sand					8.4			Bedrock 14'
2	0-2	Clayey sandy gravel					11.6			Elev. 658 feet
	2-4	Clayey sandy gravel	3"	36	29	35	11.4	25	9	
	4-6	Clayey sandy gravel					12.8			
	6-8	Sandy silt					11.0			
	8-10	Sandy silt	1/2"	1	51	48	12.1	21	4	
	10-12	Sandy silt					12.0			
	12-15	Sandy silt					11.0			
	15-17	Sandy clay	2"	2	49	49	25.6	23	7	
	17-19	Sandy clay	3"	12	47	44	14.5	22	8	
	19-21	Sandy silt	1"	3	50	47	13.4		NP	
	21-22	Sand					11.5			Bedrock 22'
3	0-2	Clayey sand					13.2			Elev. 611 feet
	2-4	Silty sand					12.2			G.W. at 6'
	4-6	Sand					14.9			Bedrock at 6'
4	0-2	Silty sand					31.3			Elev. 614.6
	2-4	Clayey sand					18.2			G.W. at 0.6'
										Bedrock at 4'
5	0-2	Silty sand					14.6			Elev. 616
	2-4	Silty sand					14.4			
	4-6	Silty sand					17.2			G.W. at 6.5'
	6-6.5	Sandstone					12.6			Bedrock at 6.5'

NOTE: Most classifications are taken from field logs

Table 1 (Continued)
Summary of Classification Tests

PART B - UNDISTURBED SAMPLES

Hole No.	Depth Sampled	Soil Type	Grain Size Distribution Max. Size	Gravel	Sand	Silt or Clay
2	15.6-16.0	Clay	#10		12	88
	16.5-18.1	Silty sand	1"	14	47	39
	18.1-18.5	Gr. Sandy clay	1"	17	39	44
	20.0-21.3	Silty sand	#10		64	36
3	0.0-1.4	Sandy silt	#10		52	48
	1.4-3.0	Silty sand	#10		63	37
	3.5-4.5	Sandy silt	#10		59	41
5	0.4-1.4	Sandy silt	#10		56	44
	1.4-3.0	Silty sand	#10		67	33
	4.0-4.5	Sandy silt	1/2"	2	54	44
	4.5-6.0	Silty sand	#10		62	38

Table 2 - Summary of Tests on Undisturbed Samples.

Hole No.	Depth of Sample Feet	Soil Type	Shear: Type of Test	Shear: ϕ Deg.	Shear: C TSF	L.L.	P.I.	S.G.	Field Density: Density pcf	Field Density: Moisture %	Consolidation: Coef. of Consolidation $x10^{-4}$ Ft2/min	Consolidation: Comp Rati
2	15.6-16.0	Clay	Triaxial UU	-	1.25	70	45	2.60	78	36.6		
			Triaxial UU(R)	-	1.25							
	16.5-18.1	Silty Sand					NP					
	18.1-18.5	Gr. Sandy Clay				26	12		108	18.8		
	20.0-21.3	Silty Sand					NP		111	13.9		
3	0.0-1.4	Sandy Silt	Triaxial CU	16½	-		NP	2.58	91	11.7		
	1.4-3.0	Silty Sand					NP		101	12.4		
	3.0-3.5	Silty Sand					NP					
	3.5-4.5	Sandy Silt					NP		112	14.8		
5	0.4-1.4	Sandy Silt	Triaxial CU	21½	-		NP		90	15.1	5.82	0.332
	1.4-3.0	Silty Sand					NP		101	13.8		
	3.0-4.0	Silty Sand										
	4.0-4.5	Sandy Silt				25	3		109	17.7		
	4.5-6.0	Silty Sand				22	6					

UU - Unconsolidated Undrained
CU - Consolidated Undrained
(R)- Remolded

FIGURES

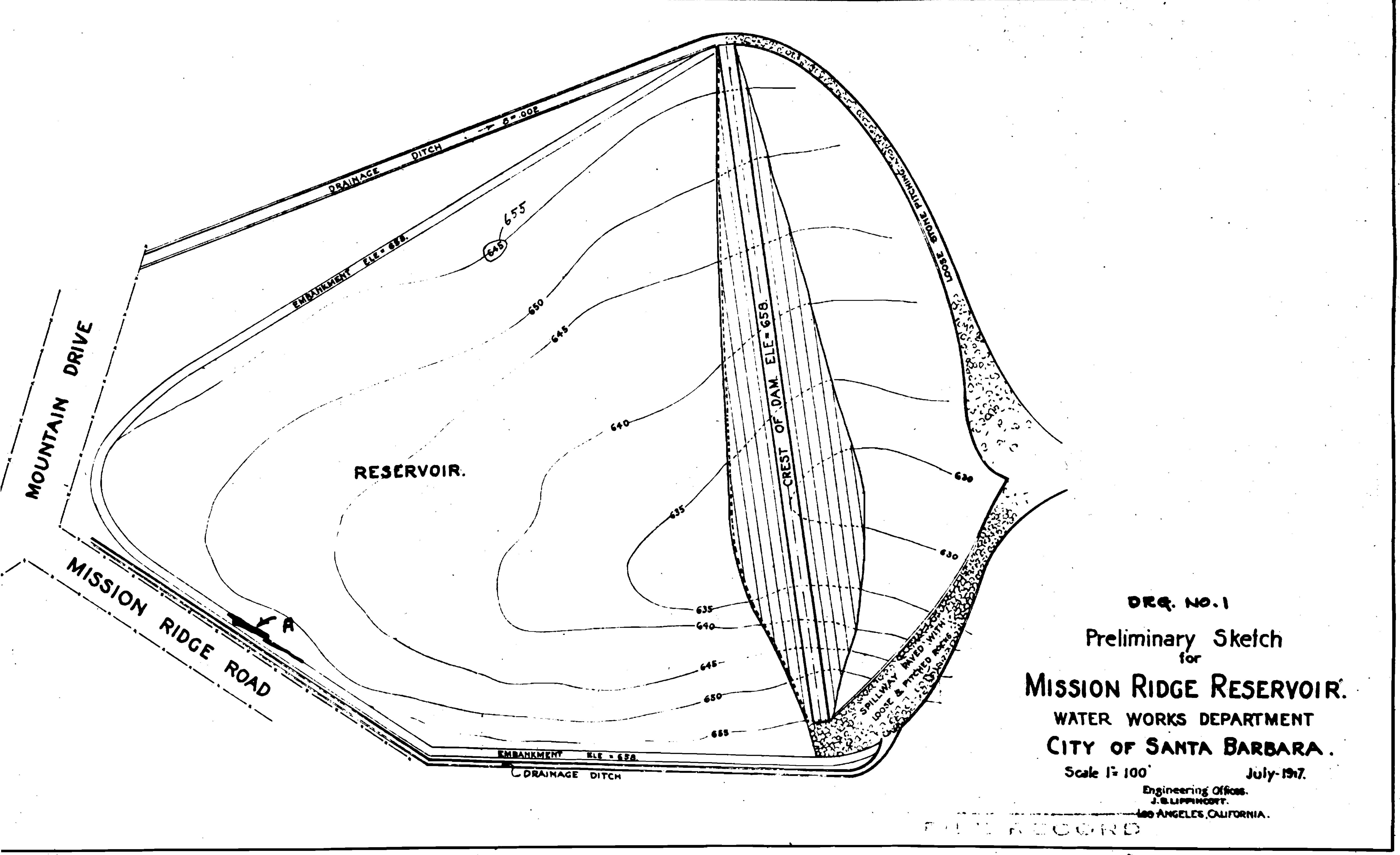
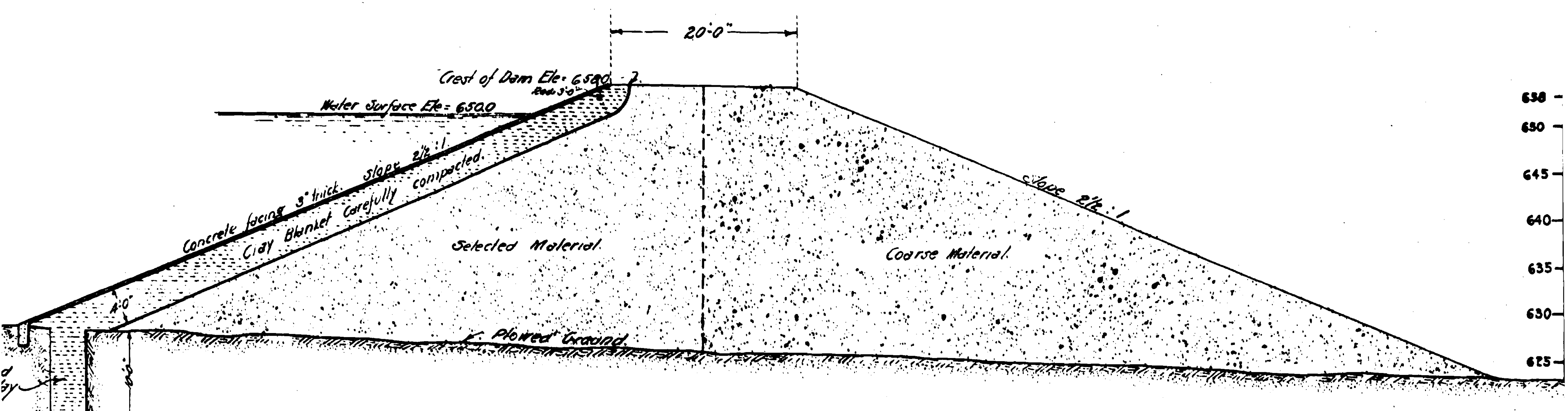
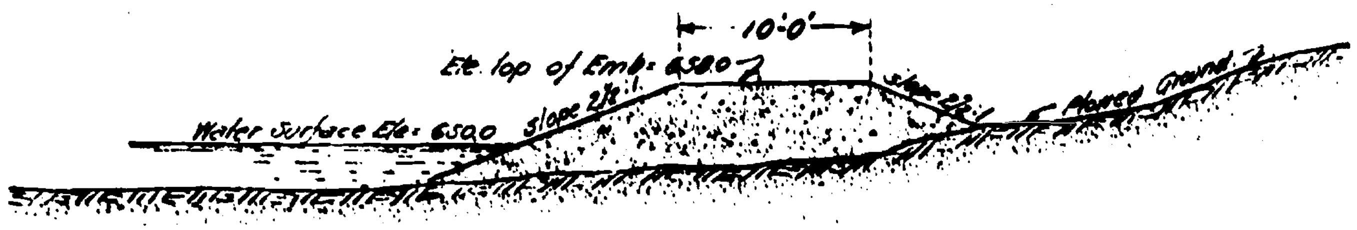

MOUNTAIN DRIVE
MISSION RIDGE ROAD
DRAINAGE DITCH
S=.002
EMBANKMENT ELE=658.
RESERVOIR.
CREST OF DAM. ELE=658.
LOOSE STONE PITCHING
SPILLWAY PAVED WITH LOOSE & PITCHED ROCKS
A
655
650
645
640
635
630
EMBANKMENT ELE=658.
DRAINAGE DITCH
DRG. NO. 1
Preliminary Sketch for
MISSION RIDGE RESERVOIR.
WATER WORKS DEPARTMENT
CITY OF SANTA BARBARA.
Scale 1"=100'
July-1917.
Engineering Offices.
J. B. LIPPINCOTT.
LOS ANGELES, CALIFORNIA.

