AD-A149 941 COOPER RIVER REDIVERSION PROJECT LAKE MOULTRIE AND SANTEE RIVER SOUTH CAROLINA INTAKE AND TAILRACE CANALS (U) CORPS OF ENGINEERS CHARLESTON SC CHARLESTON DISTRICT JUN 76 1/5

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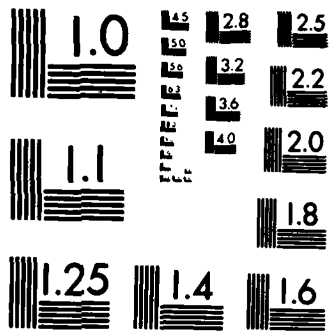
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STR FEST SECT 2

DESIGN MEMORANDUM NO. 9

COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER

SOUTH CAROLINA

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AD-A149 941

INTAKE AND TAILRACE CANALS

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U.S. ARMY ENGINEER DISTRICT, CHARLESTON

CORPS OF ENGINEERS

Charleston, South Carolina

JUNE, 1976

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SADEN-GP (2 Jul 76) 5th Ind
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

DA, South Atlantic Division, Corps of Engineers, 510 Title Building, 30 Pryor Street, S.W., Atlanta, Georgia 30303 17 May 1978

TO: District Engineer, Charleston, ATTN: SACEN-GF

Information furnished in 4th indorsement is satisfactory subject to the following comments:

a. The computations submitted by the 4th indorsement utilize a 2' projection of the base slab on each side of the U-frame structure. Plate 17 should be revised to show the required extension of the base slab.

b. Sheet 3. A more conservative and realistic uplift assumption would be based on head loss along the shortest flow path from point a to point h (i.e. straight lines a to f to g to h). Uplift at a point on the structure would be equal to the head at a point on the flow path directly under the point on the structure with appropriate adjustment for difference in elevation.

c. Sheet 4 of computations. P_{10} is indicated to be "0 due to 1 on 2 slope." Since this slope is steeper than the angle of internal friction ($\phi = 20^{o}$), if cohesion is neglected, P_{10} would not be zero, however it would be small and could reasonably be neglected. However, in calculating P_{13} the effect of the sloping backfill (1 on 2 slope) should be considered.

d. Sheet 5 of computations. In the last line "tan 65^{o}" should be "tan 55^{o}" ($\phi + 2 = 20^{o} + 35^{o} = 55^{o}$). This will result in about a 20% reduction in P_{18}.

e. Sheet 6 of computations. It appears that only 2' of the upstream wing projections were used in computing the passive resistance. Computations on sheet 2 indicate that the upstream wings project 5'. Additional passive resistance appears to be available by utilizing the 5' projection. However, development of full passive resistance at the upstream end of the structure will impose an internal tensil stress on the structure since most of the driving forces are applied on the downstream sloping portion of the structure. It appears that the first line portion of the computation of P_{20} should be

SACEN-GF (2 Jul 76) 4th Ind
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

DA, Charleston District, Corps of Engineers, P. O. Box 919, Charleston, South Carolina 29402 16 March 1978

TO: Division Engineer, South Atlantic, ATTN: SADEN-GP

The following provides information concerning Paragraph 1.f.:

Drop Structure D-1. The initial design computations that necessitated the increase of the approach walls and apron by approximately 15 ft. were based on an anticipated failure surface that could be kept within the apron area rather than the baffle area. Upon closer study, it was determined that the extra 15 feet were not needed to develop the failure surface within the apron. Therefore, we will use an 18.5 foot apron based on the attached computations.

FOR THE DISTRICT ENGINEER:

1 Incl (23 cys)
as

JACK J. LESEMANN
Chief, Engineering Division

SADEN-GP (2 Jul 76) 5th Ind 17 May 1978
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

(0.125)(2.04)(15.5)(2)(2) = 55.34 for 2' wing projections. It further appears that the Kp factor (2.04) is omitted in the first line portion of the computation for P_{21}. The second line or cohesion portion of P_{21} should be omitted, the effect of cohesion appears to have been fully accounted for in the previous terms.

f. In your computations, frictional resistance due to $\Sigma V \tan \phi$ has not been considered.

FOR THE DIVISION ENGINEER:

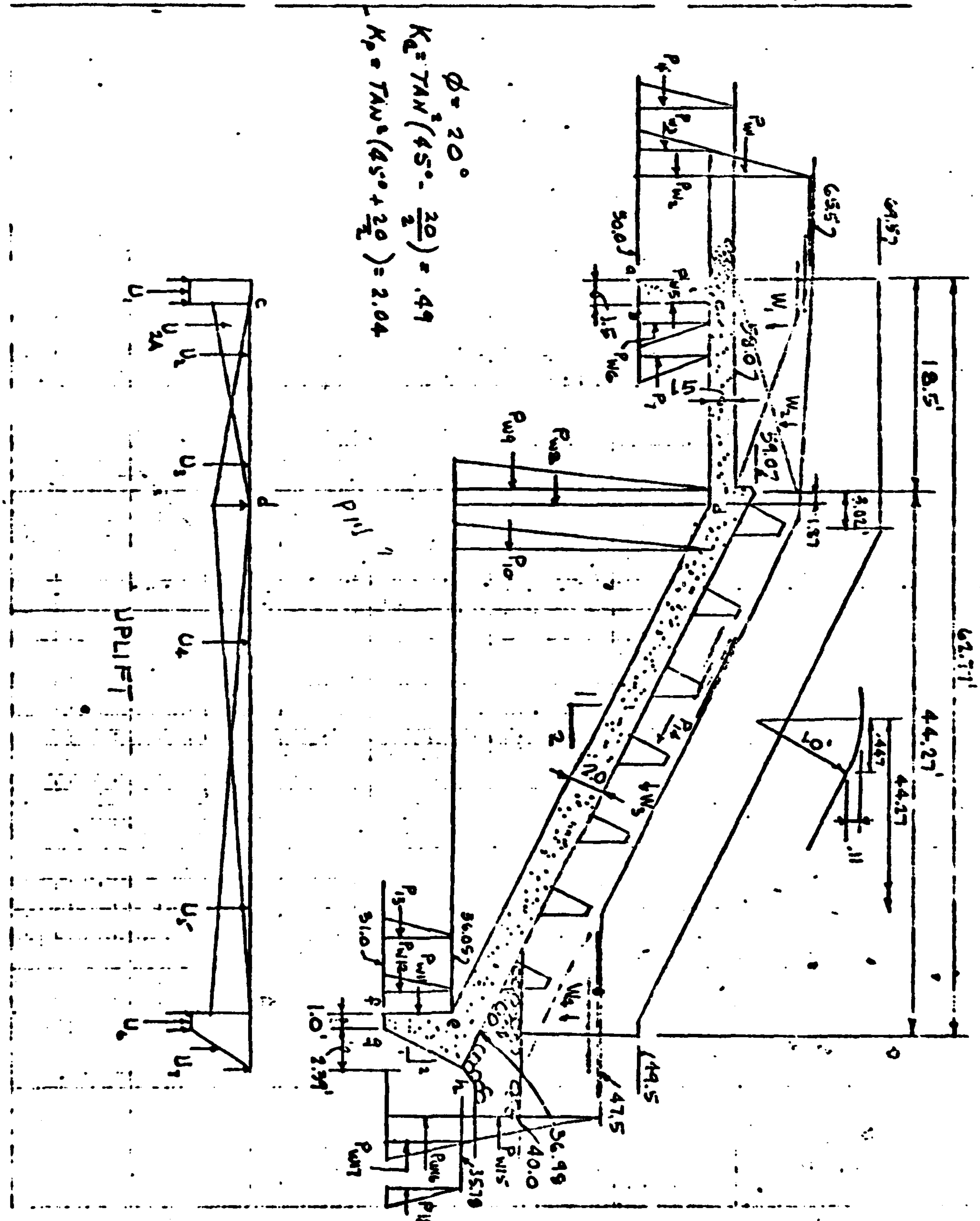
James Deene
for WILLIAM N. McCORMICK, JR.
Chief, Engineering Division

all incl wd

Copy furnished:
HQDA (DAEN-CWE-BB) w/10 cys incl 1

BY RIVERS DATE FEB 78 SUBJECT DROP STRUCTURE SHEET NO. 1 OF.

CHKD. BY ____ DATE ____ D1 JOB NO. ____



Sheet 1

BY RIVERS DATE FEB 78 SUBJECT D1 SHEET NO. 2 OF
CHKD. BY DATE JOB NO.

A. STABILITY & FDN. PRESSURES

1. CONCRETE:

	WT.	LEVER	MOMENT
APRON			
$18.5 \times 21.5 \times 1.5 \times .15 =$	89.49	9.25	827.82
$6.5 \times 1.5 \times 21.5 \times .15 =$	31.44	0.75	23.58
CHUTE			
$46.66 \times \frac{\sqrt{5}}{2} \times 21.5 \times 2 \times .15 =$	336.48	41.83	14,074.96
$1.0 \times 5 \times 21.5 \times .15 =$	16.13	62.27	1004.10
$\frac{2.5 \times 5}{2} \times 21.5 \times .15 =$	20.16	63.60	1281.94
WALLS			
$2 \times 18.5 \times 11.5 \times 1.25 \times .15 =$	19.51	9.25	180.44
$2 \times \frac{3.02 \times 11.5}{2} \times 1.25 \times .15 =$	6.51	19.51	826.93
$2 \times 10.5 \times \frac{46.12 + 49.50}{2} \times 1.25 \times .15 =$	140.29	42.15	5912.52
BAFFLES			
$14 \times \frac{.75 + 2.50}{2} \times 3.5 \times 3.75 \times .15 =$	44.79	42.15	1887.90
WINGS			
$2 \times 5 \times 1.25 \times 11.5 \times .15 =$	21.56	0.63	13.58
$2 \times \frac{3 \times 3}{2} \times 1.5 \times .15 =$	2.0	3.5	7.00
$2 \times 2 \times 1.25 \times 12.5 \times .15 =$	9.38	62.15	582.92
	737.74		26,623.69

$$\bar{x} = \frac{26,623.69}{737.74} = 36.09 > 20.92$$

$$< 41.85$$

2. CONDITION I - CONSTRUCTION COMPLETED; NO HYDROSTATIC HEAD

FOUNDATION PRESSURES

$$SP = P/A \left(1 \pm \frac{6e}{\ell}\right)$$

$$= \frac{737.74}{1350}\left(1 \pm \frac{6\,[31.39 - 36.09]}{62.77}\right)$$

$$= .546\ (1 \pm .45)$$

DISCHARGE = 995 PSF

APPROACH = 300 PSF

3. CONDITION II - FULL HYDRAULIC HEAD

HEADWATER EL. = 65.5
TAILWATER EL. = 47.5

NET HEAD = 18.0'

CREEP PATH a TO h

$1.5 + 6.5 + 18.87 + 46.29 + 5.05 + 1.0 + 5.34 = 84.55$

CREEP RATIO = $84.55/18.0 = 4.70$

SEEPAGE HEAD LOSS = 0.213 FT. PER FT.

