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REVIEW OF GREERS FERRY DAM DESIGN

MISCELLANEOUS PAPER NO. 3-582

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

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U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
OFFICE OF THE DIRECTOR
VICKSBURG, MISSISSIPPI

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SUBJECT: Review of Greers Ferry Dam Design

TO: District Engineer
U. S. Army Engineer District, Little Rock
P. O. Box 867
Little Rock, Arkansas

As part of the review of earth dam projects in the Little Rock District which you requested us to undertake, we have examined available design memoranda and other information furnished us on the design of Greers Ferry Dam. Our comments with respect to those features of the geology, soil conditions, and stability computations which may have bearing on the stability of the earthen sections of the dam (dikes 1 and 2) are given in the inclosed report, Miscellaneous Paper No. 3-582.

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ALEX G. SUTTON, JR.
Colonel, Corps of Engineers
Director

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Preface

The Waterways Experiment Station was requested by the U. S. Army Engineer District, Little Rock, to review from the soils and foundation standpoint the design of three earth dam projects in that district, namely, Beaver, Table Rock, and Greers Ferry Dams. The review of Beaver Dam is presented in WES MP No. 3-499, "Review of Beaver Dam Design," June 1962; the review of Table Rock Dam is presented in WES MP No. 3-544, "Review of Stability Analysis - Table Rock Dam," November 1962. This report presents the results of the review of Greers Ferry Dam.

The work was authorized by disposition form from the U. S. Army Engineer District, Little Rock, to the Director, Waterways Experiment Station, dated 8 May 1962, subject "Critical Review of Earth Dam Projects, Little Rock District. This report was written by Messrs. C. C. Trahan and W. B. Steinriede under the general supervision of Messrs. W. J. Turnbull, W. G. Shockley, J. R. Compton, and W. C. Sherman, Soils Division, Waterways Experiment Station.

Director of the Waterways Experiment Station during the preparation and publication of this report was Colonel Alex G. Sutton, Jr., CE. Technical Director was Mr. J. B. Tiffany.

REVIEW OF GREERS FERRY DAM DESIGN

General

1. Greers Ferry Dam, a concrete gravity structure with two earth dike sections, is located on the Little Red River in Central Arkansas. The Waterways Experiment Station (WES) was requested by the Little Rock District to review the design of Greers Ferry Dam from a soils and foundation standpoint. All data were furnished by the Little Rock District. In addition, a field inspection of the damsite was made by representatives of WES. The review has been completed and the comments presented herein concern those features of the geology, soil conditions, and stability computations which may have bearing on the stability of the dike sections of the dam. The review of the foundation design for the concrete dam did not indicate any problems from a soils or foundation standpoint, and recommendations concerning the drainage system constitute the only comments on this portion of the project.

Geology

2. A study of available photographs and geological logs of borings, along with a review of Design Memoranda Nos. 1, 7, and 8, reveals no reported factors of a geological nature which would materially affect the stability or performance of the various structures. The geological analysis and interpretation of foundation conditions are considered adequate and to have been presented in sufficient detail for design of the structures. Provisions for careful inspection and additional exploration during the construction phase incorporate adequate measures for detecting unforeseen or unusual foundation conditions.

Embankment stability

3. It is considered that the foundation explorations were sufficient to delineate foundation conditions beneath both dike sections. The foundations consist of a shallow mantle of residual soils overlying rock and are considered to present no problems relative to stability, settlement, or seepage.