FIGURE 1



4'0"

TYPICAL SECTION OF EARTH FILLED DAM.



TYPICAL SECTION OF EARTH FILLED EMBANKMENT.

Note: The material for the fill of the main body of the dam and embankments is to be excavated from the sides of the reservoir and these excavated sides are to be dressed to a continuation of the water side slopes of the embankments.

DRG. NO. 2

Preliminary Sketch
of
CROSS-SECTIONS OF DAM AND EMBANK
FOR.
MISSION RIDGE RESERVO
WATER WORKS DEPARTMENT
CITY OF SANTA BARBA

Scale 1:10' July-1917.

Engineering Offices
J.B. LIPPINCOTT.
Los Angeles, California.

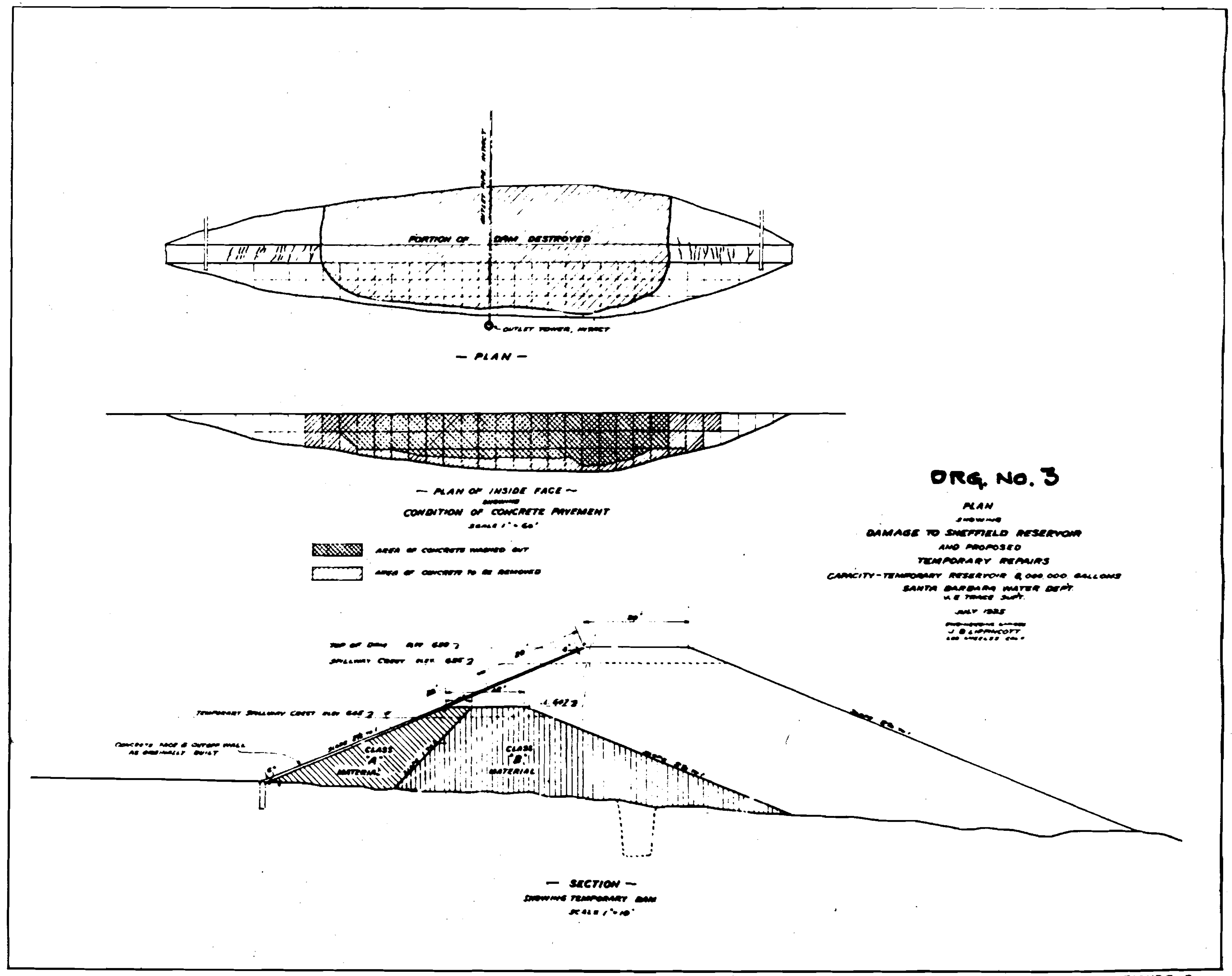
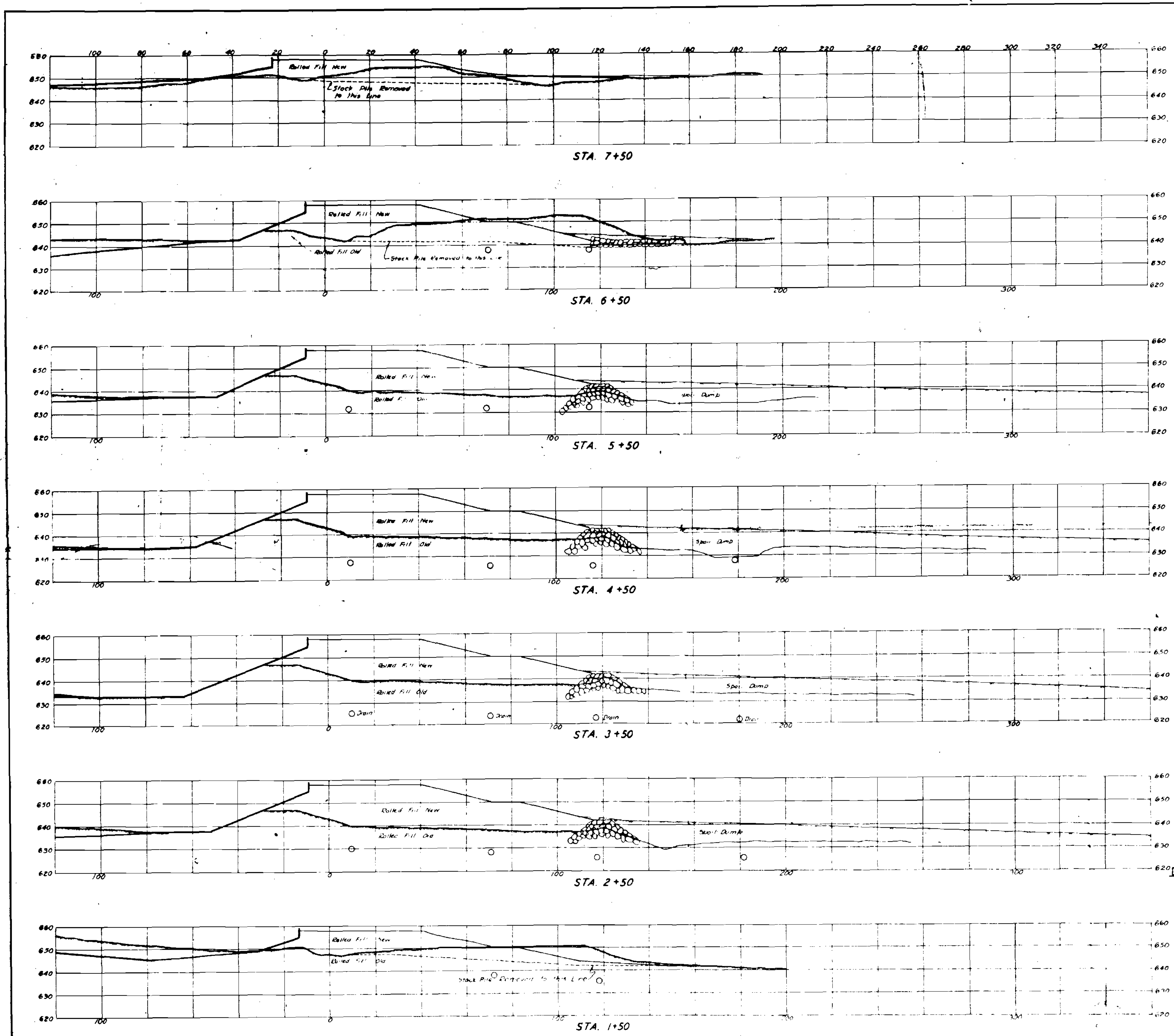

PORTION OF DAM DESTROYED
OUTLET TOWER
— PLAN —
— PLAN OF INSIDE FACE —
CONDITION OF CONCRETE PAVEMENT
AREA OF CONCRETE WASHED OUT
AREA OF CONCRETE TO BE REMOVED
DRG. NO. 3
PLAN
DAMAGE TO SHEFFIELD RESERVOIR
AND PROPOSED
TEMPORARY REPAIRS
CAPACITY-TEMPORARY RESERVOIR 8,000,000 GALLONS
SANTA BARBARA WATER DEPT.
J. B. LIPPINCOTT
CONCRETE FACE & CUTOFF WALL AS ORIGINALLY BUILT
CLASS "A" MATERIAL
CLASS "B" MATERIAL
— SECTION —
SHOWING TEMPORARY DAM