PT.	CREEP DIST.	HD. LOSS	GRADIENT EL.	PT. EL.	UPLIFT HD.
a	0	0	65.5	50.0	15.5
b	1.5	0.32	65.18	50.0	15.18
c	8.0	1.70	63.8	56.5	7.3
d	26.87	5.72	59.78	56.5	3.28
e	73.16	15.58	49.92	36.05	13.87
f	78.21	16.66	48.84	31.0	17.84
g	79.21	16.87	48.63	31.0	17.63
h	84.55	18.00	47.5	35.78	11.72

BY RIVERS DATE FEB 78 SUBJECT D1 SHEET NO. 4 OF

CHKD. BY DATE JOB NO.

HORIZONTAL FORCES-ACTIVE

		H	LEVER	M
P_{W1} =	$\frac{.0625(7.5)^2(6.25)(2)}{2}$ =	+21.97	+2.5	+54.93
P_{W2} =	$.0625(7.5)(8)(27.5)$ =	+103.13	-4.0	-412.50
P_{W3} =	$\frac{.0625(8)^2(27.5)}{2}$ =	+55.0	-5.33	-293.15
P_4 =	$\frac{.49(.125-.0625)(8)^2(27.5)}{2}$ =	+26.95	-5.33	-143.64
P_{W5} =	$.0625(9)(6.5)(27.5)$ =	-100.55	-4.75	+477.60
P_{W6} =	$\frac{.0625(6.5)^2(27.5)}{2}$ =	-36.31	-5.83	+211.68
P_{W8} =	$.0625(3.28)(20.45)(21.5)$ =	+90.13	-11.73	-1057.26
P_{W9} =	$\frac{.0625(13.87-3.28)(20.45)(21.5)}{2}$ =	+145.50	-15.13	-2201.49
P_{10} =	0 DUE TO 1 ON 2 SLOPE	0	-	0
P_{W11} =	$.0625(13.87)(5.05)(21.5)$ =	+94.12	-24.48	-2304.08
P_{W12} =	$\frac{.0625(5.05)^2(21.5)}{2}$ =	+17.13	-25.31	-433.67
P_{13} =	$\frac{.49(.125-.0625)(5.05)^2(21.5)}{2}$ =	+8.40	-25.31	-212.50
P_{14} =	(BAFFLES) = $\frac{2}{15} \times .28(3.5)(3.75)14$ =	+46.02	-11.73	-539.56
P_{W15} =	$\frac{.0625(11.72)^2(21.5)(.60\ EFF.)}{2}$ =	-55.37	-18.3	+1013.27
P_{W16} =	$.0625(11.72)(4.78)(21.5)$ =	-75.28	-24.61	+1852.64
P_{W17} =	$\frac{.0625(4.78)^2(21.5)(.60\ EFF.)}{2}$ =	-9.21	-25.[illegible]	+231.20
LOWER WALLS				
P_{W22} =	$.0625(11.72)^2(2)(2)$ =	+34.34	-18.3	-628.87
P_{23} =	$.49(.125-.0625)(1.72)^2(2)(2)$ =	+16.83	-18.3	-307.92
	TOTAL	+382.80		-4693.32

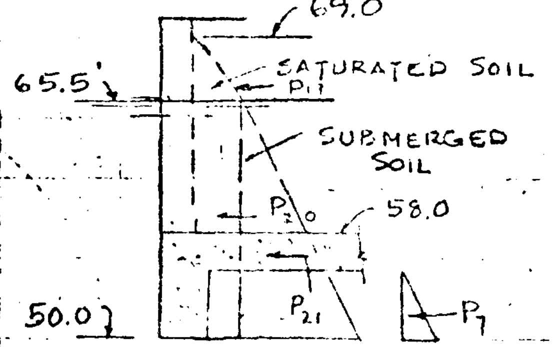
HORIZONTAL FORCES – PASSIVE – REF. ETL 1110-2-184

COHESION = C = 0.2 K/SF
INTERNAL FRICTION = ϕ = 20°

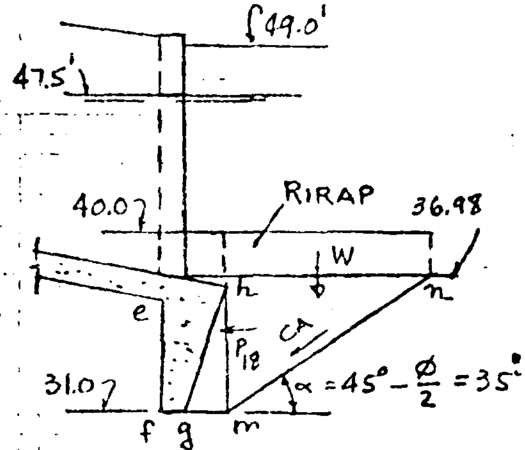
$$P_p = \frac{\gamma H^2}{2} \tan^2\left(45° + \frac{\phi}{2}\right) + 2CH \tan\left(45° + \frac{\phi}{2}\right)$$

$$\tan\left(45° + \frac{\phi}{2}\right) = \tan\left(45° + \frac{20°}{2}\right) = 1.42$$

$$\tan^2\left(45° + \frac{\phi}{2}\right) = (1.42)^2 = 2.04$$



UPSTREAM WALL AND KEY



DOWNSTREAM WALL AND KEY

DOWNSTREAM KEY

$$A = \text{PATH } g\,m\,n = 5.98/2 + \frac{5.98}{\sin 35°} = 13.42$$

$$hn = 5.98 \cot 35° = 8.54'$$

$$W = 8.54 \times \frac{5.98}{2} \times (.125 - .0625) + 8.54 \times 3.02 \times (.125 - .0625) = 3.21$$

$$P_{18} = \frac{CA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} + W \tan(\phi + \alpha)$$

$$= \frac{.2 \times 13.42}{\cos 35°(1 - \tan 20° \tan 35°)} + 3.21 \tan 65° = 11.28$$

BY RIVERS DATE FEB 78 SUBJECT D1 SHEET NO. 6 OF

CHKD. BY DATE JOB NO.

HORIZONTAL FORCES - PASSIVE (CON'T.)

	H	LEVER	M
$P_{17} = \frac{(.125-.0625)(6.5)^2(2.04)(17.5)}{2}$ =	-47.14	-5.83	$+274.83$
$+\ 2(0.2)(6.5)(1.42)(17.5)$ =	-64.61	-5.83	$+376.68$
$P_{19} = \frac{(.125)(3.5)^2(2.04)(2)(2)}{2}$ =	-6.25	$+8.67$	-54.19
$+\ 2(0.2)(3.5)(1.42)(2)(2)$ =	-7.95	$+8.67$	-68.93
$P_{20} = (.125-.0625)(6.5)(15.5)(2)(2)$ =	-25.19	-0.25	$+6.30$
$+\ 2(0.2)(15.5)(1.42)(2)(2)$ =	-35.22	-0.25	$+8.81$
$P_{21} = \frac{(.125-.0625)(15.5)^2(2)(2)}{2}$ =	-30.03	-2.83	$+84.99$
$+\ 2(0.2)(15.5)(1.42)(17.5)$ =	-154.07	-2.83	$+436.02$
P_{18} = (SEE SH 5) = 11.28×21.5 =	-242.52	-23.01	$+5581.19$
TOTAL	-617.98		$+6645.7$

PRORATE PASSIVE RESISTANCE SO THAT IT EQUALS ACTIVE PRESSURE.

$$382.80 / 612.97 = .624$$

$$.624 \times 6645.7 = +4150.2$$

BY RIVERS DATE FEB 18 SUBJECT D1 SHEET NO. 7 OF

CHKD. BY DATE JOB NO.

UPLIFT

	V	LEVER	M
$U_1 = .0625(15.34)(1.5)(27.5) =$	-39.55	0.75	-29.66
$U_2 = \frac{.0625(7.3)(18.87)(21.5)}{2} =$	-92.55	7.79	-720.98
$U_3 = \frac{.0625(3.28)(18.87)(21.5)}{2} =$	-41.58	14.08	-585.51
$U_4 = \frac{.0625(3.28)(41.4)(21.5)}{2} =$	-91.24	34.17	-3117.51
$U_5 = \frac{.0625(13.87)(41.4)(21.5)}{2} =$	-385.80	47.97	$-18,506.96$
$U_6 = .0625(17.74)(1.5)(21.5) =$	-35.76	62.27	-2226.60
$U_7 = .0625(14.68)(2.39)(21.5) =$	-47.15	63.97	-3015.91
$U_{2A} = \frac{.0625(7.3)(3)(3)(2)}{2} =$	-4.11	2.5	-10.27
	-737.74		$-28,213.40$

VERTICAL LOADS

	V	LEVER	M
$W_1 = \frac{.0625(7.5)(18.5)(15)}{2} =$	65.04	6.17	401.29
$W_2 = \frac{.0625(5\frac{1}{2})(18.5)(15)}{2} =$	43.36	12.33	534.62
$W_3 = .0625[4(46.66)(15)(.60\text{ EFF.})-300] =$	86.24	41.83	3607.21
$W_4 = \frac{.0625(6.05)(12.1)(15)(.60\text{ EFF.})}{2} =$	20.59	61.13	1258.60
WATER & EARTH OUTSIDE OF WALLS:			
$W_5 = (2)(2)(60.27)(4)(.0625)$	60.27	31.39	1891.57
$W_6 = (2)(2)(.125-.0625)(12)(62.77) =$	188.31	31.39	5911.05
WATER FORCE ON BAFFLES (SEE SH. 4) $\frac{1}{\sqrt{5}} \times .280(3.5)(3.75)14 =$	23.01	37.6	864.22
TOTAL	486.8		14,486.6

BY RIVERS DATE FEB 78 SUBJECT D1 SHEET NO. 8 OF

CHKD. BY DATE JOB NO.

SUMMARY OF CONDITION II

LOADS	HORZ.	VERT.	MOM.
HORZ. ACTIVE	+382.8		-4693.3
HORZ. PASSIVE	-382.8		+4150.2
UPLIFT		-737.7	-28,213.4
VERTICAL		+486.8	+14,486.6
CONCRETE		+737.7	+26,623.7
		+486.8	+12,353.8

$$x_H = 12,353.8 / 486.8 = 25.38 > 20.92$$
$$< 41.85$$

FACTOR OF SAFETY AGAINST SLIDING

$$FS = 612.98 / 382.8 = 1.60$$

FACTOR OF SAFETY AGAINST UPLIFT

$$FS = \frac{486.8 + 737.7}{737.7} = 1.66$$

FACTOR OF SAFETY AGAINST OVERTURN

$$FS = \frac{4150.2 + 14,486.6 + 26,623.7}{4693.3 + 28,213.4} = 1.38$$

$$\text{SOIL PRESS.} = \frac{486.8}{1350}\left(1 \pm \frac{6[25.38 - 31.39]}{62.77}\right)$$
$$= .361\ (1 \pm .57)$$

DISCHARGE = 566 PSF

APPROACH = 155 PSF

SADEN-GP (2 Jul 76) 3rd Ind
SUBJECT: Cooper River Rediversion Project Lake Moultrie and
Santee River, South Carolina - Intake and Tailrace Canals
DM No. 9

DA, South Atlantic Division, Corps of Engineers, 510 Title Building,
30 Pryor Street, S. W., Atlanta, Georgia 30303 25 January 1978

TO: District Engineer, Charleston, ATTN: SACEN-GP

Information furnished is satisfactory subject to the following comment:

Paragraph 1.f. The need for increasing the approach walls and apron approximately 15 feet in order to obtain full passive resistance on the increased key depth is not readily apparent; backup computations should be furnished. Prestressed foundation anchors should also be considered as an alternative for increasing the sliding resistance of the drop structure.