4. Record samples. Record samples taken from the dike embankments indicated a considerable variation in materials in both the impervious core and random fill sections. The materials consisted primarily of lean and fat clays and shale with varying amounts of gravel and sand. Atterberg limits for the embankment materials are shown in the plasticity chart (see fig. 1). Because of the wide variation in material types and the limited number of record samples, a direct comparison of the densities of the record samples with the maximum densities determined in the laboratory could not be made. Results of standard compaction tests listed in table 3 of DM No. 8 indicate optimum water contents ranging from 12.7 to 26.0 percent and maximum dry densities ranging from 95.8 to 123.0 lb/cu ft. These values represent both the core and random fill materials. It was assumed in design that mixing of the borrow soils would occur to such an extent that no one type of material would form an extensive or continuous zone in the embankment and that field densities of 97.5 percent or greater of standard AASHO density would be obtained during construction. It is understood that zonation was effected during construction by selected routing of hauling equipment so that materials with the greater percentages of rock fragments were placed toward the outer slopes of the dikes. Record samples from the impervious core section indicated placement water contents ranging from 14 to 28 percent and dry densities ranging from 96 to 117 lb/cu ft. The data from the record samples indicate an unusually wide range of placement conditions, reflecting the variability of the materials used.

5. Results of laboratory shear strength tests on the record samples are tabulated in tables 1 and 2 and are shown graphically in figs. 2 and 3. The reference numbers shown adjacent to shear envelopes in figs. 2 and 3 correspond to reference numbers shown in tables 1 and 2. Shear strengths were selected for check stability computations utilizing the results of laboratory shear strength tests on record samples from the foundation and from the impervious core and random fill sections of dike 1. Shear strength values selected were as follows:

	Shear Strength					
	Q Tests		R Tests		S Tests	
	$\phi°$	c,tsf	$\phi°$	c,tsf	$\phi°$	c,tsf
Foundation	15	0.60	15	0.20	25	0.20
Impervious core	--	--	14	0.40	25	0.0
Random fill	--	--	13	0.45	25	0.0

The above-listed shear strengths are shown graphically in figs. 2 and 3, together with the shear strength envelopes used in design. The design shear strengths consisted generally of combined strength envelopes (see plate 19 of DM 8); consequently, a direct numerical comparison with the shear strengths based on record samples is not possible. In some cases the shear strengths used in design were more conservative than WES selected values, while in other cases the strengths used in design were less than those used in our computations. In general, the shear strengths based on record samples did not differ greatly from the design shear strengths.

6. As no Q tests were performed on record samples from the impervious core and random fill, a Q shear strength of $\phi = 0°$, $c = 1.25$ ton/sq ft was selected for both the impervious core and the random fill based on the results of shear strength tests performed in connection with design studies (see plate 17 DM 8).

Check stability
analyses by WES

7. Check stability analyses were performed on a typical section of dike 1 (see fig. 4). Dike 1 is about 80 ft high, whereas dike 2 is only about 30 ft high. Strength tests on record samples from dike 2 (see fig. 3) indicate that materials used in dike 2 were similar to those used in dike 1. Although no stability analyses were performed on dike 2, it is considered that this dike is inherently more stable with respect to sliding than dike 1 because of its lesser height.

8. Results of the stability analyses are shown in table 3. The factors of safety shown are the most critical of several arcs analyzed. The locations of critical arcs are shown in fig. 4. For comparison, the factors of safety computed for design are also shown in table 3. A close agreement may be

noted between WES computed factors of safety and the factors of safety computed for design, except for the construction condition. The WES factor of safety for the construction condition was 2.47 as compared to 1.7 computed for design. Based on the results of the stability analyses, it is considered that the dikes are safe with respect to sliding.

Drainage system
for concrete dam

9. Our only comment with respect to the concrete gravity dam is in connection with measurements for checking the effectiveness of the grout curtain and the adequacy of the drainage system. Reference is made to WES MP No. 3-499, "Review of Beaver Dam Design," June 1962, containing our comments on the drainage system for the concrete section of Beaver Dam. Our recommendations that piezometers be installed beneath that dam to check the effectiveness of the grout curtain and the drain holes to determine whether the existing drains will need to be cleaned, or whether additional drains may be necessary in the future, are equally applicable to the concrete section of Greers Ferry Dam.