FIGURE 3



APPLICATION No. 11-2
APPROVED AS TO SAFETY
December 22, 1936

CITY OF SANTA BARBARA

P.W.A. NO. CALIF. 1312-1 R SHEFFIELD RESERVOIR

CROSS-SECTIONS OF DAM SHOWING PRESENT CONDITION AS CONSTRUCTED

QUINTON, CODE & HILL-LEEDS & BARNARD
ENGINEERS CONSOLIDATED-LOS ANGELES

DRAWN	SCALE 1 IN. = 20 FT. NOV. 1, 1936
APPROVED	7643-SB24.2

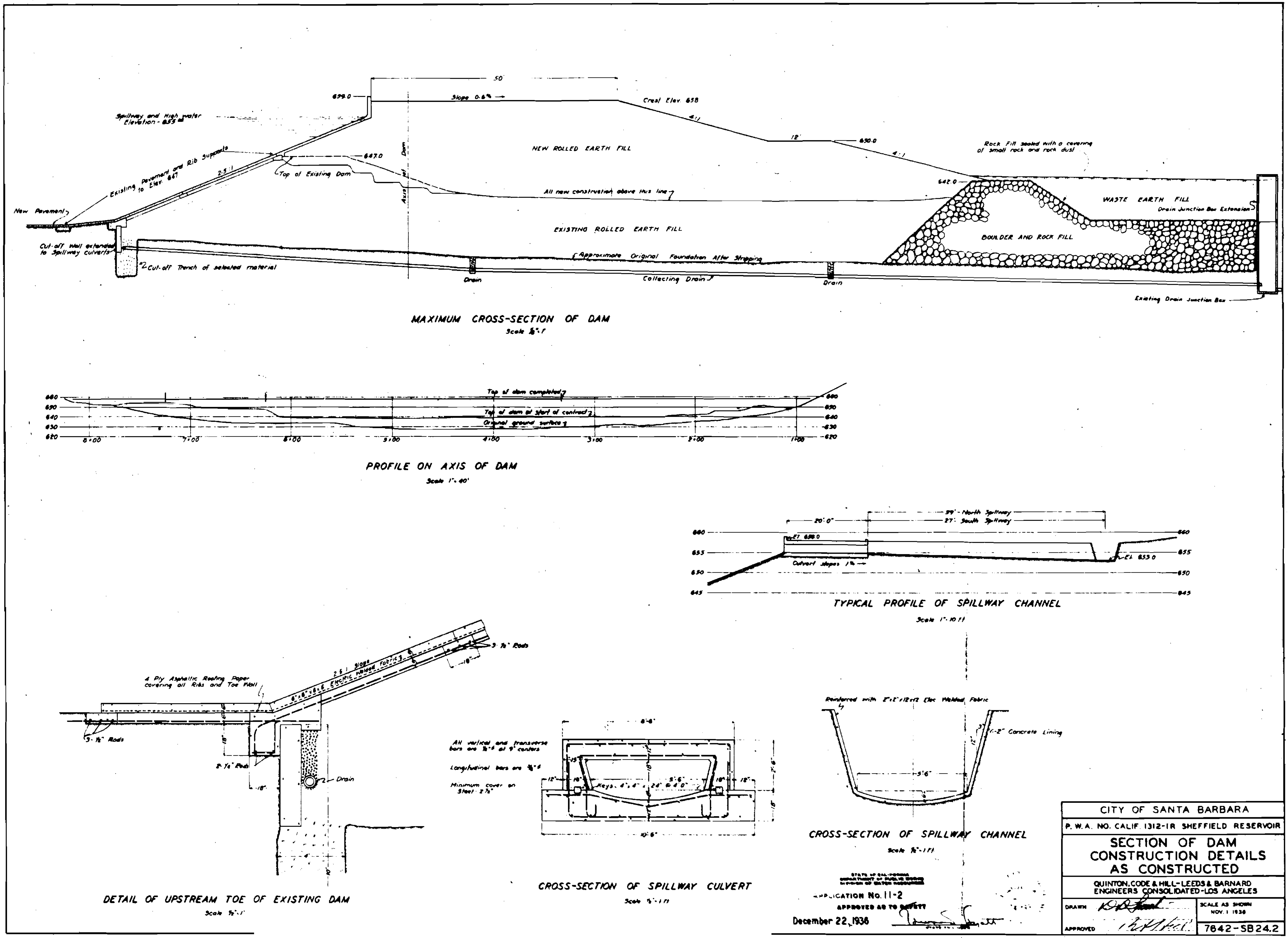
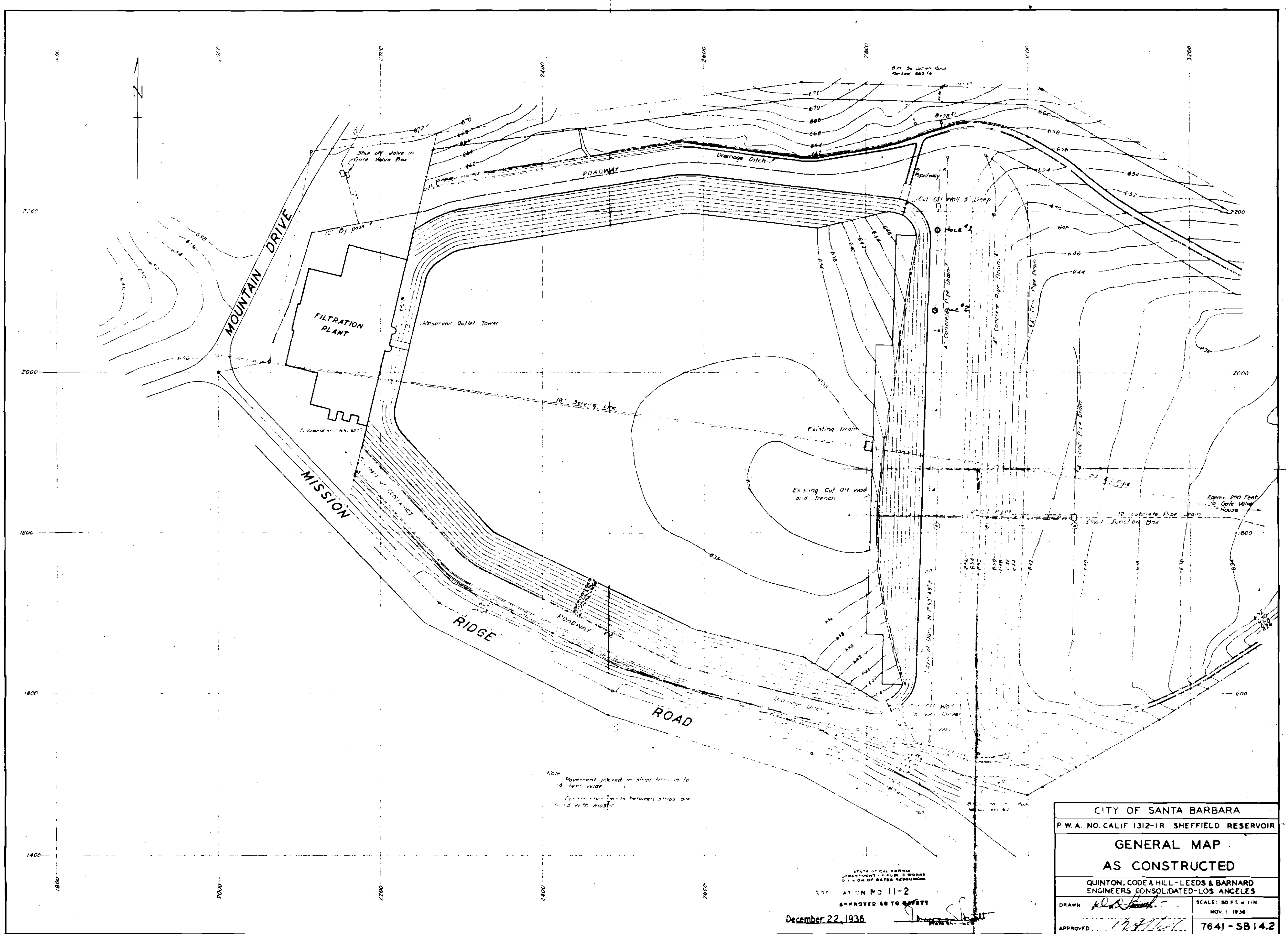

MAXIMUM CROSS-SECTION OF DAM
Crest Elev. 658
NEW ROLLED EARTH FILL
EXISTING ROLLED EARTH FILL
All new construction above this line
Top of Existing Dam
Axis of Dam
Spillway and High water Elevation - 655.00
Existing Pavement and Rib Supports to Elev. 647
New Pavement
Cut-off Wall extended to Spillway culverts
Cut-off Trench of selected material
Approximate Original Foundation After Stripping
Collecting Drain
Drain
Rock Fill sealed with a covering of small rock and rock dust
WASTE EARTH FILL
BOULDER AND ROCK FILL
Drain Junction Box Extension
Existing Drain Junction Box
PROFILE ON AXIS OF DAM
Scale 1" = 40'
Top of dam completed
Top of dam at start of contract
Original ground surface
TYPICAL PROFILE OF SPILLWAY CHANNEL
Scale 1" = 10 ft
39' - North Spillway
27' - South Spillway
Culvert slopes 1%
El. 655.0
DETAIL OF UPSTREAM TOE OF EXISTING DAM
4 Ply Asphaltic Roofing Paper covering all Ribs and Toe Wall
2.5:1 Slope
6"x6"-6x6 Electric Welded Fabric
Drain
CROSS-SECTION OF SPILLWAY CULVERT
All vertical and transverse bars are ½" φ at 9" centers
Longitudinal bars are ⅜" φ
Minimum cover on Steel 2½"
CROSS-SECTION OF SPILLWAY CHANNEL
Scale ½" = 1 ft
Concrete Lining
STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES
APPLICATION No. 11-2
APPROVED AS TO SAFETY
December 22, 1936
CITY OF SANTA BARBARA
P. W. A. NO. CALIF. 1312-1R SHEFFIELD RESERVOIR
SECTION OF DAM
CONSTRUCTION DETAILS
AS CONSTRUCTED
QUINTON, CODE & HILL-LEEDS & BARNARD
ENGINEERS CONSOLIDATED-LOS ANGELES
DRAWN
APPROVED
SCALE AS SHOWN
NOV. 1 1936
7642-SB24.2

FIGURE 5



N
MOUNTAIN DRIVE
MISSION
RIDGE
ROAD
FILTRATION PLANT
ROADWAY
Drainage Ditch
Spillway
Reservoir Outlet Tower
Shut off Valve in Gate Valve Box
Existing Drain
Existing Cut Off Wall and Trench
Junction Box
Approx. 200 Feet to Gate Valve House
18" Service Line
CITY OF SANTA BARBARA
P.W.A. NO. CALIF. 1312-1R SHEFFIELD RESERVOIR
GENERAL MAP
AS CONSTRUCTED
QUINTON, CODE & HILL-LEEDS & BARNARD
ENGINEERS CONSOLIDATED-LOS ANGELES
DRAWN
APPROVED
SCALE: 50 FT. = 1 IN.
NOV. 1 1936
7641 - SB 14.2
STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES
APPROVED AS TO SAFETY
December 22, 1936