FOR THE DIVISION ENGINEER:

James D. Wace

Incl wd

for WILLIAM N. McCORMICK, JR.
Chief, Engineering Division

Copy Furnished:
HQDA (DAEN-CWE-BB)
w/10 cys Incl

SADEN-GK (2 Jul 76) 1st Ind
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

DA, South Atlantic Division, Corps of Engineers, 510 Title Building, 30 Pryor Street, S. W., Atlanta, Georgia 30303 17 December 1976

TO: District Engineer, Charleston ATTN: SACEN-G

1. The Intake and Tailrace Canals DM is approved subject to the following comments:

a. The DM should contain a section which briefly discusses the environmental aspects of the project. The status of the environmental statement should be presented in the discussion (date of filing with CEQ, etc.). It should also be noted that the present detailed studies do not include any significant variation from those impacts described in the EIS.

b. Page 1, paragraph 1. In the last line, change the # sign to a $ sign.

c. Page 34, paragraph 93.b. Retaining Walls EM 1110-2-2502 should be referenced.

d. Page 34, Paragraph 93.e. EM 1110-2-2502 requires U-frame design for at-rest pressure. Suggest the economics of a cantilevered walls versus a U-frame wall be investigated.

e. Page 34, paragraph 93.f. The assumed values for the drop structure should be replaced by the soil values determined from site borings and type of backfill as soon as they are available.

f. Page 35, paragraph 93.g. It is doubtful that the downstream riprap would remain in place during a major flow. Sliding stability should be analyzed without passive resistance at downstream key.

g. Page 37, paragraph 99. Roman numerals IV and V suggest Modified Mercalli scale not Richter scale. Reference should be corrected.

h. Page 43, paragraph 116. In the consideration of constructing the bridges prior to canal construction, sufficient data should be presented in DM No. 10 to evaluate the effects of horizontal or vertical movements on the bridge piers as a result of excavation.

SADEN-GK (2 Jul 76) 1st Ind 17 December 1976
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

i. Plate 16. The riprap blanket on both the channel bottom and side slopes of the outlet channels for the three drop structures should be extended to provide a total length of at least 50 feet to prevent undermining the structures. The upstream approach channels should be riprapped at least throughout the side slope transition. The outlet channel geometry for structures D-2 and D-3 should be modified so as to be similar to structure D-1; i.e., they should have a 20-foot horizontal section just downstream of the chute before transitioning up to meet the channel bottom.

2. The date you expect to respond should be furnished SADEN-GK by 5 January 1977.

FOR THE DIVISION ENGINEER:

1 Incl
wd 11 cys

WILLIAM N. McCORMICK, JR.
for Chief, Engineering Division

Copy furnished:
HQDA (DAEN-CWE-B)
w/10 cys Incl 1

SACEN-GP (2 Jul 76) 2nd Ind
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

DA, Charleston District, Corps of Engineers, P. O. Box 919, Charleston, South Carolina 29402   5 December 1977

TO: Division Engineer, South Atlantic, ATTN: SADEN-GK

1. The following are in reference to SADEN-GK 1st Indorsement dated 17 December 1976, subject as above.

a. Paragraph 1a. Concur. Revised page 13 and new page 13a are inclosed.

b. Paragraph 1b. Concur. Revised page 1 is inclosed.

c. Paragraph 1c. Concur. Revised page 34 is inclosed.

d. Paragraph 1d. Cantilevered walls have been investigated and compared with a U-frame designed for at-rest pressure. Due to the low height of the structure and high saturation level, the at-rest pressures do not change the required concrete thicknesses. The amount of reinforcing does increase slightly but the addition of base slab heels for the cantilever walls increases the concrete quantity required and also increases the area to be excavated and backfilled during construction. It is recommended that the U-frame design be retained.

e. Paragraph 1e. Concur. Two borings have been drilled at each drop structure site to confirm the soil and rock materials available for the foundation and backfill for the structures. Soil values for final design will be assigned based on laboratory testing of project soil and rock types.

f. Paragraph 1f. Please refer to Plate 17 and Appendix E. If the passive resistance on the downstream key is neglected, the upstream key depth must be increased to elevation 44.5 to maintain stability against sliding. Also, the approach walls and apron would have to be extended upstream about 15 feet in order to obtain full passive resistance on the increased key depth. This results in a considerable increase in structure costs. It is believed that a better design can be obtained by sizing the downstream riprap to make certain it will remain in place during major floods. A partially pre-formed 3-foot deep scour hole has been provided with a 3-foot thick riprapped base. Although the discharge velocities cannot be accurately determined for this type structure, they are expected to be less than 10 fps. A 3-foot blanket designed in accordance with ETL 1110-2-120 should safely withstand these velocities and permit development of full passive resistance at the

SACEN-GP (2 Jul 76) 2nd Ind 5 December 1977
SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

lower key. Calculations shown in the Design Memorandum Appendix E were brief and for the purpose of preliminary sizing. More refined computations indicate that both the upper and lower keys should be deepened by two feet and the approach walls and apron should be extended some six feet farther upstream.

g. Paragraph 1g. Concur. Revised page 37 is inclosed.

h. Paragraph 1h. Concur.

i. Paragraph 1i. Concur with first and second sentence. In regard to outlet channel geometry for structures D2 and D3, it is expected that both structures will be on rock; therefore, it is believed that no horizontal section is necessary. However, if future borings do not indicate rock, the outlet channel geometry will be revised to conform with D1.

2. Additional well inventory data is furnished for Appendix F. Remove and destroy existing Appendix F and replace with inclosed Appendix F. This completes the inventory of wells expected to be affected by project construction.

3. Revised Plates 8, 9, and 14 showing the intake canal levee alignment changes between Stations 320+00 and 339+60 are inclosed. These changes have caused some alterations in the disposal areas between Stations 320+00 and 339+60 and are reflected in the material use chart on revised page 25. Appendix G including Plate G-1 is added to show the revised intake canal levees in the vicinity of the powerhouse whose design was previously included in DM No. 7, Preliminary Design Report, St. Stephen Powerplant, 15 August 1976. A complete discussion of the intake canal levee realignment is covered in the revisions to the Powerhouse Foundation Analysis Report, February 1976. Revised pages IV, VII, VIII, 1, 2, 13, 13a, 14, 25, 26, 33, 34, 37, 38, 43, and 44 are inclosed. Boring logs of additional levee foundation investigations for Appendix A and revised pages VII and VIII of Appendix B are also inclosed.

FOR THE DISTRICT ENGINEER:

JACK A. LESEMANN
Chief, Engineering Division

2 Incls
Incl 1 w/d
Added 1 incl
2. Revisions with Ind (23 cys)

DEPARTMENT OF THE ARMY
CHARLESTON DISTRICT, CORPS OF ENGINEERS
P O BOX 919
CHARLESTON, S.C. 29402

SACEN-G 2 July 1976

SUBJECT: Cooper River Rediversion Project, Lake Moultrie and Santee River, South Carolina - Intake and Tailrace Canals DM No. 9

Division Engineer, South Atlantic
ATTN: SADEN-GK

1. Transmitted are 23 copies of the subject Intake and Tailrace Canals DM No. 9, submitted for approval in accordance with applicable provisions of ER 1110-2-1150, dated 1 October 1971, as revised by changes up to Change 7 dated 22 July 1974.

2. A draft of this design memorandum was reviewed by hydrology, hydraulics and soil mechanics representatives from your staff in conference at SAC on 14-15 April 1976. Their comments and suggestions made at the draft review conference have been incorporated in this final report.

3. A permit application is presently being prepared for this project pursuant to 33 CFR 209.145, Federal Projects Involving the Disposal of Dredged Material in Navigable and Ocean Waters (Final Regulations in Federal Register 22 July 1974). The entire project will be included in the scope of the permit application. Public notice is planned for mid-July.

1 Incl (23 copies)
fwd sep

HARRY S. WILSON, JR.
Colonel, Corps of Engineers
District Engineer

DALE P. GREGG
Lt Colonel, Corps of Engineers
Deputy District Engineer

COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

DESIGN MEMORANDUM 9

INTAKE AND TAILRACE CANALS

U. S. ARMY ENGINEER DISTRICT, CHARLESTON

CORPS OF ENGINEERS

CHARLESTON, SOUTH CAROLINA

This Design Memorandum on Intake and Tailrace Canals is submitted in accordance with applicable provisions of ER 1110-2-1150. It is the ninth of a series covering project studies for the Cooper River Rediversion Project.

Title	Date Submitted	Design Memorandum No.
General Design Memorandum	Jan 72	1
General Design Memorandum, Supplement No. 1, Comparison of Alternative Plans	Oct 73	1
Turbines, Governors, and Generators	Jun 73	2
Entrance Channel In Lake Moultrie	Mar 74	3
Access Roads and Construction Facilities	May 74	4
Real Estate, Area 1	Sep 74	5
Site Selection and Geology	May 75	6
Preliminary Design Report - Powerplant	Jan 76	-
Powerhouse Foundation Analysis	Feb 76	-
Relocation of Seaboard Coast Line Railroad Bridge	Jun 76	8
Intake and Tailrace Canals	Jul 76	9

COOPER RIVER REDIVERSION PROJCT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

SCHEDULE FOR SUBMISSION OF FUTURE DESIGN MEMORANDUM

Title	Scheduled Submittal Date
Real Estate, Area 2	Nov 1976
Feature Design - Powerplant, Switchyard	Apr 1977
Water Quality Studies Work Plan	Apr 1977
Construction Materials	Apr 1977
Fish Hatchery	Jun 1977
Utilities Relocation	Jul 1977
Primary and Secondary Road Relocation	Jul 1977
Cooling Water System	Jul 1977
Water Quality Monitoring Equipment	Nov 1977
Instrumentation	Mar 1981

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS DM

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COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

DESIGN MEMORANDUM NO. 9

INTAKE AND TAILRACE CANALS

PERTINENT DATA

DRAINAGE AREA	Square miles
Lake Moultrie	15,000
Lake Marion	14,700
RESERVOIR CAPACITY	Acre-feet
Maximum power pool	
Lake Moultrie	1,110,000
Lake Marion	1,450,000
Minimum power pool	
Lake Moultrie	450,000
Lake Marion	350,000
ELEVATIONS	Feet, msl
Top of dam	
Lake Moultrie	88.0
Lake Marion	88.0
Maximum water surface	
Lake Moultrie	75.2
Lake Marion	76.8
Top of gates	
Lake Moultrie	--
Lake Marion	76.8
Spillway crest	
Lake Moultrie	--
Lake Marion	63.0
Maximum power pool	
Lake Moultrie	75.2
Lake Marion	75.7
Minimum power pool	
Lake Moultrie	60.0
Lake Marion	60.0
Normal tailwater	
Lake Moultrie	7.2
Lake Marion	27.0
Minimum tailwater	
Lake Moultrie	-1.5
Lake Marion	26.0

PERTINENT DATA (Cont'd)