Summary

10. In summary, it is considered that the geology and soil conditions beneath the concrete dam and earth dikes were adequately explored and reasonably well defined. The earth dikes were properly designed with adequate factors of safety provided. A system of piezometers should be installed beneath the concrete dam to permit a check on the effectiveness of the grout curtain and drainage system and to indicate any decrease in efficiency of the drain holes with time.

Table 1

Results of Tests on Record Samples, Dike No. 1

Ref No.	Location: Station	Location: Offset from Centerline	EL	Atterberg Limits: LL	Atterberg Limits: PI	w %	γ_d lb/cu ft	Q: ϕ°	Q: c tsf	R: ϕ°	R: c tsf	S: ϕ°	S: c tsf
Foundation													
1	285+00	175' DS	421	77	52	23	97	20	0.6	19	0.1	17	0.3
2	285+00	175' US	425	47	29	22	98	19	0.8			27	0.3
3	280+00	100' DS	412	40	23	25	95	9	0.6	18	0.4		
4	270+00	175' DS	413	31	17	16	104	22	0.7	10	0.3		
5	280+50	190' US	417	24	12	18	104			14	0.1		
6	275+00	175' DS	412	38	23	17	97			14	0.5		
7	270+00	175' DS	413	31	17	16	104			20	0.2	30	0.2
8	265+00	175' DS	415	44	26	18	100			16	0.2		
9	270+00	100' US	418	28	13	19	105					34	0.2
10	265+00	100' DS	423	39	20	20	101	20	0.8	25	0.3	27	0.2
11	294+00	26' US	466	71	52	28	97			8	0.3	19	0.2
12	290+00	12' DS	437	64	38	19	112			18	1.1		
13	285+00	35' US	424	46	29	19	109			30	0.4	28	0.4
14	285+00	30' DS	424	66	45	27	105			12	0.4		
15	280+00	30' DS	414	78	56	32	90			13	0.3	19	0.2
16	280+00	25' US	424	63	39	30	95			15	0.2		
Impervious Core													
17	290+00	--	445	67	43	28	96			12	0.9	23	0.2
18	285+00	--	425	64	43	19	106			14	0.3		
19	290+00	5' US	450	71	44	21	100			14	0.4	23	0
20	270+00	12' DS	422	56	36	19	104			17	0.5	23	0.2
Random Fill													
21	294+00	--	472	77	52	23	105			13	0.6	19	0.2
22	290+00	100' US	450	43	24	20	111			13	1.0	32	0.2

Table 2

Results of Tests on Record Samples, Dike No. 2

Ref No.	Location: Station	Location: Offset from Centerline	EL	Atterberg Limits: LL	Atterberg Limits: PI	w %	γ_d lb/cu ft	Q: $\phi°$	Q: c tsf	R: $\phi°$	R: c tsf	S: $\phi°$	S: c tsf
Foundation													
23	34+00	43' US	480	77	50	29	96	6	0.4	16	0.2	19	0.2
24	9+00	50' US	505	100	69	37	80			17	0.1		
25	15+00	80' DS	490	54	36	16	111			15	0.2		
26	15+00	30' DS	485	88	61	38	83	8	0.3	18	0.3	20	0.3
27	30+00	100' US	455	45	28	20	103	23	0.3	24	0.5	31	0.3
28	30+20	50' US	457	35	21	17	105			23	0.6		
29	25+00	10' US	468	31	17	15	118			29	0.7	30	0.2
30	31+00	12' DS	468	48	31	20	105			24	0.6	27	0.4
Impervious Core													
31	30+00	--	469	69	44	23	105			18	0.3	23	0
32	20+00	--	484	76	50	24	102			18	0.3	28	0
33	25+00	--	485	40	19	14	117			16	0.7	21	0.3
34	20+00		494	52	30	16	112			14	0.6	25	0.3
Random Fill													
35	20+00	40' DS	491	58	37	15	116					32	0.3
36	30+00	50' DS	475	54	31	16	113					25	0.1
37	35+00	50' US	484	61	39	25	103			14	0.5		
38	20+00	30' US	488	55	33	19	108					18	0.4