FIGURE 4

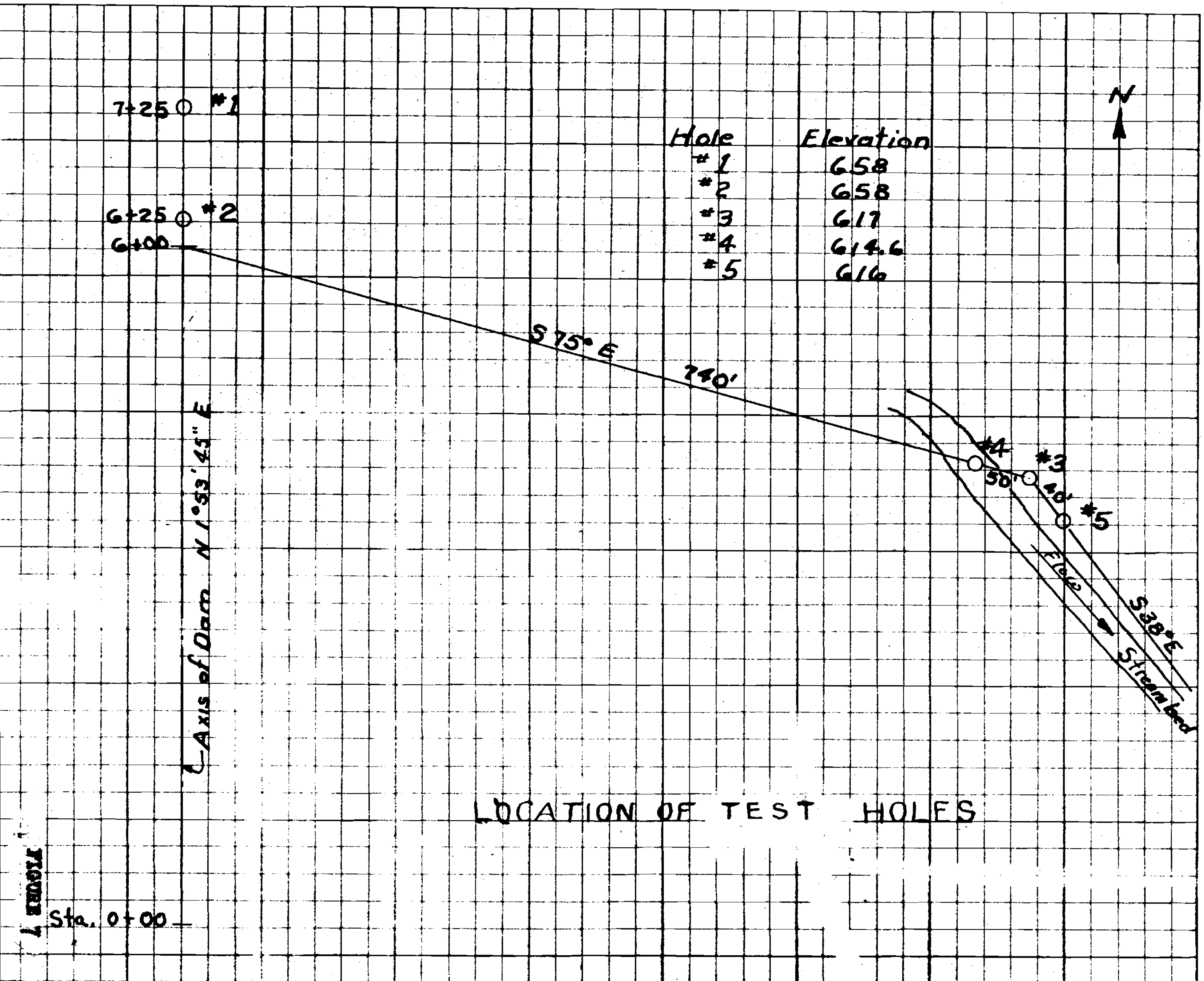
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, LOS ANGELES DISTRICT

Subject: Flow Slide Phenomena - Sheffield Dam

Computation: Location of Auger Holes   No.

Computed by: C.A.F.   Checked by: Date: 2/24/49

Page



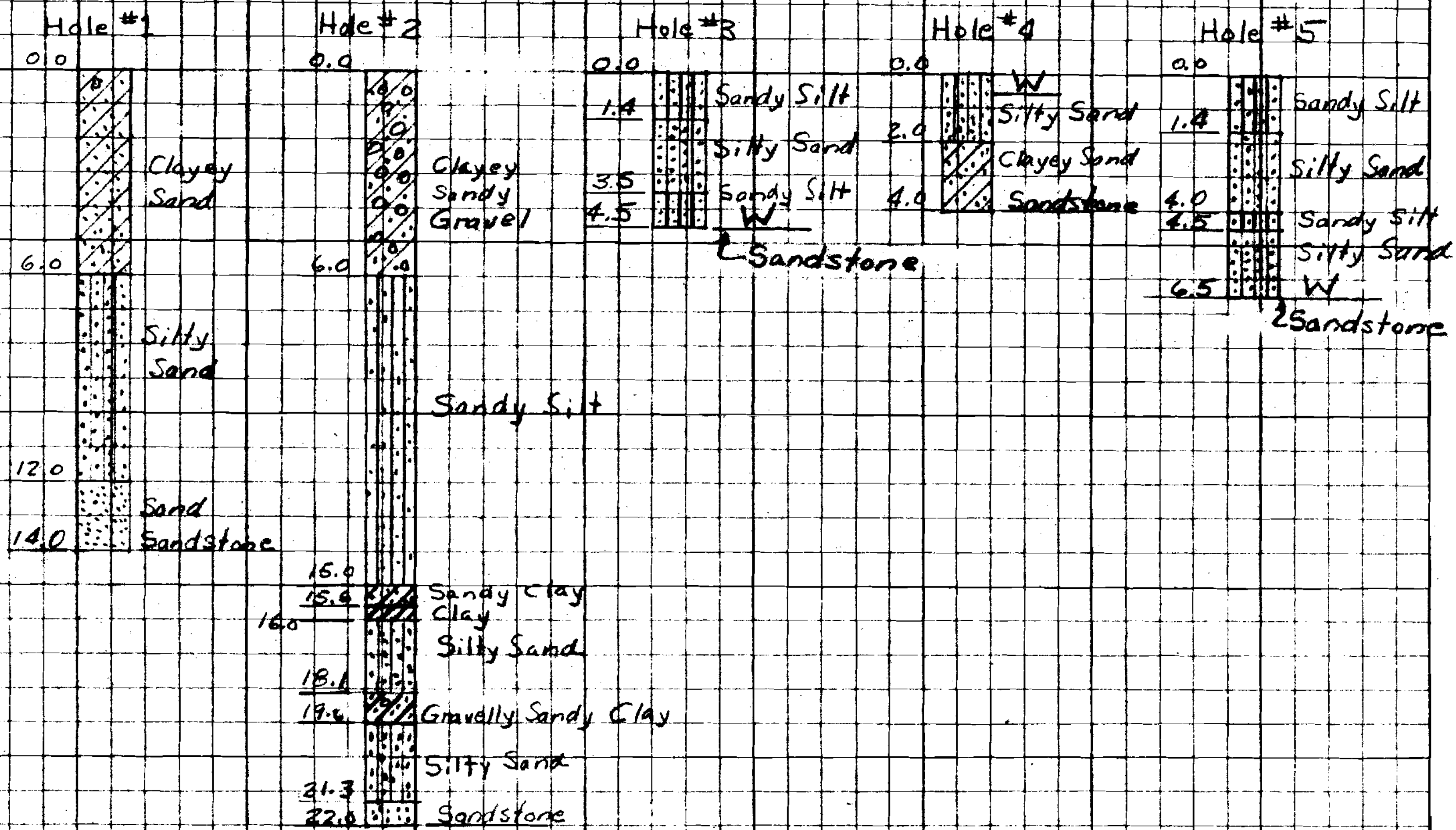
Hole	Elevation
#1	658
#2	658
#3	617
#4	614.6
#5	616

LOCATION OF TEST HOLES

FIGURE 7

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, LOS ANGELES DISTRICT

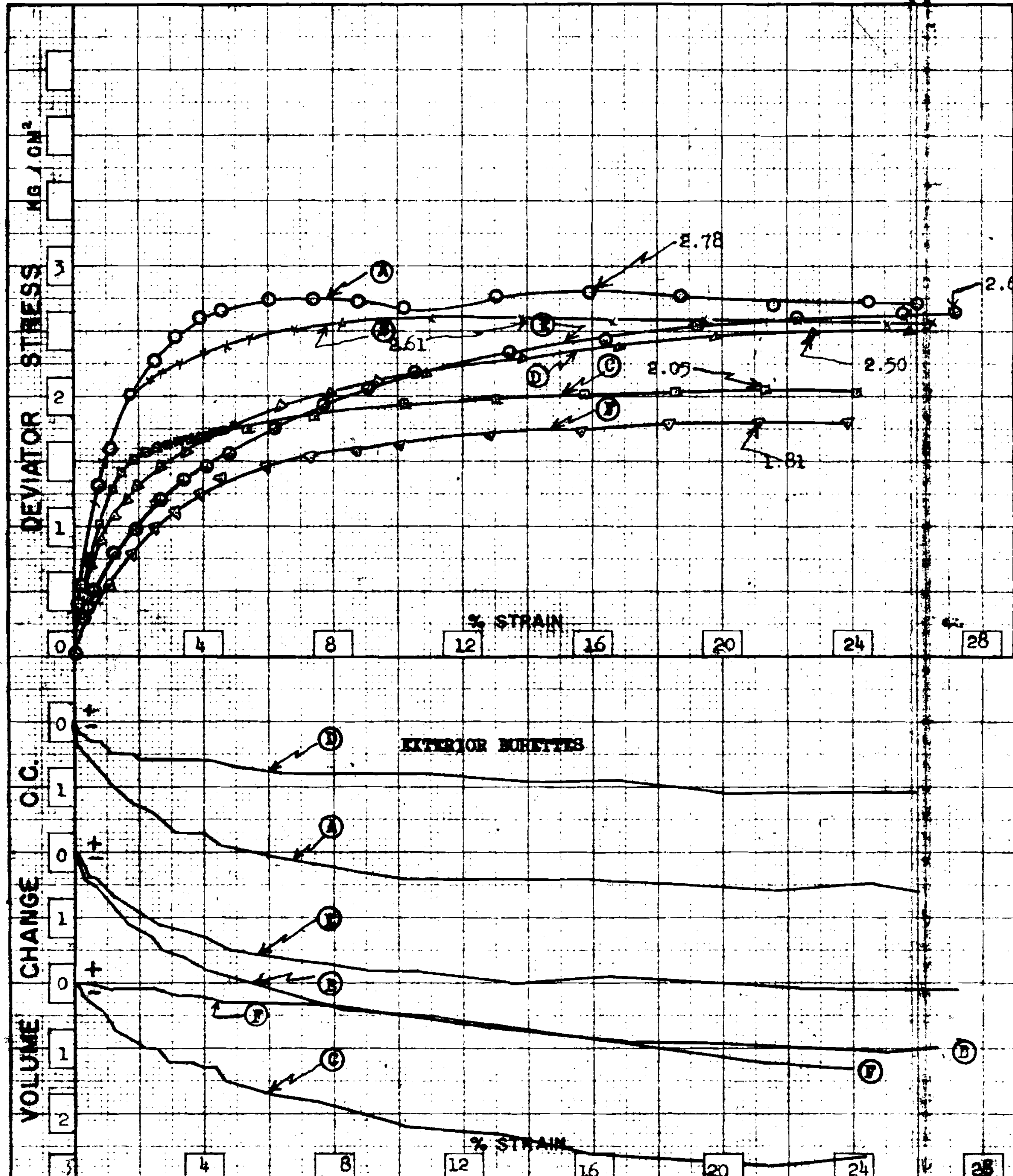
Subject: Flow Slide Phenomena - Sheffield Dam
Computation: Log of Test Holes
Computed by: C.A.F
Checked by:
Date: 6/1/49
No.
Page