WILSON DAM (Forms Lake Marion	
Completion date	23 March 1942
Length - miles	7.8
Height of spillway - feet	48
Spillway	
Design capacity - cfs	800,000
Length - feet	3,400
Gates	
Number	62
Size - feet	14 x 50
INTAKE AND TAILRACE CANALS	
Canal length - miles	9.4
Intake canal invert elevation - msl	50
Tailrace canal invert elevation - msl	0.0
Maximum operating tailwater elevation - msl	23.1
Maximum discharge - cfs	24,500
Maximum intake canal velocities - fps	3.2
Maximum Tailrace canal velocities - fps	7.6
Canal bottom width - feet	285
Canal side slopes	1 vertical to 3 horizontal
ENTRANCE CHANNEL IN LAKE MOULTRIE	
Channel length - feet	13,534
Channel invert - to station 39+34 - msl	64
Channel width - to station 89+34 - feet	1,500
Channel invert - from station 115+34 - msl	54
Channel width - from station 115+34 - feet	385
Maximum discharge - cfs	24,500
Maximum channel velocity - fps	3
Channel vertical to 3 horizontal	
EXCAVATION QUANTITIES	
Entrance channel	2,780,000 CY
Intake and tailrace canals	15,336,000 CY
CONSTRAINTS IN COOPER RIVER TO LAKE MOULTRIE	
Strawberry Landing railroad bridge - width - feet	33
Lock size at Pinopolis Dam - feet	60 X 180
Average channel depth - feet	25
Average channel width - feet	300
ACCESS ROADS	
Powerhouse access road (length to be constructed) - miles	0.78
Tailrace access road (length to be constructed) - miles	0.74

COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

DESIGN MEMORANDUM 9

INTAKE AND TAILRACE CANALS

INTRODUCTION

1. Authorization. The Cooper River Rediversion Project, which will reduce shoaling and restore the historic saline regimen to Cooper River and Charleston Harbor, was authorized by the River and Harbor Act of 1968 (P.L. 90-483, 90th Congress, S. 3710, August 13, 1968). Section 101 of the 1968 Act is quoted in part as follows:

> ".....That the following works of improvement of rivers and harbors and other waterways for navigation, flood control and other purposes are hereby adopted and authorized to be prosecuted under the direction of the Secretary of the Army and supervision of the Chief of Engineers, in accordance with the plans and subject to the conditions recommended by the Chief of Engineers, in the respective reports hereinafter designated..... Cooper River, Charleston Harbor, South Carolina: Senate Document Numbered 88, Ninetieth Congress, at an estimated cost of $35,381,000...."

2. Purpose. The purpose of this design memorandum is to present an analysis of the physical characteristics, performance criteria and construction considerations upon which the design of the Intake and Tailrace Canals and the interior drainage facilities shall be based.

3. Scope. The data presented in this design memorandum on the selection of Intake and Tailrace Canals and interior drainage facilities are based on optimum development of economic power potential of Lake Moultrie when discharge through the existing Jeffries Hydro Plant at Pinopolis is reduced to 3,000 cfs average release.

Included in the canal design data are the determination of the canals alignment and configuration, subsurface exploration data, hydrology and hydraulic analysis, excavation quantities, an excess excavated material disposal plan, and interior drainage facilities design for the Intake and Tailrace Canals. Preliminary canal design was co-ordinated with other interested agencies, particularly those concerned with excavation and disposal of materials, e.g., U. S. Fish and Wildlife Service and South Carolina Wildlife and Marine Resources Department.

4. Project description. A serious silt and shoaling problem developed in Charleston Harbor subsequent to 1942 when silt-laden fresh water of Santee River was diverted through Pinopolis hydro powerplant, down the Cooper River and into the salt water of the harbor. This generates density currents in the harbor which trap sediments until deposited. The project will provide for rediversion of most of the Santee River waters from above Pinopolis Dam into the Santee River through a proposed canal about 11.7 miles in length. The canal would begin at the northeast corner of Lake Moultrie and proceed generally eastward to a proposed hydroelectric plant just north of St. Stephen, South Carolina, then continue on to an intersection with the Santee River at Mattassee Lake. The 84,000 kilowatt hydroelectric plant would generate power using the rediverted flow in the canal and compensate for the limitation in flow and loss of power at the existing Pinopolis plant (Jeffries Hydro Plant) owned and operated by the South Carolina Public Service Authority. The plan provides for fish and wildlife facilities, including a fish lift at the new powerplant and a replacement fish hatchery on the canal bank below the powerplant.

There is no requirement of local cooperation, and the costs to the United States shall not include betterments to others arising from the increase in capacity provided by the new power facility. The Secretary of the Army, acting thru the Chief of Engineers, is authorized to determine and enter into agreement with South Carolina Public Service Authority, or its successors, in interest, for apportionment of costs between the United States and the South Carolina Public Service Authority.

5. Related reports. The design of the Intake and Tailrace Canals presented in this report is one component of the overall project design. The design of related project features is presented in the following completed design reports:

Entrance Channel DM No. 3	March 1974
Access Roads and Construction Facilities DM No. 4	May 1974

Site Selection and Geology DM No. 6	May 1975
Powerhouse Preliminary Design Report	January 1976
Powerhouse Foundation Analysis	February 1976
Groundwater Report	October 1975

SUBSURFACE INVESTIGATIONS

6. Previous investigations. Locations of borings are shown on Plate 18. Logs of borings are presented in Appendix A, Geology and Soils. Borings have been taken in the area of the Intake and Tailrace Canal as follows:

a. 1965. In June and July, 1965, nine borings, CS-1 through CS-9 were drilled for the Project Document Plan along the canals alignment later shown in the GDM. Due to dense timber and swamp, the borings for the tailrace canal were offset up to 2,600 feet south of the GDM canal centerline. The borings were advanced by Standard Penetration Test equipment (1-3/8" I.D. split-spoon sampler, 30-inch drop, 140 lb. hammer) and techniques through soil, and were cored with a 4 x 5½" rock bit below refusal.

b. 1970. Between October 1970 and May 1971 thirty borings (CS-11 through CS-27) were drilled along the proposed GDM intake and tailrace canals alignment, known as the "base line", to better define the material to be excavated from the canals. Piezometers were installed at the locations of CS-18, CS-20, CS-21, CS-24 (soil and rock), CS-25 and CS-27 (soil and rock). The continuously sampled borings were advanced by Standard Penetration Test equipment and techniques through soil and were cored below refusal with a 4 x 5½" rock bit. Soils were field classified according to the Unified Soil Classification System. Undisturbed samples were obtained with Shelby tube equipment in selected borings for laboratory strength testing.

c. 1972. Between June and December, 1972, fifty-one borings were drilled throughout the project site area to establish site geology, regional groundwater and tailrace canal top of rock conditions. Twenty-four split-spoon/cored borings (GS-1 through GS-24) were drilled adjacent to the GDM canals alignment as the initial phase of the U. S. Geological Survey's project groundwater study. The GS-series borings were advanced through soil by Standard Penetration Test techniques and were continuously sampled. Rock bits 4 x 5½, or greater, in size were used to core below drive sample refusal. In the tailrace canal area five continuously sampled borings (T-1, 2, 3, 6 and 8) were drilled to better define top of rock in the Santee River Flood Plain using the same drilling equipment and procedures as the GS-series borings. Supplementary information on location of top of rock in the general flood plain was obtained by twenty-two probe borings (TA-1 through T-22) advanced by 5-inch auger, NX fishtail and 5-5/8" roller bit through soil and into geologic top of rock. Rock in TA-1 and TA-2 was cored with a 4 x 5½" core barrel. Tailings were field classified to establish approximate top of rock. Known rock outcrops in the tailrace exit (Lake Mattassee/Santee River junction) were mapped and several cross sections were probed to refusal (assumed top of rock) with hand-driven rods during 1972.

d. 1973. The final subsurface investigation phase of the USGS groundwater study was completed in June 1973 with the installation of twenty observation wells throughout the project site (see Plate 18). The wells were installed using reverse rotary techniques, revert drilling mud, air cleaning of the hole, tremie sand filter placement, around 6-inch diameter plastic screens, and thick cement grout seals at top of rock. After surging and developing, rudimentary pump tests were run on the wells by USGS.

7. Investigations for this report. Boring locations are shown on Plate 18. In 1974 twenty-two probe borings (A-1 through A-22) were taken in the tailrace canal area to better define the location of top of rock for preliminary canal alignment studies. The borings were advanced to refusal with a 5-inch diameter auger (helical or square) in two-foot increments. Samples were taken from auger tailings in borings A-1, A-2, A-3, A-4 and A-14. In 1975 seventy-eight continuously sampled, split-spoon drive borings were drilled in the intake canal (IT-1 through 28) and tailrace canal (T-9 through 34A) areas to determine types and strengths of cut slope and excavation materials at the selected canal alignments. The borings were advanced by Standard Penetration Test techniques through soil and were cored below refusal with a 4 x 5½ rock bit. The soil samples were visually classified in the field by trained soils inspectors according to the Unified Soil Classification System. Observation wells were established in borings T-11 and IT-2B-1 by sealing 6-inch casing at the top of each boring to measure the artesian water level encountered in these borings.

8. Future subsurface investigations.

a. Intake canal. Approximately ten to sixteen continuously sampled split-spoon/core borings are planned in the intake canal prior to plans and specs preparation to better define (1) properties of materials within excavation limits and (2) geologic stratigraphy of rock formations between Lake Moultrie and the powerhouse. A test pit is planned near intake canal station 290+00 to establish optimum equipment and unit excavation costs for excavation of limestone rock within the canal limits.

b. Tailrace canal. Approximately ten continuously sampled split-spoon/core borings and approximately ten probe (auger) borings are planned in the tailrace canal to better define properties of materials within excavation limits. Test excavations are planned for at least two locations in the tailrace canal to establish optimum equipment and unit excavation costs for excavation of sandstone and shale rock within the canal limits. Locations of the test pits in the tailrace canal are tentatively selected at the SCL Railroad bridge relocation site (Station 420+00) and at Lake Mattassee's junction with the Santee River (Station 595+00).

c. Interior drainage structures. A split-spoon/core boring is planned for each of three major interior drainage concrete drop structures presented in this report. The borings will be drilled to better define excavated materials and foundation properties.

GEOLOGIC INVESTIGATIONS

9. Powerhouse investigations. Details or previous geologic investigations in the powerhouse area are discussed in the powerhouse Site Selection and Geology DM No. 6, May 1975, and the powerhouse Foundation Design Report, March 1976, including sources of geologic information and a top of rock map of the Powerhouse area.

10. Previous canal and levee geologic investigations. Shallow borings were taken in Lake Moultrie in 1973 for design of the entrance channel for the intake canal. Geologic conditions in the lake area were presented in the Entrance Channel in Lake Moultrie DM No. 3, March 1974, including a top of rock map. The deep groundwater study borings in 1972 (GS-Series) and boring CS-18 at the Powerhouse established the straitigraphy for the remainder of the project. Top of rock and ancient river channels in the tailrace area were unsuccessfully investigated with seismic equipment in 1972. Earthquake history and seismic activity in the project area were researched and presented in the powerhouse Site Selection and Geology DM No. 6, May 1975.

11. Canal and levee geologic investigations for this report. A deep boring, IT-27, was drilled at the shore of Lake Moultrie to better define stratigraphy between the Entrance Channel (Lake Moultrie) and the Powerhouse in search of evidence of earthquake-related displacements in the project area. Selected rock samples were subjected to paleontological analysis to establish stratigraphy by age. General geology, earthquake history and paleo reports are presented in Appendix A, Geology and Soils. Geologic top of rock was better defined within the excavation limits at one location in the intake canal and at several locations in the tailrace canal by shallow cored borings (IT-series and T-series).