Table 3

Results of Stability Analysis

Condition	Slope	Test Type	Factor of Safety WES	Arc	DM No. 8
Construction	US	Q	2.47	1	1.7
Construction	DS	Q	1.97	2	--
Rapid Drawdown (el 498-461)	US	R	1.40	1	1.4
Critical Pool (el 448)	US	R	1.52	1	1.5
Steady Seepage	DS	R	1.34	2	1.4
Steady Seepage	DS	S	1.31	3	1.3

NOTE: All factors of safety were computed by the circular arc method. WES factors of safety were computed using shear strengths based on tests on record samples; factors of safety from DM No. 8 were computed using design shear strengths.

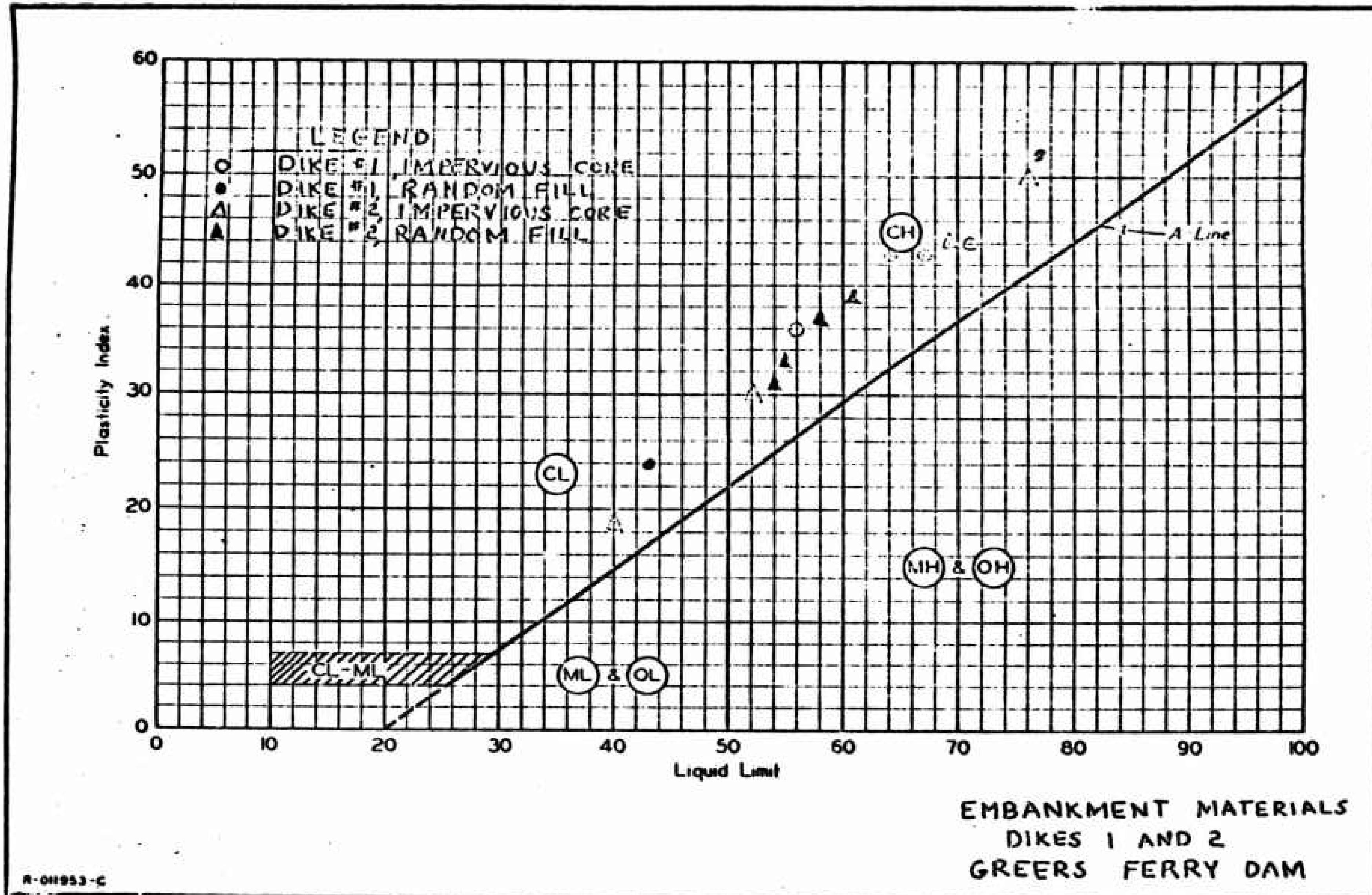
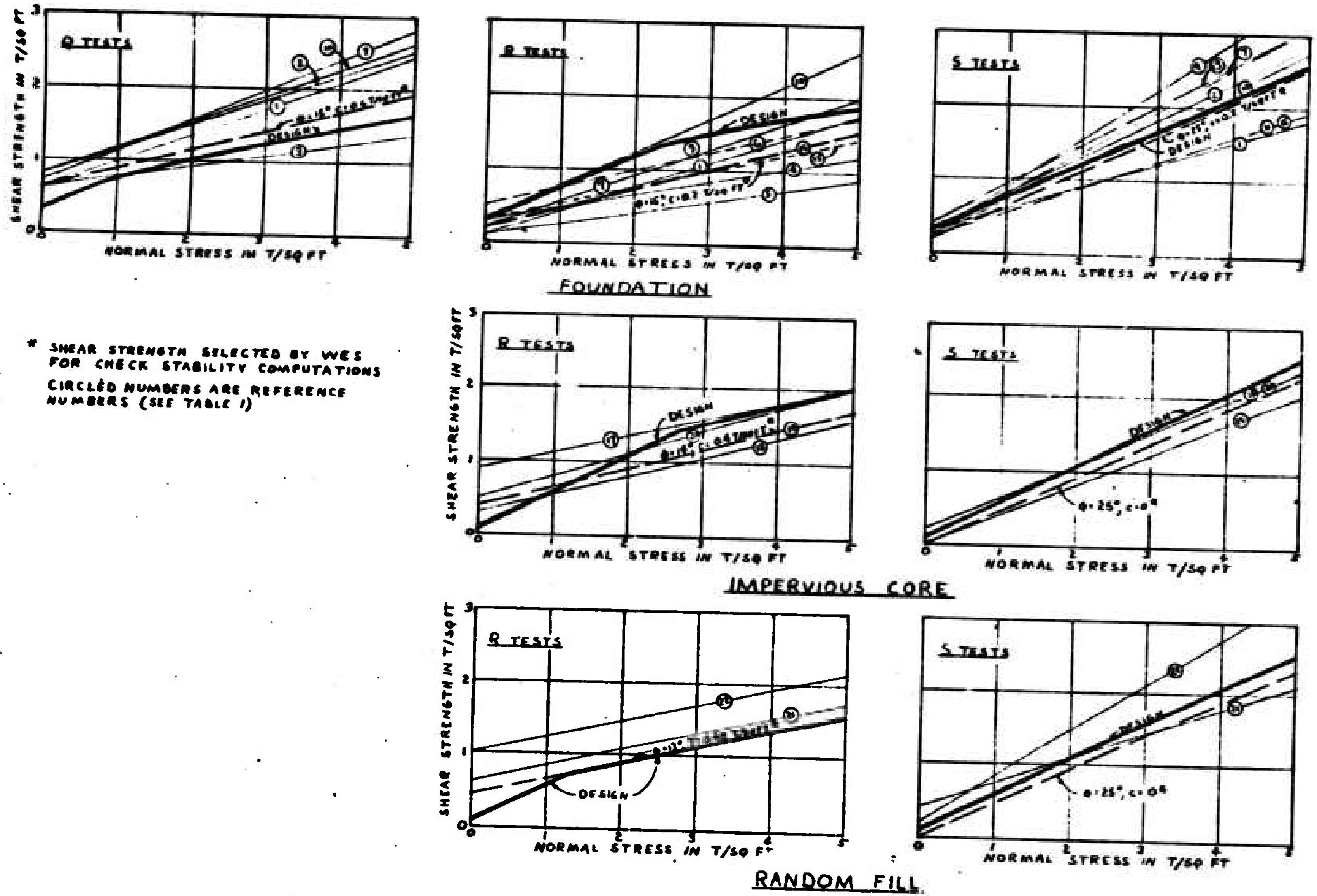