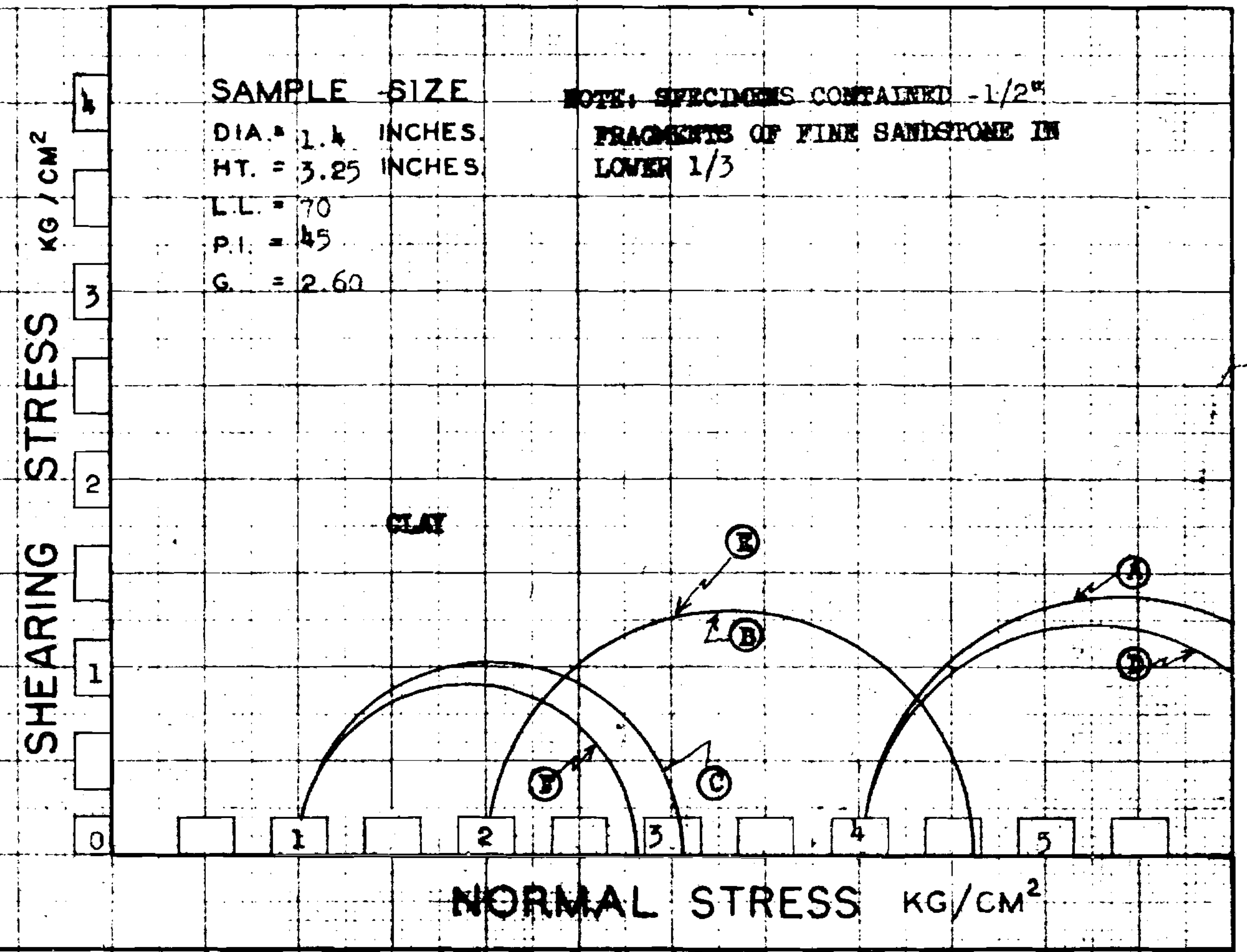


LOG OF TEST HOLES

FIGURE 8

CORPS OF ENGINEERS U.S. ARMY





CURVE	LATERAL PRESSURE KG / CM²	INITIAL DRY DENSITY LB. / CU. FT.	MOISTURE BEFORE TEST %	FINAL MOISTURE %	% SATURATION
A	4.0	81.7	30.4	29.5	78
B	2.0	83.4	28.2	27.6	76
C	1.0	85.5	27.3	26.8	78
D	4.0	84.4	30.4	29.5	83
E	2.0	86.7	28.2	27.6	82
F	1.0	87.8	27.3	26.8	82

TRIAXIAL SHEAR TESTS

UNCONSOLIDATED-UNDRAINED

PROJECT: SHEFFIELD DAM

DISTRICT: LOS ANGELES

LAB. SERIAL Nº LA-4077

HOLE Nº 2

FIELD SAMPLE Nº 1

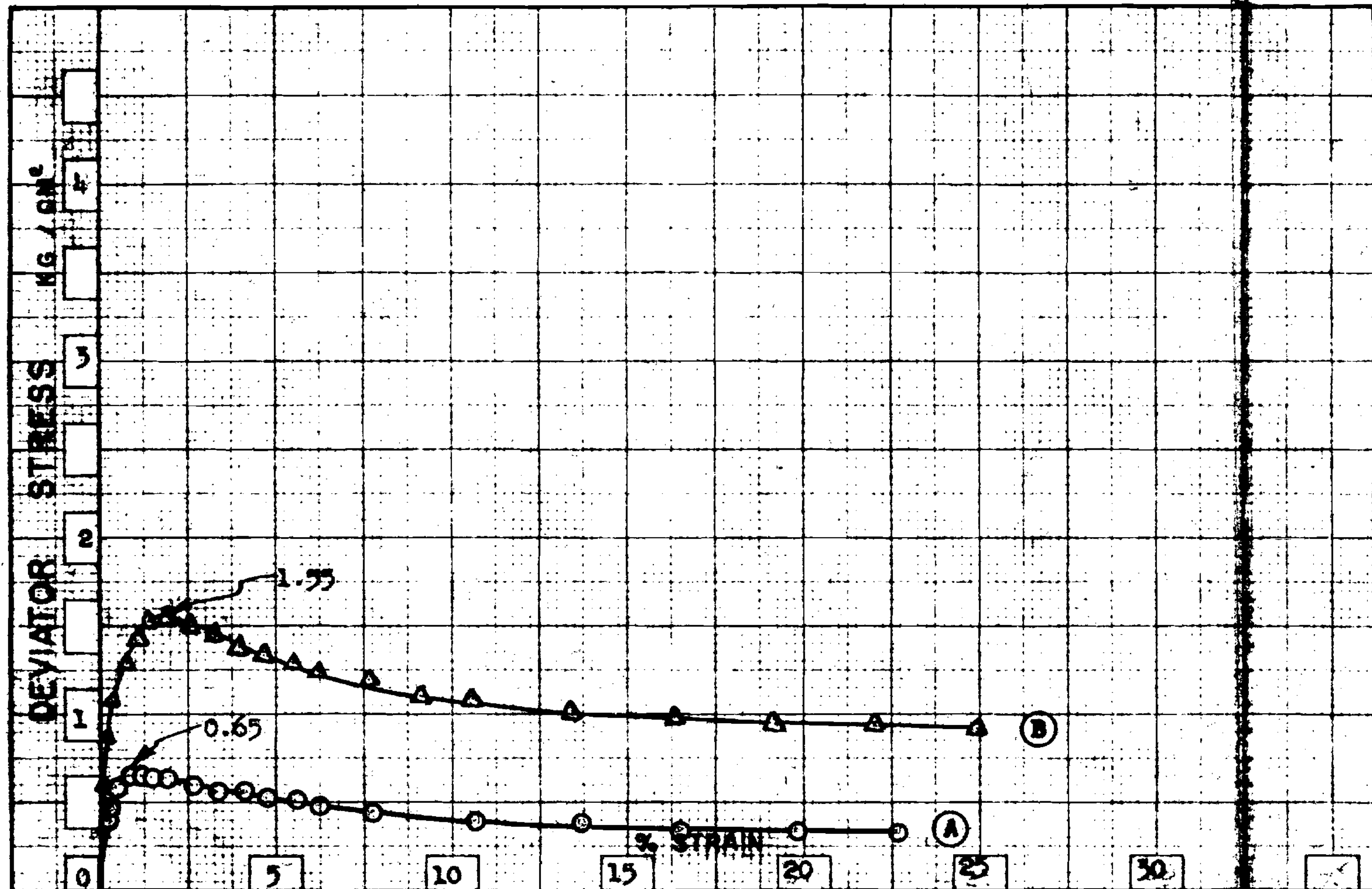
DEPTH 15.6'-16.0'

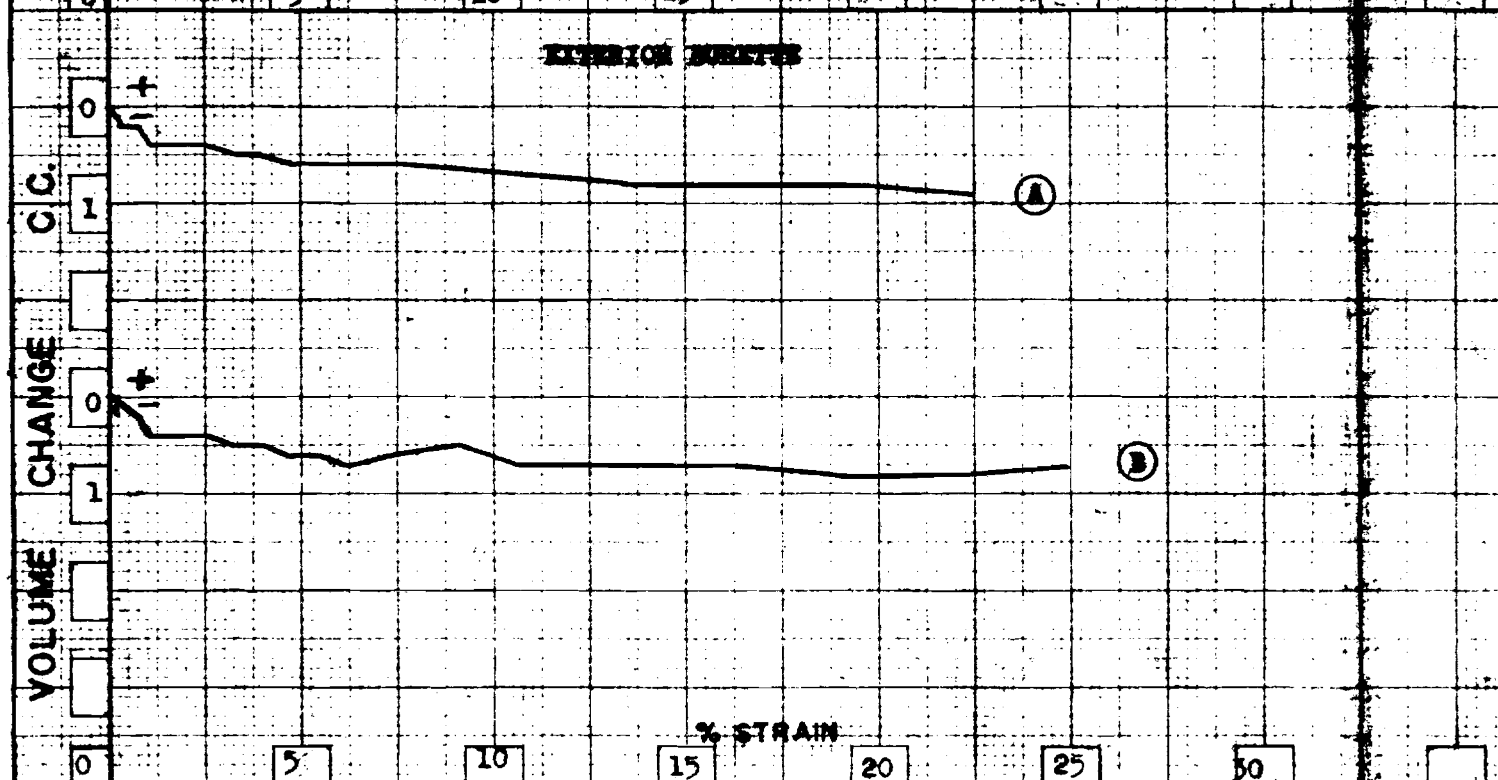
CONDITION OF SPECIMENS A, B & C UNDISTURBED. D, E & F ARE REMOLDS OF A, B & C RESPECTIVELY.