12. Groundwater study. Details of site geology related to groundwater conditions are presented in report, "The Effect of the Cooper River Rediversion Canal on the Groundwater Regimen of the St. Stephen Area, South Carolina", October 1975, prepared for Charleston District by U. S. Geological Survey. The report evaluated the effects of powerhouse and canal construction and operation on the project site groundwater regimen. The above groundwater report will be supplemented with yearly groundwater data and evaluations by USGS. The Corps' agreement with USGS provides for continued monthly monitoring of thirteen observation wells for an indefinite number of years. The present projected end of groundwater monitoring is three to five years after power-on-line (rediversion) depending on the detectable changes in groundwater conditions immediately after rediversion.

13. Future geologic investigations. Records of seismic activity in and around the project area will be collected, compiled and published in periodic supplemental reports to the Site Selection and Geology DM No. 6. The University of South Carolina and U. S. Geologic Survey currently sponsor on-going programs aimed at determining earthquake mechanisms and seismic level prediction in South Carolina. Co-operative exchange of project geologic data between the Corps and these researchers in the past assures the Corps of access to their seismic study data, results, conclusions, etc. that develop in the future. Seismic instrumentation will be installed in the powerhouse to produce seismic activity data for comparison with existing activity records.

PHYSIOGRAPHY AND TOPOGRAPHY

14. Physiography. The Cooper River Rediversion Project lies within the Lower Pine Belt of the Coastal Plain Physiographic Province, which is a band of loose to indurated sands, silts, and clays with some limestone, and sandstones. The province is from 100 to 200 miles wide starting at Cape Cod, and terminating at the Texas-Mexican border. The Coastal Plain is divided into seven subdivisions; however, the boundaries and names vary somewhat. For this report, Mr. Charles B. Hunt's breakdown of the Province will be used. The seven subdivisions are as follows: Embayed Section, Cape Fear Section, Sea Islands Downwarp Section, Peninsular Arch Section, East Gulf Coastal Plain Section, Mississippi River Alluvial Section, and the West Gulf Coastal Plain. This report will be concerned with two subdivisions, Cape Fear Arch and Sea Islands Downwarp. The common boundary of these two areas lies near the Santee River in the vicinity of the site.

15. Topography. The proposed canal area is low-lying, relatively flat terrain. There is a maximum relief of about 80 feet from the canal entrance in Lake Moultrie to its terminus in the Santee River. The

proposed intake canal traverses relatively flat uplands (maximum relief 10 feet) for most of its length, then joins a small stream intercepting the southwest corner of the powerhouse site. The ground surface drops about 30 feet from the upland to the stream thalweg, then another 30 feet to the Santee River flood plain just downstream from the powerhouse site. The tailrace canal traverses the flood plain (maximum relief 5 feet) along the base of the upland "hill" to the Santee River, where ground surface drops about 20 feet to the existing riverbed. The Santee River flood plain is denoted as swampland on topographic maps, however, this designation misrepresents the true character of the flood plain topography. The flood plain is, in fact, low-lying hardwood timber land subject to seasonal flooding. Typical swamp-related features, such as large areas of ponded water, heavy aquatic growth and thick deposits of organic soils, are absent from this flood plain.

REGIONAL GEOLOGY

16. Regional Geology. The Cooper River Rediversion Project lies within the Atlantic Coastal Plain Physiographic Province. Very little is known of the geologic history of the area now comprising the Coastal Plain of South Carolina prior to the Upper Cretaceous. However, during Triassic time the underlying rocks of the area were apparently partially fractured by faults and intruded by basic lava. Cooke has suggested that the recent movements along one of these triassic faults may have been the cause of the Charleston earthquake of 1886 and the subsequent tremors in the region around Summerville, South Carolina. In early Cretaceous time there was a broad nearly level plain sloping slightly to the southeast, comprised of schists, granites, and other crystalline rocks like those of the Piedmont Province. Continental warping then occurred at the end of Lower and Middle Cretaceous time that domed-up the region now occupied by the Appalachian Mountains, and tilted down the land lying to the east, south, and southwest of it. The sea then transgressed upon the margin of the continent, possibly as far as the present Fall Line. From this time forward to recent times, the history of this area was marked by periodic recessions and transgressions of the sea, causing the deposition of the Upper Cretaceous, and the later Tertiary sediments.

SITE GEOLOGY AND SOILS

17. Site geology. The approximate location of the powerhouse at the center of the project site is at latitude 33°25'45" N, longitude 79°56'00" W. The project canal alignment is underlain by

clays, sands, limestones, shales and sandstones. A thorough geologic description of the project site is presented in Appendix A, Geology and Soils. Geologic profiles are shown on Plates 19 through 25, and A-2 and A-3. There is an upper zone of Tertiary soils in the proposed intake canal consisting primarily of unconsolidated red, orange and grey, dense to soft, interbedded clays, silts and fine sands varying in thickness from 30 to 85 feet. These soils overlie the Santee Limestone Formation of Eocene Age in Lake Moultrie, which formation overlies the interbedded grey to black sands, shales, sandstones and limestones of the Black Mingo Formation of Lower Eocene age. The Black Mingo crops out northwest of the proposed canal along the Santee River; maximum thickness is about 250 feet. The formation underlying the Black Mingo is the Pee Dee of Cretaceous age. In the proposed tailrace canal Tertiary soils directly overlie the interbedded Black Mingo sands, sandstones, shales and limestones in thickness up to 20 feet. The beds in the powerhouse area consist of marine sediments striking northeast and slightly dipping, less than one degree, to the southeast. No faults or evidence of faulting were logged in any soil or rock cores at the project site.

18. Soils. The soils underlying the proposed intake and tailrace canal alignments are shown in profile and cross section on Plates 19 through 25. Near-surface clayey sands (SC) appear to mantle most of the proposed intake canal alignment. Beneath the clayey sands (SC) are interbedded silty sands (SP-SM) and fat clays (CH-MH) overlying geologic top of rock. Along the tailrace alignment the near-surface soils are lean clays (CL) overlying dense, cemented sands and shales, which may be considered as geologic top of rock.

EARTHQUAKE HISTORY

19. Earthquake events. From 1754 to 1971, there have been 438 "earthquakes" with epicenters located in the State of South Carolina. Four hundred and two of these were within a 50-mile radius of the proposed Cooper River powerhouse location. The most prominent was the Charleston earthquake of 31 August 1886. It registered an intensity of 10, killed 60 people, damaged $23,000,000 worth of property, and was felt at Boston, Milwaukee, Cuba, and as far east as Bermuda. There were two epicentral points, one near Woodstock, 16 miles N. 30°W. from Charleston, and the other about 13 miles due west of Charleston. A more recent earthquake occurred on 22 November 1974, near Summerville, South Carolina, having a Richter scale intensity of 4.5. Dr. Talwani, Seisomologist, University of South Carolina, said they were unable to pinpoint the focal plane mechanism; however, the epicenter was located at latitude 32° - 52.4'N; longitude 80° - 8.6'W at a depth of 9.7 km. One hour later an aftershock epicenter was located at latitude 32° - 51.7' N; longitude 80° - 8.3'W at a depth of 9.2 km. Additional discussion of seismic history is presented in Appendix A.

20. Seismic activity monitoring system. The U. S. Geological Survey has 10 geophone monitoring stations in the Columbia, South Carolina, and Charleston, South Carolina, area with Columbia being the central recording station. The U.S.G.S. fielded six additional portable instruments to record the aftershocks of the quake occuring 22 November 1974.

21. Seismic studies. The U.S.G.S., in cooperation with the University of South Carolina, is now conducting an extensive investigation of the seismicity of the entire State of South Carolina. The monitoring stations noted above form a basic part of that investigation. In addition to these monitoring stations, several deep borings are planned by the U.S.G.S. One was drilled in January 1975 north of Summerville, South Carolina. In addition to the above program, several individual seismologists are studying the seismic regime of the state. Dr. Bollinger, Professor of Seismology and Geology at VPI, is presently conducting such a study, covering the southeastern states, which is concentrated in the South Carolina area. The results of these investigations and any other pertinent data available will be published in a future supplement to this design memorandum.

22. Recent developments in causative mechanism studies. A paper presented in "Geologic Notes", Fall 1973, Vol. 17, No. 3, by William W. Beck, Jr. on a mineralogical study of barrier islands of Pleistocene Age, included a map showing the locations of these islands. This is of interest especially in Berkeley County as Beck's location of the barrier island, "Penholoway", is running in a northeast-southwest direction nearly paralleling Withington's lineation. The lineation could be a barrier island and this would explain why the feature intersects the known beach ridges in the state. For barrier island locations, see Figure 4 in DM 6, Site Selection and Geology.

Another paper presented in "Geologic Notes", Winter 1973, South Carolina, by D. J. Colquhoun and C. D. Comer, reported an arch in the Charleston Area, with an axial strike to the northwest, with the northeast flank dipping 65 to 70 feet per mile and the southwest flank dipping 25 to 30 feet per mile. In their opinion, the Charleston area has been thoroughly surveyed with seismic equipment and no other significant structural anomalies were revealed. It is suggested by the above authors that this arch, known as the Stono Arch, may be associated with recent tectonic and earthquake activity occurring in the Charleston and Summerville area.

23. A thorough review of the earthquake history of the project area is presented in DM 6, "Geology and Site Selection". The study of the seismic regime of the state has not progressed at the same rate as has the notoriety of its possible dangers and damage to planned structures. A review of programs in progress since 1974 leaves little to alter the available facts on Cooper River Rediversion Project. The

project lies within a zone 3 seismic risk area. Since 1974, U. S. Geological Survey studies indicate that the fault mechanism responsible for production of this risk appears to lie to the southeast of the project area and trends away from the geographic locality of Cooper River Rediversion. Investigative efforts of the U.S.G.S. and the University of South Carolina have shifted to areas between Charleston and Orangeburg. Their deep boring program of 1975 resulted in no definitive results; however, the early 1976 program revealed some strata offsets of up to 15 feet in the Oligiocene - Pliocene deposits. This evidence can be indicative of an existing fault approximately 40 miles southeast of the project in an area between the Edisto and Ashley Rivers. Due to the limitations of the investigations, they were not able to delineate a strike trend for the fault. No evidence of faulting was found in a study of the Moncks Corner area approximately 10 miles south of the project and investigated in 1974 by the U.S.G.S. Recent attempts by some investigators have been made to downgrade the intensity of the Charleston earthquake of 1886. Dr. Bollinger, Professor of Geology and Seismology at VPI believes that his studies indicate that the 1886 Charleston earthquake still rates an intensity event of X on the Modified Mercalli Scale. In summary, studies to date have not indicated the presence of faults in the Cooper River Rediversion Project area, nor have they led to downgrading of the seismicity of the project area.

GROUNDWATER

24. The groundwater regimen in the project area has been monitored since 1970 with piezometers installed in bore holes. Observation wells (20) were added to the groundwater monitoring system in 1973. Groundwater levels were recorded during all drilling operations and were noted on boring logs. Water levels were noted at the completion of drilling and again 24 hours later. Piezometer records are presented in Appendix A, Geology and Soils. Observation well records, chemical analysis of groundwater and crude transmissibilities of geologic formations are presented in U.S.G.S. report, "The Effect of the Cooper River Rediversion Canal on the Groundwater Regimen of the St. Stephen Area, South Carolina". The groundwater conditions in the powerhouse area are presented in Site Selection and Geology DM 6, May 1975. Groundwater levels in the intake canal vary from elevation 70 msl near Lake Moultrie to about elevation 75 msl midway along the canal to about elevation 50 msl at the powerhouse. Artesian springs are present along the base of the hillside slopes just upstream from the powerhouse. Seasonal fluctuations in groundwater occur in the tailrace canal area at the edge of the Santee River floodplain. Levels vary from about elevation 16 in the floodplain during dry periods (usually in the late fall) to artesian flow from top of ground (average elevation 21) in wet periods (usually winter and spring). Assumed groundwater tables are shown on geologic sections, Plates 19 through 25.