FIG. 1 - PLASTICITY CHART



* SHEAR STRENGTH SELECTED BY WES FOR CHECK STABILITY COMPUTATIONS

CIRCLED NUMBERS ARE REFERENCE NUMBERS (SEE TABLE I)

FIG.2 - RESULTS OF LABORATORY SHEAR STRENGTH TESTS ON RECORD SAMPLES, DIKE No. I

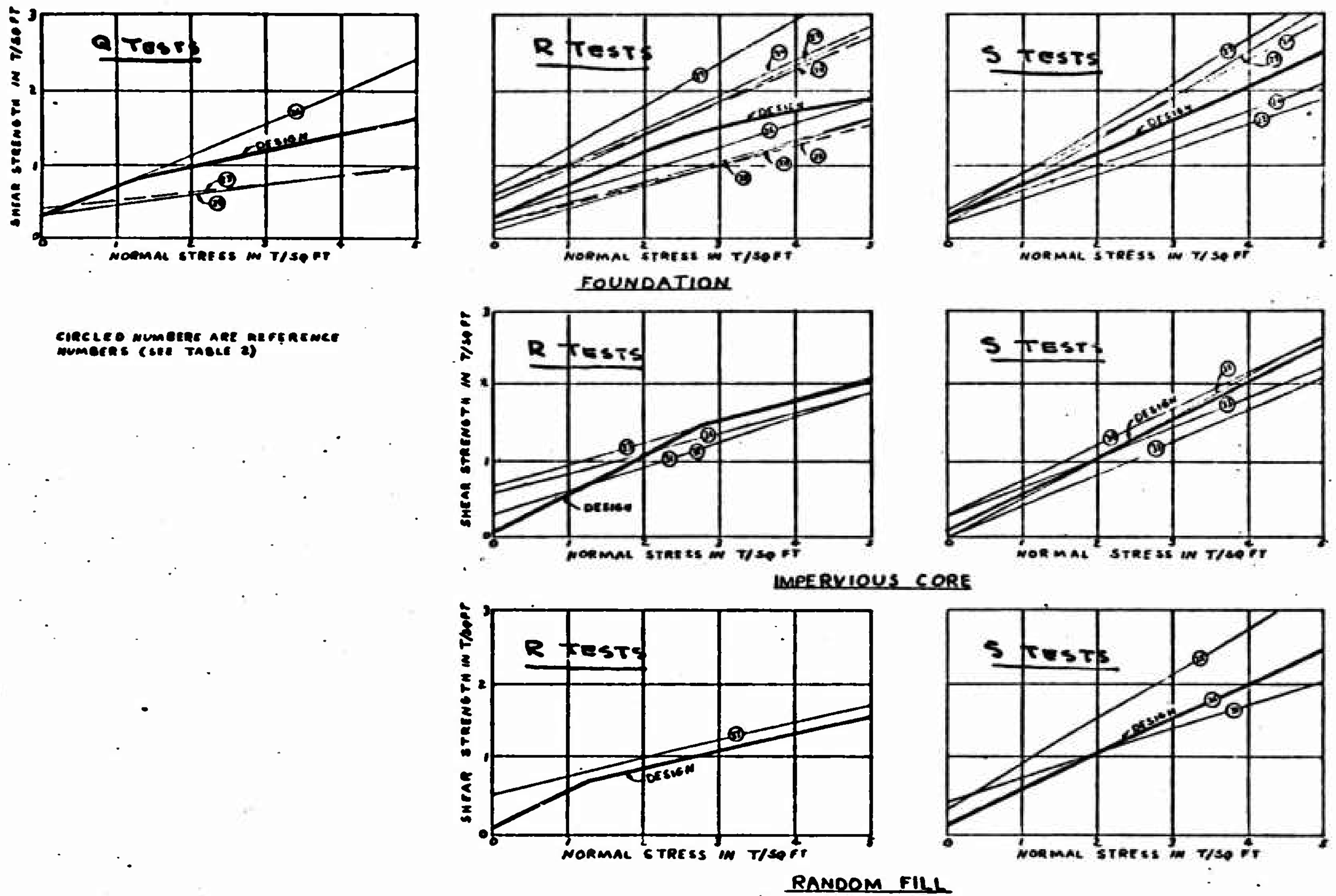
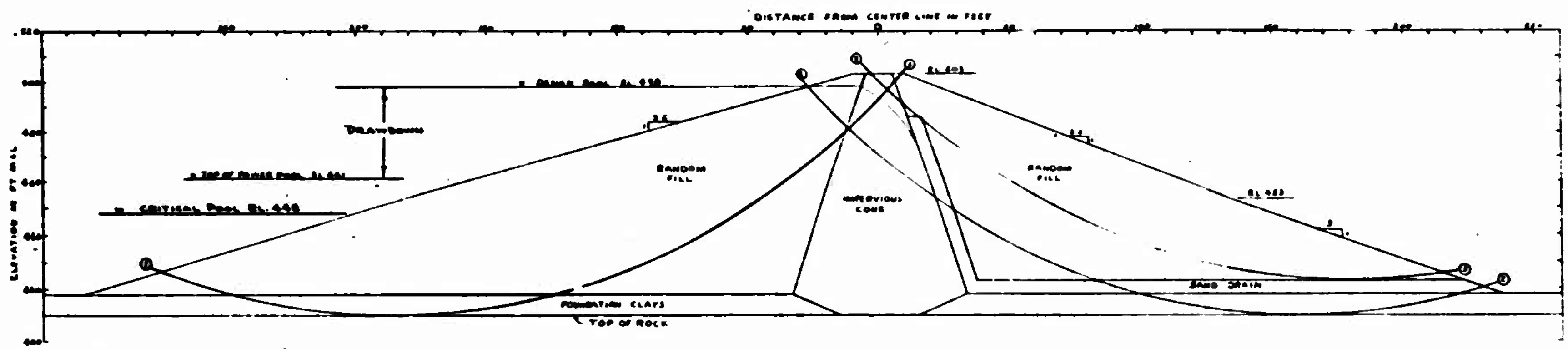


FIG. 3 - RESULTS OF LABORATORY SHEAR STRENGTH TESTS ON RECORD SAMPLES, DIKE No. 2



NOTE: ARC ① IS CRITICAL FOR CONSTRUCTION, CRITICAL POOL AND SUDDEN DRAWDOWN CONDITIONS FOR UPSTREAM SLOPE.
ARC ② IS CRITICAL FOR CONSTRUCTION, STEADY SEEPAGE (R TEST) CONDITIONS FOR DOWNSTREAM SLOPE.
ARC ③ IS CRITICAL FOR STEADY SEEPAGE (S TEST) CONDITION FOR DOWNSTREAM SLOPE.
FACTORS OF SAFETY FOR CRITICAL ARCS SHOWN IN TABLE 3.

FIG. 4. CRITICAL ARC FOR STABILITY ANALYSIS OF DIKE NO. 1.

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