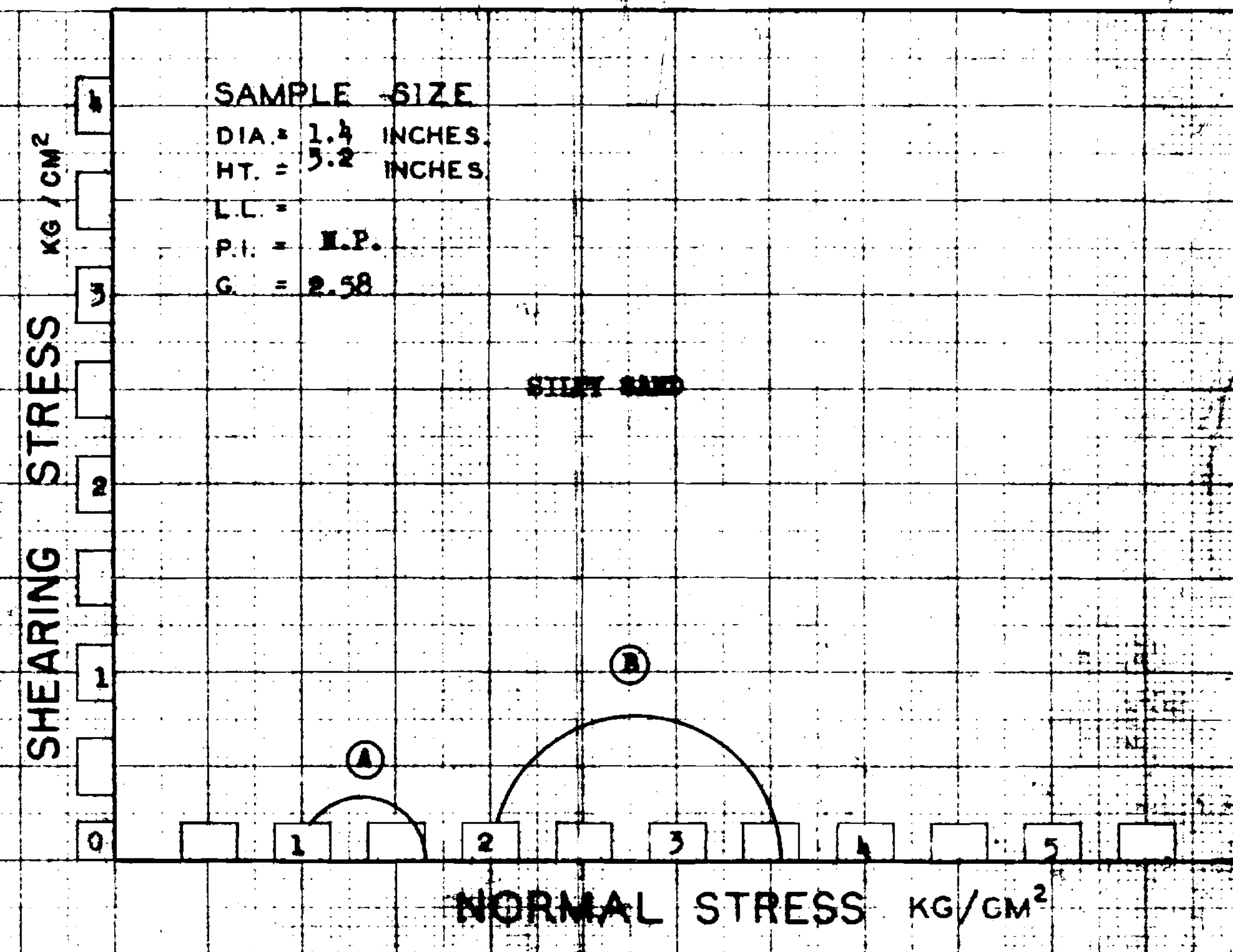
SOUTH PACIFIC DIVISION TESTING LABORATORY

TESTED	COMPUTED	CHECKED	DRAWN
W.H.M.	E.A.H.	G.A.F.	H.A.M.

FIGURE 9







CURVE	LATERAL PRESSURE KG / CM²	INITIAL DRY DENSITY LB. / CU. FT.	MOISTURE BEFORE TEST %	FINAL MOISTURE %	FINAL % SATURATION
A	1.0	91.1	13.8	23.1	86
B	2.0	88.9	13.5	22.2	81

TRIAXIAL SHEAR TESTS

CONSOLIDATED - UNDRAINED

PROJECT: SHEFFIELD DAM — DISTRICT: LOS ANGELES

LAB. SERIAL Nº LA-4079 — HOLE Nº 3

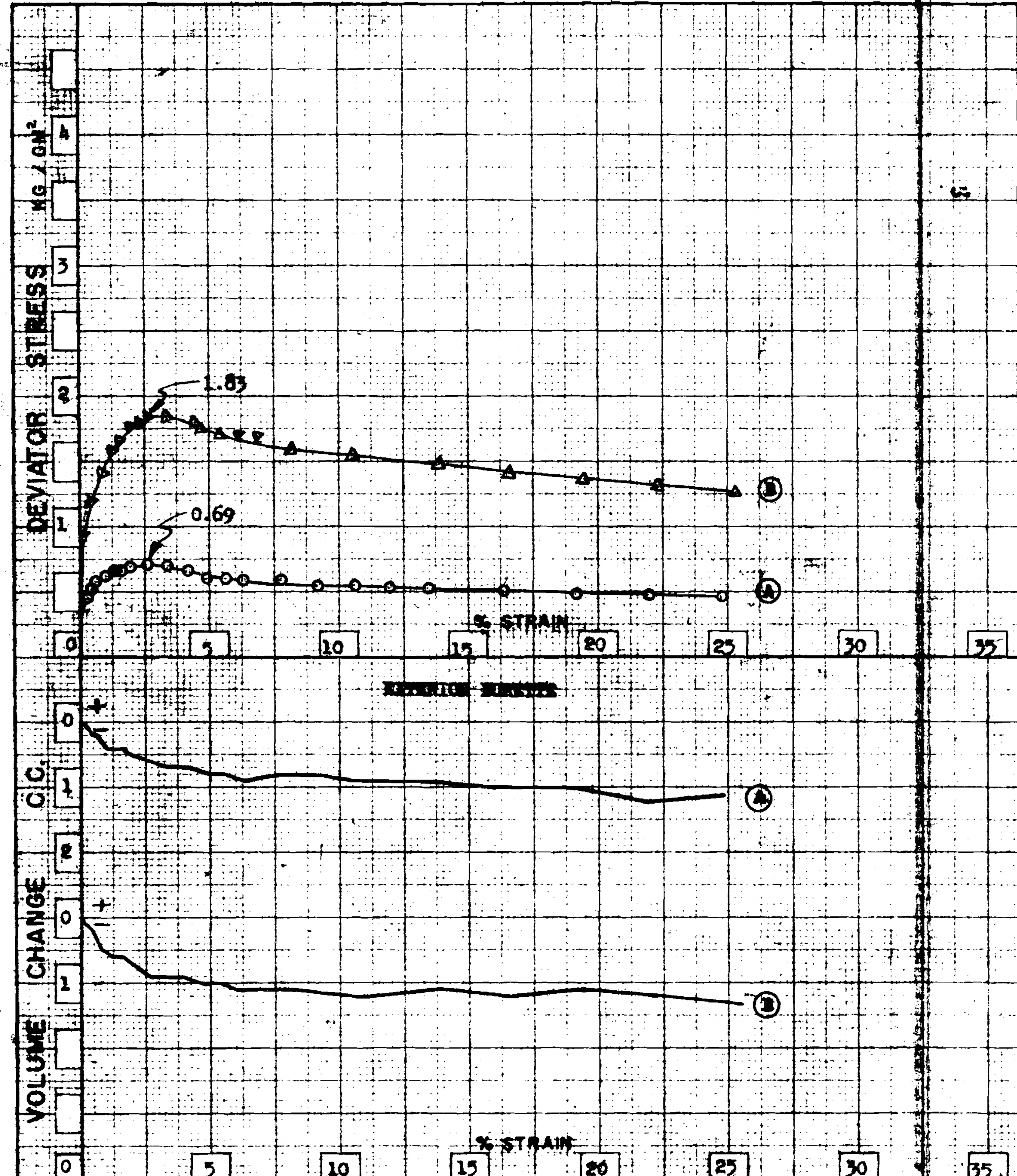
FIELD SAMPLE Nº 1 — DEPTH 1.0' - 1.4'