LABORATORY TESTING

25. General. Soil samples from drive-sampled borings were sealed in plastic jars to preserve moisture content for testing. Undisturbed samples of soil were obtained by Shelby tubes which were sealed at both ends to preserve moisture content. Soil laboratory classifications by the Unified Soil Classification System (USCS) are noted on the boring logs along with field classifications. USCS classifications show soil symbols in parenthesis. Pertinent laboratory testing correspondence and test data are presented in Appendix A, Geology and Soils. All soil testing was performed in Corps laboratories, either South Atlantic Division Laboratory or New England Division Laboratory, as noted on the test data sheets.

26. Previous testing. Classification tests and triaxial shear tests were performed by SADL on selected disturbed and undisturbed soil samples from borings made during the site selection phase. The laboratory soil classifications and triaxial test results were presented in Appendix II, Volume II, General Design Memorandum, January 1972. A summary of undisturbed samples tested, material classifications, sample elevations and strengths was shown in Table 4, same report. The material selected for shear tests was considered to be the weakest material encountered above the invert (elevation 50) of the GDM intake canal. No testing of rock samples was performed during this phase.

27. Testing for this report.

a. Soil. Classification tests (USCS) were performed on selected disturbed soil samples to confirm classifications made by field personnel (trained soils inspectors). Each sample submitted to the laboratory was visually classified and tested for moisture content. Atterberg Limits and gradations were determined for representative samples. Consolidation tests, unconfined compression tests, Q and R triaxial shear tests, and S direct shear tests were performed on undisturbed soil specimens and on remolded specimens from composite soil-type samples. Compaction tests were also performed on the composite soil-type samples. Since the weak grey clay found at Lake Moultrie exhibited a joint system, undisturbed cube samples were submerged in water at SAD Laboratory for observation of structure deterioration. The samples crumbled rapidly in water. A sandy clay soil found in the powerhouse area was analyzed for mineralogy by X-ray diffraction. The soil sample (from elevation 17 to elevation 18 msl in boring 51) diffractogram was estimated to show 77% montmorillonite.

b. Rock. Engineering properties of rock encountered in the canal subsurface investigations were not tested. Previous testing of rock in the powerhouse foundation was discussed and presented in Site Selection and Geology DM No. 6, May 1975. Paleontological tests were performed by University of South Carolina on rock cores to determine the geologic age of rock formations under Lake Moultrie, the Intake Canal and the Powerhouse.

CONDITIONS OF SPECIAL ENGINEERING SIGNIFICANCE

28. Soft soil. Low blow count (0 to 3 blows/foot) clay soils were discovered within the proposed intake canal slope height between Lake Moultrie (Station 135+00) and State Rt. 35 (Station 194+00). Subsurface investigations (see boring IT-27) delineated the extent and thickness of the soft soil deposits. Undisturbed samples were laboratory tested to determine physical properties (shear strength, consolidation characteristics, etc.) of the soft soil in-situ. Tested shear strengths were low enough to affect cut slope design and physical property test results indicate that these clay soils will not be suitable (too wet) for levee construction. Undisturbed cube samples of these clay soils crumbled rapidly into joint-bounded pieces when submerged in water at SAD Laboratory.

29. Top of rock. Hard rock will be encountered within limited segments of the proposed excavations for the intake and tailrace canals. Soft rock will also be encountered in several segments of the Tailrace Canal. In the Intake Canal hard rock, geologically classified as limestone, occurs above the canal invert between stations 245+00 and 295+00. Approximately four feet (vertical) of the limestone would be removed during excavation of this segment of the Intake Canal, possibly requiring pre-blasting before excavation by a small to medium-sized dragline. The soft rock, geologically classified as shale, along with thin layers of hard limestone and siltstone would be removed by small to medium-sized dragline excavation of the Tailrace Canal without pre-blasting. Soft rock in the Tailrace Canal varies in excavated thickness from fourteen (14) feet at the Powerhouse to zero below the SCL Railroad bridge to three (3) feet in Lake Mattassee near the Santee River.

30. Groundwater. Perched groundwater of limited volume occurs at several locations along the Intake Canal. The normal groundwater table, as well as the perched tables, appears to charge springs outcropping in the powerhouse area. Spring flow varies with the seasons and many springs in the powerhouse and tailrace areas exhibit artesian head. Pump test results (see Site Selection and Geology DM No. 6, May 1975) indicate soil transmissibilities are low at the powerhouse. The normal groundwater table in the Santee floodplain (the Tailrace Canal) is very close to ground surface and at several boring locations has exhibited up to two (2) feet of artesian head. Based on the

predominance of granular soils in the floodplain, the tailrace soil transmissibilities are expected to be much higher than for the intake canal soils. Pump tests run by USGS in the floodplain observation wells show higher soil transmissibilities than the upland pump tests.

31. Santee River flood history. The Santee River floodplain is subject to complete inundation at frequent intervals. Inundation persists for three to six weeks depending on flood severity. Flood waters are released over the Lake Marion spillway dam (Wilson Dam), approximately 34 river miles above the proposed tailrace exit, when necessary to prevent lake levels at Lake Marion from exceeding elevation 76.8 feet msl. Wilson Dam spills have occurred as often as six times per year, with most of them occurring more frequently during the Spring season of the year. The most recent major spillage of flood waters occurred during March and April 1975 which inundated the floodplain to elevation 29.7 msl at the Lake Mattassee USGS stream gaging station.

32. Seismic activity. The project site is located in an active seismic area. Seismic events of low magnitude are recorded frequently in the Charleston/Summerville area, approximately 45 miles southwest from the site. High magnitude events have occurred in the Charleston area, such as the magnitude 10 (Richter scale) earthquake in 1886.

ENVIRONMENTAL ASPECTS

32a. General. The discharge from Lake Moultrie into the Cooper and Santee Rivers will be reapportioned so that the flow in the Cooper River will be decreased by an average of 12,600 cfs and flow in the Santee River will be increased by a like amount. This increased discharge into the Santee River and its estuary will move isohalines seaward, raise water levels in adjacent swamps, dilute pollution, and increase the rate of delta formation. The decreased discharge into the Cooper River will change the hydraulic character of the harbor from a stratified estuary to a vertically mixed estuary and thereby greatly reduce harbor shoaling. Additionally, isohalines will move landward, water levels in adjacent rice fields will be lowered, dilution of pollution will be reduced in the upper harbor and increased in the lower harbor.

32b. Fish and wildlife. Fishery resources will be adversely affected in the Cooper River but will be more than offset by corresponding gains in the Santee River. A new fish hatchery will be constructed below the new powerhouse to mitigate the possible reduced effectiveness of the existing hatchery at Pinopolis. A fish lift will also be constructed in the new powerhouse to mitigate the possible reduced effectiveness of the Pinopolis Lock in passing blueback herring into Lake Moultrie. A fish diverter is authorized for the Santee River if one appears to be needed to divert herring into the new tailrace canal. Timber growth and mast production will be increased in the Santee River swamps which will also improve the quality of wildlife habitat. Salinity reduction in the Santee estuary will also improve the waterfowl value of about 38,000 acres of marsh. Land requirements for the rediversion canal and new powerhouse will eliminate about 450 acres of wildlife habitat; however, disposal areas required for construction will be revegetated to provide food and cover for wildlife. The reduced rate of shoaling in Charleston Harbor will also reduce the requirement for disposal areas which have often been located in wildlife habitat.

32c. Environmental Impact Statement. The final EIS for this project was filed with CEQ on 14 January 1975. The present detailed studies do not include any significant variation from the impacts described in the EIS.

HYDROLOGIC STUDIES

33. General. Hydrologic studies presented in the General Design Memorandum, paragraphs 40 through 52, provided the basic hydrologic criteria for the preliminary design of the rediversion canal, the St. Stephen hydroelectric plant and the interior drainage plan. Pertinent project data associated with Lakes Marion and Moultrie and with Wilson Dam are presented in a pertinent data table following the Table of Contents of this report and in Appendix B, Hydrology and Hydraulic Design. Since publication of the GDM, additional hydrologic studies have been conducted in response to comments contained in the GDM and subsequent design conferences, and to provide more precise criteria for design of the St. Stephen Powerhouse. These additional studies are summarized in this portion of the DM. Details concerning the additional studies, procedures used and results obtained can be found in Appendix B.

34. Basic hydrologic design concept. The existing Santee-Cooper reservoir system, consisting mainly of Lakes Marion and Moultrie, Wilson Dam and Spillway, the Marion-Moultrie diversion canal, Pinopolis Hydro-electric plant and the small hydro-electric plant located at Wilson Dam, will not be altered by the rediversion

project. The average flows through the reservoir system of 500 cfs through the small hydro-plant and 15,500 cfs through and out Lake Moultrie will also not be affected by rediversion; neither will the water supply or inflow into the system, or flows through the Marion-Moultrie diversion canal. The rediversion project will only alter the point at which some of the Lake Moultrie outflow occurs. Prior to rediversion discharges at the Pinopolis Plant entered Cooper River with a maximum peak rate of about 27,500 cfs. The average flow rate was about 15,500 cfs. After rediversion, the average flow will be restricted to 3,000 cfs. The remaining 12,500 cfs will be diverted through the new St. Stephens Hydro-electric plant and into the Santee River. Therefore, the only flow regimens altered by the rediversion project will be those in the Cooper River and those in the Santee River generally below where the tailrace enters the river. This report addresses only the change in flow regimen of the Santee River.

35. Update of Survey Report reservoir operation study. The reservoir operation study presented in Appendix E to the July 1966 project survey report entitled "Hydro-Electric Power Generation Study", was updated for this report. The Survey Report covered the period from 1908 through 1963. The update started in 1964 and continued through 1975. The purpose of this continuation of the Reservoir Operation Study was to obtain additional spill data to improve the accuracy of the Santee River regulated discharge frequency analysis. A copy of the computer print-out for the 1964-1975 period is presented as Exhibit B-1 in Appendix B. The power values obtained from the 1908-1963 operation study and contained in the Survey Report were not updated. Should this at some later date be needed, this can readily be accomplished from the data contained in the computer print-outs.

36. Standard project flood. Reference SADVY 1st Ind to GDM, para. 2c. In order to determine the impact of standard project flood stages on design of the Powerhouse and the tailrace riverside levee, the standard project flood was determined and routed through the Santee Cooper reservoir system and down the Santee River to Jamestown. The Standard Project Flood inflow hydrograph to Lake Marion was developed by the Savannah District for the Charleston District. The report prepared by them outlining their study is presented as Exhibit B-2 in Appendix B. As shown there and on Plate B-6, Appendix B, the Standard Project Flood peak inflow rate is 631,260 cfs. Shown also on Plate B-6 are the spill hydrograph (peak 578,000 cfs), the estimated hydrograph near the powerhouse (peak 556,000 cfs, 53.9 stage ft. msl), the peak lake elevation at Marion (77.32 ft.) and the corresponding peak elevation at Lake Moultrie (75.55 ft.). Critical design elevations at the powerhouse, the floor and deck levels, are set at 57.0 ft. msl. This elevation is above the SPF level of 54 feet msl at the Powerhouse.

37. Santee River - discharge frequency. Discharge frequency curves for both natural and regulated conditions were developed. These curves are shown on Plate B-5, Appendix B. One curve shown is for the

natural condition near Wilson Dam (inflow to Lake Marion); another curve shows the regulated condition for the river reach between the powerhouse and tailrace canal confluence; and the remaining curve shows the condition for the river reach just below the tailrace canal where river discharges are influenced by powerhouse releases. These curves were developed using HEC's Regional Frequency Computer Program and stream gaging records varying in length from 34 to 84 years, spill data obtained from the updated Survey Report Reservoir Operation Study and flood routings using an unsteady flow computer model. For the purpose of Santee River discharge frequency analysis, the monthly routing interval used in the period of record operation study was inadequate. Therefore, for spill periods only, a daily routing was performed. The methods and procedures used in this analysis are fully explained in Appendix B.

38. Santee River stage frequency. Stage frequency curves were not developed; however, the stage for a given frequency flood can be derived at two project locations using the discharge frequency curve (Plate B-5) and the stage-discharge rating curves shown on Plate B-7, Appendix B. These two rating curves are at the powerhouse and the USGS gage at Lake Mattassee.

39. Santee River stage-discharge duration. A tabulation of all spills obtained from the updated Reservoir Operation Study are presented in Table B-4, Appendix B. This table also lists discharge-duration values for each spill. A flow duration curve of spills is shown on Plate B-8, Appendix B.

40. Santee River stage and discharge for selected flood frequencies. A summary of design discharges and stages for selected frequencies and locations are shown on Table 1.

Table 1

Santee River Flood Data

Flood Frequency (Years)	Inflow to Lake Marion (1,000's cfs)	Powerhouse(1) Discharge (1,000 cfs)	Powerhouse(1) Stage (Ft. msl)	Lake Mattassee(2) Discharge (1,000 cfs)	Lake Mattassee(2) Stage (Ft. msl)
10	157.0	108.0	36.5	130.0	31.3
25	250.0	188.0	41.2	205.0	35.1
50	330.0	265.0	44.6	282.0	38.8
100	425.0	365.0	48.2	380.0	42.5
SPF	631.0	555.8	53.9	576.9	48.3

(1) Mile 59.5 - See Plate B-6.

(2) Mile 51.7 - See Plate B-6, below Tailrace Canal and Santee River confluence.

41. Lake Moultrie design water level. The hydrologic studies for the Lake Moultrie design elevation were presented in the approved "Entrance Channel in Lake Moultrie", DM No. 3, dated March 1974, para. 28. The design lake elevation of 74.0 feet msl was taken as the second lowest pool elevation during the peak load month, August, for the period of record (1908-1972), equivalent to a 50-year frequency of occurrence.

42. Interior drainage. The location of the project intake and tailrace canals interferes with and in many places prevents drainage of adjacent local areas. The recommended plan to relieve this situation and to provide drainage for each of these areas is shown on Plate B-12, Appendix B. Generally, all major structures were designed to safely pass the 50-year frequency flood. Design flow conditions for the drainage ditches or canals varied from minimal sizes up to about the 25-year flood. Instead of the rational method, as used in the GDM, a regional frequency analysis using coastal plain gaging stations was used to determine design discharges. This change in computing methods resulted from GDM comments. The hydrologic analysis conducted and the hydraulic design criteria for the recommended drainage plan is presented in Appendix B.

HYDRAULIC DESIGN

43. Basic design criteria. The General Design Memorandum (DM No. 1), states in paragraph 5 that the proposed powerplant will be sized to discharge about 24,500 cfs at the rated head of 49 feet. The entrance channel configuration presented in Design Memorandum No. 3, Entrance Channel in Lake Moultrie, was selected to provide 24,500 cfs to the powerhouse, via the intake canal, with a net head drop to the tailrace of 49 feet at a lake (Lake Moultrie) elevation of 74.0 feet msl. The entrance channel invert was set at elevation 64 msl at canal station 0+00 with a bottom width of 1,500 feet. Between station 89+34 and station 115+34 the channel bottom narrows from 1,500 feet to 385 feet and the invert drops from elevation 64 msl to elevation 54 msl. The proposed intake canal begins at station 135+34.

44. Canal studies. Various combinations of canal width, and invert elevations that will satisfy the above design requirements are shown on Plate B-14, Appendix B, Hydrology and Hydraulic Design. The curves on the plate show the inter-relationship between canal bottom widths and invert elevations for the intake and tailrace canals. Invert elevations of 0.0 and 3.5 feet for the tailrace and 50 and 54 feet for the intake are shown. The tailrace canal invert was selected at elevation 0.0 feet msl for this report as the optimum balance between rock excavation and canal width, with consideration given to the existing thalweg of the Santee River. Canal side slopes were established

by stability analyses at 1 vertical on 3 horizontal or flatter. Based on the curves on Plate B-14 and consideration of rock excavation, the optimum intake canal cross section would have a bottom width of 285 feet and an invert of 50 feet msl. The corresponding tailrace canal cross section would have a 285-foot bottom width at an invert elevation of 0.0 feet msl. Hydraulic design aspects of the intake and tailrace canals are presented in Appendix B, while optimization studies are presented in Appendix C, Alternate Studies. The intake and tailrace canal cross sections are shown on Plates 13, 14 and 15. A 750 foot long transition reach would be constructed between the box cut section of the entrance channel and the proposed intake canal cross section. The channel invert in this reach would transition from elevation 54 to elevation 50 msl.

45. Tailrace tailwater rating curve. The tailwater rating curve used in the design of the proposed intake and tailrace canals is shown on Plate B-25. This curve was determined from water surface profiles computed between the Santee River at Lake Mattassee and the proposed powerhouse. Starting conditions for the water surface profiles were determined from data available at the U. S. Geological Survey gaging station, Santee River below St. Stephens (02171650). Manning's "n" for the computations was assumed to be 0.025.

46. Velocity studies. The canals would be excavated in soils varying from cohesive clay to cohesionless fine sand. Also present would be cemented sands and weak sedimentary rock. The proposed intake and tailrace canals were designed for a maximum water velocity of 3.5 feet per second so that erosion protection would not be required for the major portion of the canal length. Velocity studies for the tailrace immediately downstream from the powerhouse are presented in Appendix B. Maximum velocities in the tailrace are predicted to be in the range of 6.0 to 7.6 feet per second for a distance of about 4,000 feet downstream from the powerhouse during start-up, which may be a daily event.

47. Surge studies. Because surges of various sizes have been reported at similar canal-type hydropower projects, surge development in the intake canal due to gate closing or other phenomena was evaluated. The study showed that a surge of approximately 2.3 feet in height can develop in the intake canal with a 5 second closure of the powerhouse gates.. This is not considered to constitute a hazard to any small boats in the canal at the time since the wave length is over 2,000 feet. Refer to Appendix B for results of the surge studies conducted and the methods used.

48. Forebay storage. Drawdown of the intake canal water level due to start-up of the turbines was calculated for the critical combination of water level and operating conditions. The greatest drawdown was approximately 2.3 feet for the most rapid gate

opening considered. Since gate opening drawdown is small, forebay storage is not considered necessary in the Intake Canal at the Powerhouse. Forebay studies are presented in Appendix B.

49. Wave studies. The fetch in Lake Moultrie is longest in the southwest to northeast direction, where it approaches 15 miles. Wave heights and run-up were calculated for design hurricane storms. The existing Lake Moultrie dikes and the proposed intake canal levees would be vulnerable to wave action from the lake during storms. A shallow water wave height of approximately 4.7 feet was predicted to occur during a design hurricane, with accompanying wave run-up on a 1 vertical on 3 horizontal slope of about 4.0 feet. Details concerning the wave studies are presented in Appendix B.

50. Levee heights and freeboard. The design crest of the existing dike around Lake Moultrie was elevation 85 msl; however, survey data in the vicinity of the project show that dike crest elevations vary between 85 and 86 feet msl. The recommended crest elevation of the proposed levee paralleling the Intake Canal and at its junction with the existing dike would be elevation 86 msl. This elevation was selected after considering the natural protection afforded the existing and proposed levees by trees and underbrush fronting the existing dike, and computations for wind set-up, wave set-up and wave run-up. Materials excavated from the canals in excess of levee material requirements would be disposed behind the proposed levees to heights up to elevation 97 feet msl. The disposed materials behind the levees would contribute significant additional protection should storm surge water levels exceed elevation 86 msl. Therefore, the primary purpose for having levees with designed crest elevations along the intake canal is to construct stable side slopes within the possible range of canal water levels. Design storm parameters and results of "set-up/run-up" studies are presented in Appendix B.

51. The levee heights in the Tailrace Canal were established between the 30-year and 50-year Santee River flood event for the river side (left descending) levee and at the 10-year flood event for the land side (right descending) levee. The 50-year flood level (elevation 45 msl) at the powerhouse is one foot less than the top of the existing Seaboard Coastline Railroad embankment (elevation 46 msl), which crosses the proposed tailrace canal at about station 419+00. No freeboard was included in the design crest elevation (45 msl) for the river side levee between the powerhouse and the SCL Railroad. The proposed river side levee crest would slope from elevation 45 msl at the SCL Railroad to elevation 35 msl at the Tailrace Canal exit. The land side levee would start at the SCL Railroad embankment (canal station 419+00) and connect with the Lake Mattassee access road. The land side levee crest elevation would be 35 msl. The primary purposes of the land side levee are to provide (1) operation and maintenance access during normal operating conditions up to about the 10-year flood level and (2) flood protection

during construction of the tailrace interior drainage ditch and excavated material disposal areas. The 10-year Santee River flood level was selected to provide sufficient construction protection and maintenance access. A freeboard of about 3.5 feet was included in the land side levee crest elevation.

52. Future instrumentation. Stage records in the Santee River have been obtained manually during major Wilson Dam spills. It is planned to install at least one additional automatic gage in the Santee River at the Seaboard Coastline Railroad. The gage would provide continuous water level data and information concerning reverse flow in the Santee River following initiation of powerhouse releases.

53. Interior drainage.

a. General. The proposed intake and tailrace canals intersect several small drainage areas (see Plates 1 and B-12)) that normally flow southwest to northeast into the Santee River. The interior drainage plan basically provides for collecting the intercepted flows along the south boundary of the project and dropping the collected water into the Tailrace Canal at two selected locations. Several methods of collecting and draining the intercepted flows were studied. Refer to Appendix C, Alternate Studies, for details of alternative interior drainage plans considered. Locations of collector ditches and drop structures, and directions of drainage, are shown on Plates 6 through 12 and on Plate B-12, Appendix B.

b. Intake canal. The proposed plan for intake canal interior drainage provides two collector ditches, one of which would collect runoff along the south project boundary starting at approximate canal station 232+00 and drain toward Lake Moultrie. The collector ditch then would join an existing ditch at the headwaters of Halfway Swamp to drain the collected runoff south away from the project into the Cooper River basin. A larger ditch would collect runoff for the remainder of the intake canal beginning at approximate canal station 233+00 draining toward and around the Powerhouse to empty into the Tailrace Canal at approximate canal station 408+00. Collector ditch invert elevations and grades are shown in profile on Plates B-29 through B-32. Maximum collector ditch grade would be 4.8 feet per thousand feet. Two large concrete drop structures would be required near the Powerhouse (see Plate 9). The drop structures proposed are USBR Type IX structures, with drops of approximately 16 and 18 feet.

c. Tailrace canal. The proposed tailrace canal interior drainage plan would collect runoff along the land side canal levee with a single ditch starting at the Seaboard Coastline

Railroad and draining into the Tailrace Canal at approximate canal Station 601+00. A large concrete drop structure would be required at approximate canal station 596+00, under the Lake Mattassee access road. The drop structure would be a USBR Type IX structure with a drop of approximately 16 feet. Collector ditch invert elevations and grades are shown in profile on Plate B-32.