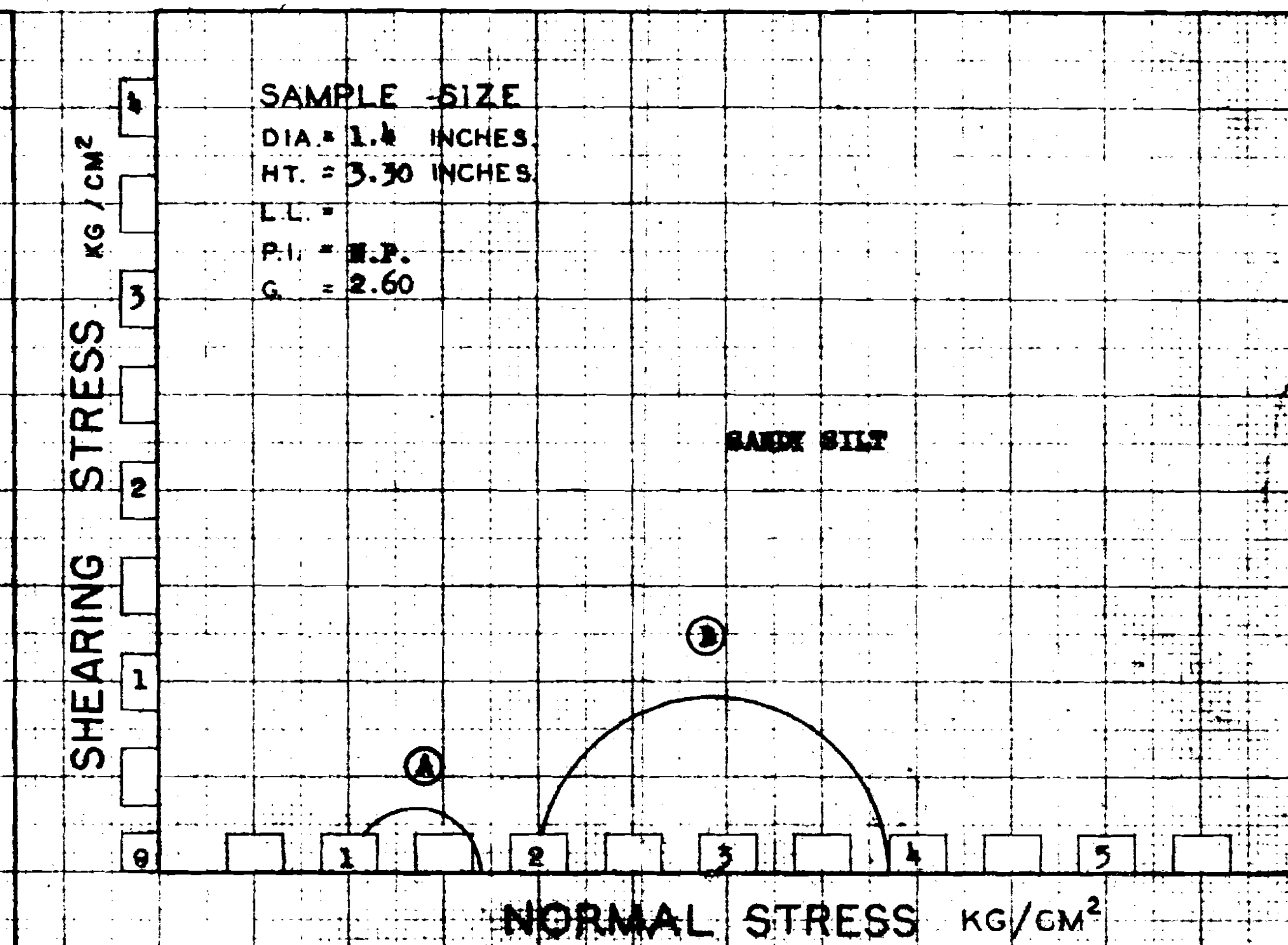
CONDITION OF SPECIMENS UNDISTURBED

SOUTH PACIFIC DIVISION TESTING LABORATORY

TESTED	COMPUTED	CHECKED	DRAWN
V.M.	E.A.H.	G.A.F.	W.E.B.

FIGURE 10





CURVE	LATERAL PRESSURE KG / CM²	INITIAL DRY DENSITY LB. / CU. FT.	MOISTURE BEFORE [illegible] %	FINAL MOISTURE %	FINAL % SATURATION
A	1.0	86.1	14.7	25.8	88
B	2.0	91.5	13.8	22.9	93

TRIAXIAL SHEAR TESTS

CONSOLIDATED -- UNDRAINED

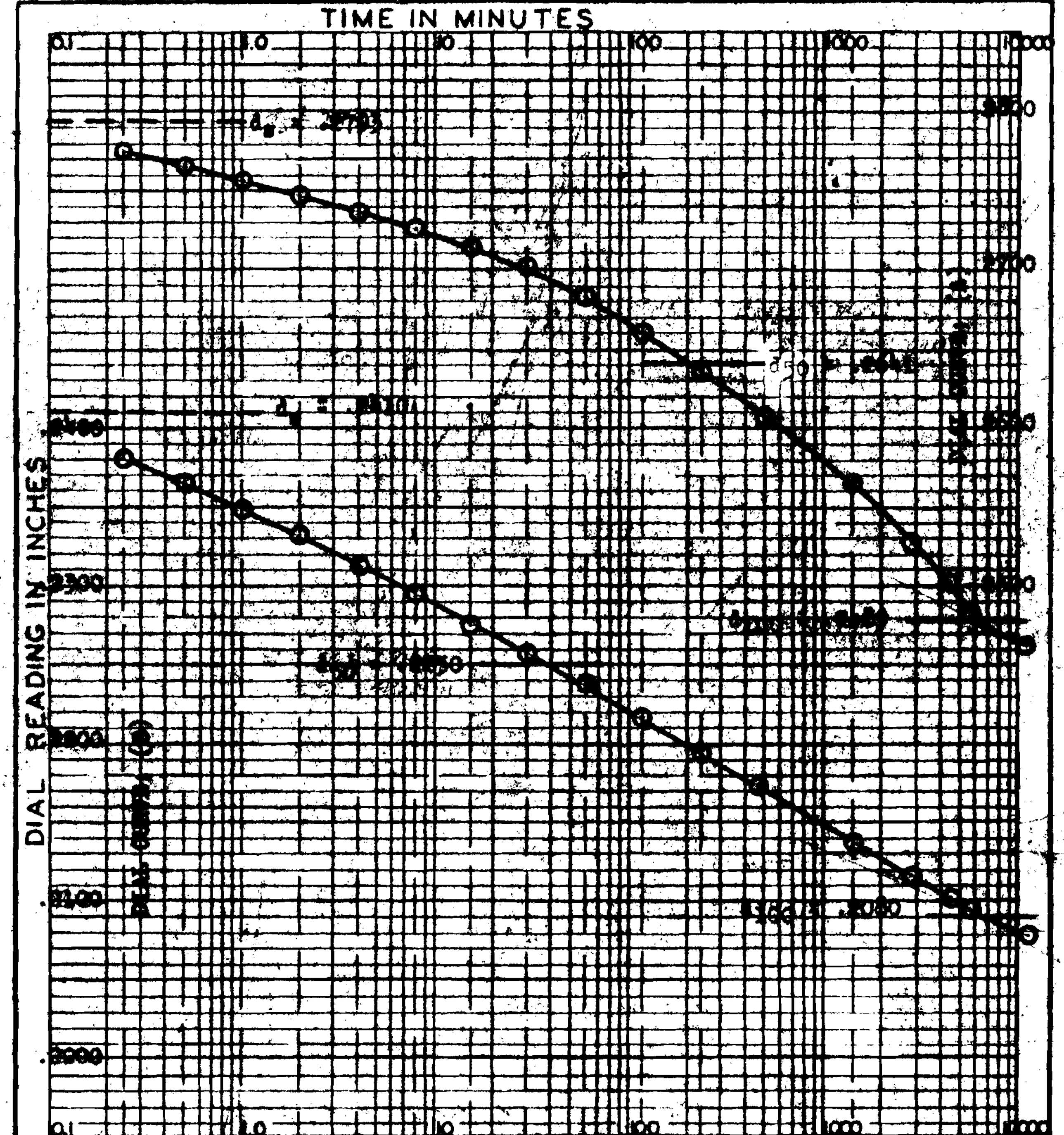
PROJECT: SHEFFIELD DAM
DISTRICT: LOS ANGELES
LAB. SERIAL Nº LA-4082
HOLE Nº 5
FIELD SAMPLE Nº 1
DEPTH 0.4' - 1.4'
CONDITION OF SPECIMENS UNDISTURBED

SOUTH PACIFIC DIVISION TESTING LABORATORY

TESTED	COMPUTED	CHECKED	DRAWN
E.A.H.	E.A.H.	G.A.F.	W.E.B.

FIGURE 11

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS



CURVES

	A	B
P_1 KG/CM^2	0.2	0.4
P_2 KG/CM^2	0.4	0.8
t_{50}, MIN	200	40
t_{100}, MIN	6000	6300

SPECIMEN SATURATED AFTER FINAL DIAL READING UNDER 0.2 KG/CM^2 LOAD.

SOUTH PACIFIC DIVISION
TESTING LABORATORY

TIME-COMPRESSION DIAGRAM

SHEFFIELD DAM

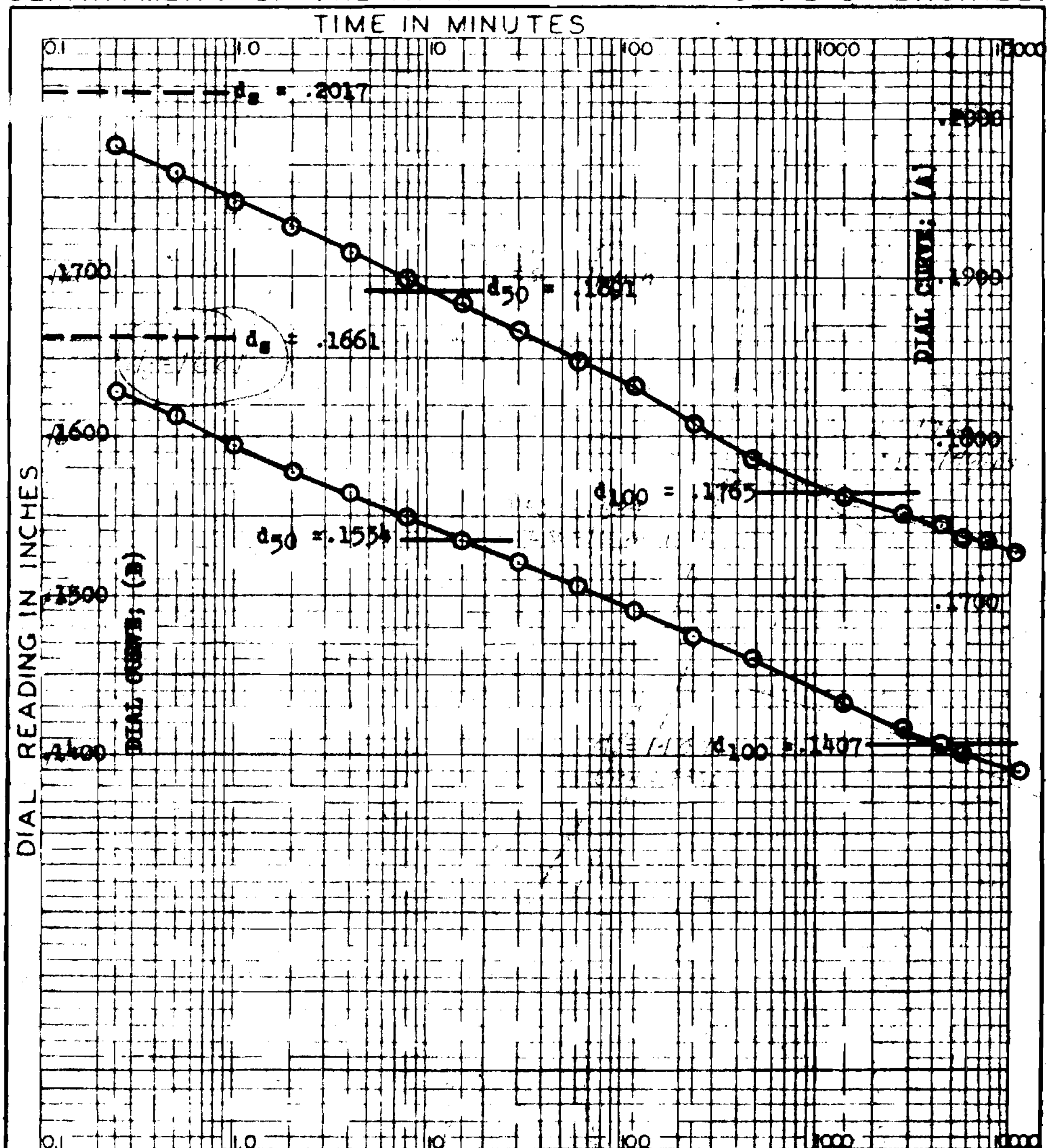
SERIAL NO. LA-4082

HOLE NO. 5 DEPTH 0.4 - 1.4

TESTED	COMPUTED	CHECKED	DRAWN
C.B.D.	C.B.D.	G.A.F.	W.E.B.

FIGURE 12

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS



CURVES

	A	B
P_1 KG/CM^2	0.8	1.6
P_2 KG/CM^2	1.6	3.2
t_{50}, MIN	11	16
t_{100}, MIN	1400	4300

SOUTH PACIFIC DIVISION
TESTING LABORATORY

TIME-COMPRESSION DIAGRAM

SHEFFIELD DAM

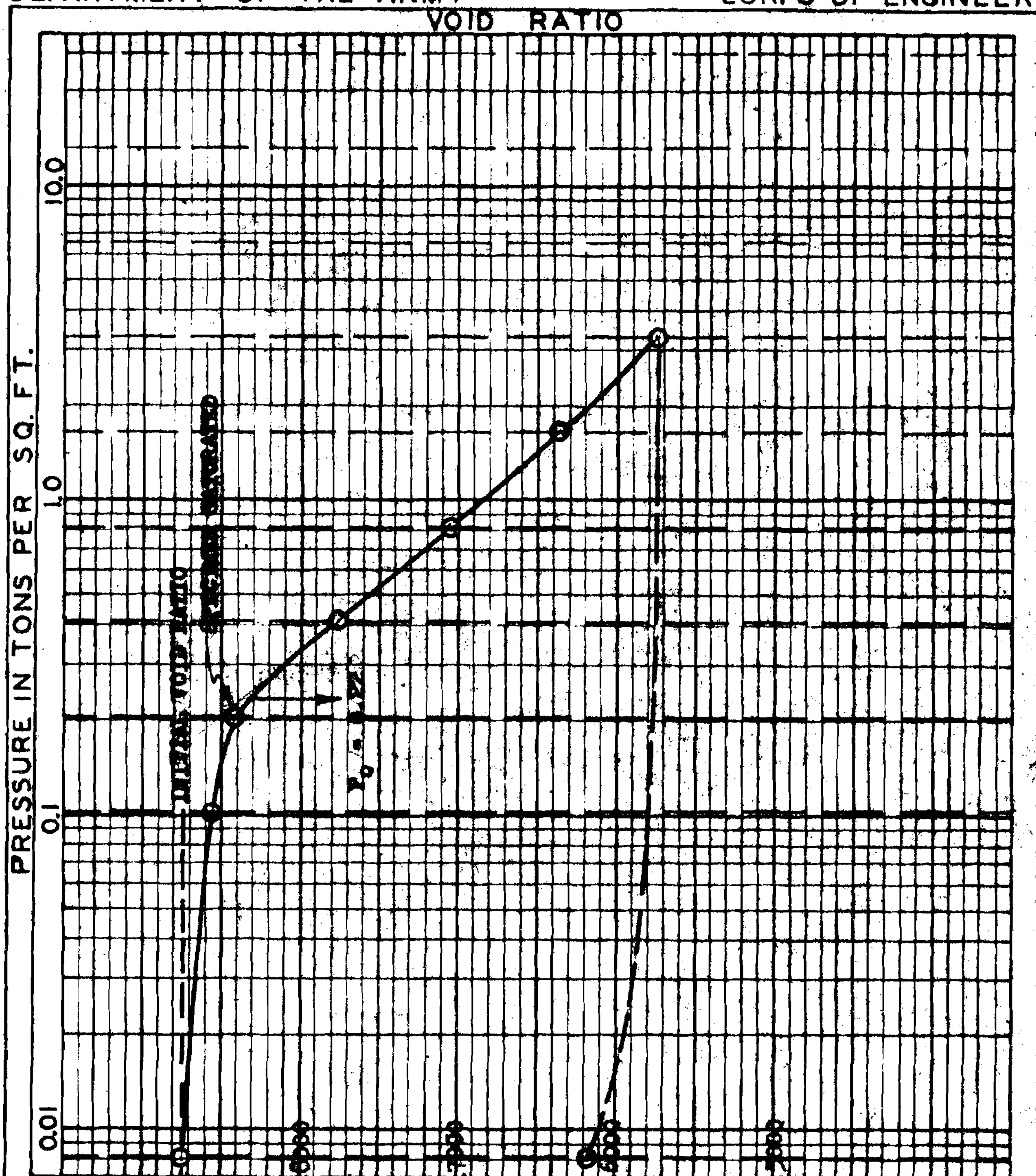
SERIAL NO. LA-4082

HOLE NO. 5 DEPTH 0.4' - 1.4'

TESTED	COMPUTED	CHECKED	DRAWN
C.B.D.	C.B.D.	G.A.F.	W.E.B.

FIGURE 13

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS



SPECIMEN UNDISTURBED

INITIAL DENSITY 86.5 LBS/CU. FT.
INITIAL MOISTURE 14.5%
INITIAL DEGREE SATURATION 43.1%
FINAL DEGREE SATURATION 82.4%
SPECIFIC GRAVITY 2.60

SOUTH PACIFIC DIVISION
TESTING LABORATORY

CONSOLIDATION

SHEFFIELD DAM

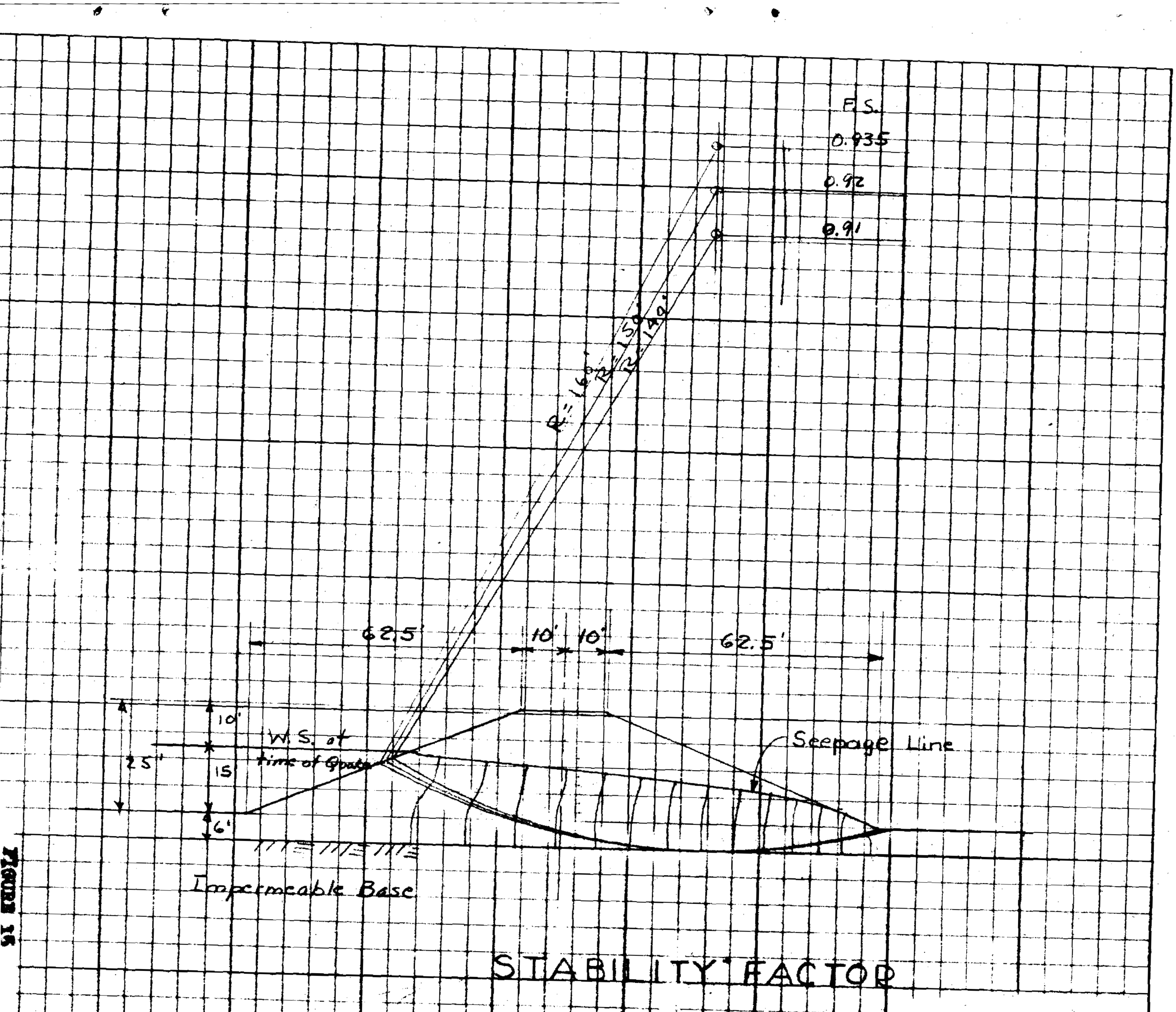
SERIAL NO. LA-4082
HOLE NO. 5 DEPTH 0.4' - 1.4'

TESTED	COMPUTED	CHECKED	DRAWN
C.B.D.	C.B.D.	G.A.F.	W.E.B.

FIGURE 14

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, LOS ANGELES DISTRICT

Subject Flow Slide Phenomena - Sheffield Dam
Computation Critical Circle
Computed by C.A.F. Checked by Date 5/27/49
No.
Page



STABILITY FACTOR

FIGURE 15