54. Exterior drainage. The natural drainage to the north and northeast from the intake and tailrace canals is considered exterior drainage in this report because the project would have only minor effect on the drainage features. Small collector ditches would be constructed along the toes of the intake canal excess excavated material disposal areas to provide outlets for disposal area run-off to existing drainage features. Two culverts (C-7 and C-8) would be required at two road crossings. In the Tailrace Canal the existing Mattassee Run drainage channel would be relocated to accommodate the riverside levee. The Mattassee Run channel would then join the Tailrace Canal at Station 596+80. A general plan view of the proposed exterior drainage plan is shown on Plate B-12. The proposed locations of the exterior drainage ditches, culverts, directions of flow and Mattassee Run channel are shown on Plates 6 through 12. Discussion of the design details of the exterior drainage plan is presented in Appendix B.

ALIGNMENT STUDIES

55. Powerhouse site location. The location of the powerhouse shown on Plate 9 was recommended in the approved Site Selection and Geology DM No. 6, 2 May 1975. The powerhouse has been rotated approximately 26 degrees clockwise about the centerpoint of the site (hole P-18) as a result of changes in the intake canal alignment generated from the orientation recommended in DM No. 6 by the alternative alignment studies.

56. Entrance channel alignment. The alignment of the entrance channel shown on Plate 1 was recommended in the approved Entrance Channel in Lake Moultrie DM No. 3, March 1974.

57. Intake and tailrace canals. Three alternative intake and tailrace canal alignments were studied to determine the most suitable alignment for feature design. The alignments studied are described in detail in Appendix C, Alternate Studies, and shown in the plan on Plate C-1. The selected canals alignment is shown on Plate 1. The alternative alignment studies were coordinated with the Federal and State fish and wildlife agencies and the University of South Carolina Institute of Archeology and Anthropology. The selected alignment for the intake and tailrace canals represents the most feasible combination of construction cost, subsurface conditions, hydraulic efficiency and environmental impact.

INTAKE CANAL DESIGN

58. Cross section studies. The optimum intake canal cross section was determined from the hydraulic design curves on Plate B-14, Appendix B, and in consideration of the amount and type of material to be excavated from the canal. Hard rock was encountered at elevation 54 msl between canal Stations 255+00 (S.C. Route No. 45) and 330+00 during subsurface investigations for this report. The assumed top of rock surface in this portion of the canal is shown in profile on Plate 19. The proposed intake canal cross section has the invert at elevation 50 msl, a bottom width of 285 feet and cut slopes at 1 vertical on 3 horizontal. The cut slopes between canal Stations 134+30 and 150+50 were flattened to 1 vertical on 3.5 horizontal for stability reasons explained below. A berm of variable width (30 feet to 130 feet) was set at elevation 78 msl for the dual purpose of cut slope maintenance access and earthquake stability. The top of power pool at Lake Moultrie would be elevation 75.2 msl. Intake canal cross sections are shown on Plates 13 and 14.

59. Slope stability analyses. The 1 vertical on 3 horizontal intake canal cut slopes proposed in the General Design Memorandum were analyzed for stability under the loading conditions prescribed in EM1110-2-1902, Stability of Earth and Rockfill Dams. Computer-aided analyses were performed for assumed circular arc and wedge-type failures of the cut slopes. Earthquake design is discussed in more detail in subsequent paragraph 97. Design strengths, soil profiles, computation procedures and results of the stability analyses are presented in Appendix D, Slope and Levee Stability Analyses. The 1V on 3H cut slopes were flattened to 1V on 3.5H near Lake Moultrie because of soft clay soils encountered near the canal invert elevation in subsurface investigations for this report. The 1V on 3H slope values would be stable for the remainder of the intake canal under normal gravity loading and under earthquake loading with accelerations up to about 0.05 g. Earthquake accelerations as high as 0.15 g, the recommended design coefficient for earthquake Zone 3 (see Plate 26), would theoretically fail cut slopes steeper than about 1V on 8H in the intake canal; therefore, a berm was established at elevation 78 of sufficient width for the cut slope to fail without destroying the levees. The intake canal cut slope design between station 339+62 and the powerhouse is presented in the powerhouse Foundation Design Report, February 1976. The canal cut slope heights would be less than 5 feet in this section of the intake canal nearest the Powerhouse.

60. Excavation and disposal of excavated materials. Refer to Table 2, Materials Usage Chart, for excavation volumes and distribution of excavated materials. The major portion of the intake canal

excavation is expected to be performed by dragline without elaborate dewatering facilities. Upper soils are predominantly dry sandy clays (SC) suitable for construction of the levees. The soils above the groundwater table can be excavated by scraper and pan for disposal in the levees or disposal areas. Soils below the groundwater table can be excavated by dragline and placed directly in or hauled to disposal areas adjacent to the levees. The groundwater table at some locations in the intake canal is within 10 feet of the ground surface, as shown on geologic profile and geologic sections on Plates 19 through 23, and varies as much as 10 feet seasonally. Piezometer records are shown in Appendix A, Geology and Soils. It is anticipated that the limestone rock in the intake canal can be excavated by a large dragline; however, rock excavation methods will be confirmed by a test excavation prior to preparation of plans and specifications for the canal construction. Ditching and pumping from sumps are expected to accomplish any dewatering necessary for dragline excavation. The excavation, levee embankment and disposal area slopes would be trimmed and dressed by bulldozer, or other suitable equipment.

61. Erosion control. The intake canal was designed to keep flow velocities below 3.5 feet per second. Maximum degree of curve in the canal alignment was set at less than 3 degrees to keep velocities within the design maximum. Based on the velocity scour criteria in EM1110-2-1602, May 1971, vegetative cover (grass) would be applied to the upper intake canal slopes during construction to protect the upper canal cut slopes from velocity scour and surface water run-off erosion. The mouth of the intake canal at Lake Moultrie would be subject to storm wave action from the lake; however, stone slope protection was not considered necessary on the upper canal cut slopes.

TAILRACE CANAL DESIGN

62. Cross section studies. The invert elevation for the Tailrace Canal was set at 0.0 msl in consideration of (1) the amount and type of rock expected to be encountered along the proposed canal alignment, (2) the hydroelectric head requirement of 49 feet, and (3) the existing thalweg of the Santee River. The optimum tailrace canal cross section was determined from the hydraulic design curves on Plate B-14 in conjunction with the selection of the intake canal cross section. The proposed tailrace canal alignments avoids excavation in hard rock except for a minor amount in the area of the Seaboard Coastline Railroad (canal Station 419+00). The assumed top of rock surface at the railroad crossing is shown on Plate 24. The tailrace canal cross section would have a bottom width of 285 feet at elevation 0.0 msl and cut slopes at 1 vertical on 3 horizontal. A berm of variable width (37 to 90 feet) was set at elevation 26.0 msl (2 to 4 feet

above maximum normal tailwater level) for cut slope maintenance access, earthquake stability and drawdown attrition. The cut slopes at the Powerhouse between elevation 26 msl and elevation 46 msl were set at 1 vertical on 3.5 horizontal, based on cut slope design studies presented in the Powerhouse Foundation Analysis, February 1976. The canal invert rises at the tailrace exit from elevation 0.0 msl to elevation 5.0 msl in a distance of 600 feet. Tailrace canal cross sections are shown on Plates 14 and 15. The canal cross section design at the railroad will be presented in forthcoming Seaboard Coastline Railroad Relocation, DM No. 8; however, the railroad bridge and abutments will be designed to avoid constriction of the design canal cross section.

63. Slope stability analyses. The 1 vertical on 3 horizontal tailrace canal cut slopes proposed in the GDM were analyzed for stability following the same procedures as for the Intake Canal. Design strengths, soil profiles, computation procedures and results of the stability analyses are presented in Appendix D, Slope and Levee Stability Analyses. The amount of daily drawdown expected in the tailrace made a significant difference in loading conditions governing slope values in comparison with the intake canal. The 1V on 3H slope values are stable under normal gravity loading, including drawdown and high groundwater table, and under earthquake loading with accelerations up to about 0.05 g. In the same way as the Intake Canal, these cut slopes would theoretically fail to an inclination of about 1V on 8H under earthquake accelerations as high as 0.15 g, the design coefficient for earthquake Zone 3 (see Plate 26). A berm was established at elevation 26.0 msl of sufficient width for the cut slope to fail without destroying the levees. The tailrace canal cut slope design between the Powerhouse and approximate canal Station 378+00 is presented in the Powerhouse Foundation Analysis, February 1976.

64. Excavation and disposal of excavated materials. Refer to Table 2, Materials Usage Chart, for excavation volumes and distribution of excavated materials. The tailrace canal excavation is expected to be performed by dragline in the wet without dewatering. A thin surface layer (upper 11 feet) of lean clays (CL) can be excavated by scrapers and pans during dry seasons and used to construct the major portion of the river-side levee. The remainder of the river-side levee and the land-side levee would be constructed of soft rock fill, sandy clays (SC) and silty sands (SM) spread and re-worked to dry before being placed in the embankments. The lowest groundwater levels recorded are approximately six feet below ground surface and many areas of the Tailrace Canal have groundwater at the ground surface during the wet season in the spring. Soft weathered shale, sandstone and limestone will be encountered in the

tailrace canal excavation, as well as cemented fine sand. These consolidated materials can be excavated by a small dragline without blasting or ripping, and occur at several locations below geologic top of rock just above the invert of the canal. Dewatering of the Tailrace Canal is not considered practical or necessary. Excavation slopes above the groundwater table and embankment slopes will be trimmed and dressed by bulldozer. A test excavation into hard sandstone is planned for the SCL railroad crossing area prior to preparation of plans and specifications to confirm rock excavation unit cost estimates.

65. Erosion control. Velocities up to a maximum of about 7.6 feet per second are estimated to develop at the downstream end of the powerhouse stilling slab as the turbines start up and the tailwater increases, causing a flood wave to propagate downstream. However, these maximum velocities quickly reduce downstream and are expected to be less than 6.0 fps about 4,000 feet below the Powerhouse. The degree of curvature in the canal alignment downstream from the SCL railroad crossing (Station 419+00) was set at 3 degrees to keep velocities from exceeding the 3.5 feet per second maximum design value. Following the criteria outlined in EM1110-2-1601, riprap has been provided at the end of the powerhouse stilling slab and on the canal cut slopes covered in the Powerhouse Foundation Analysis, February 1976. The riprap protection on the cut slopes would extend from the Powerhouse to about canal Station 395+62. The canal bottom would be excavated in shale and cemented sand between the Powerhouse and the SCL railroad crossing (Station 419+00); therefore, the bottom of the canal would not be riprapped. The canal section at the relocated SCL railroad bridge and embankment would also be protected with riprap as provided for in the forthcoming Seaboard Coastline Railroad Relocation, DM No. 8. The remainder of the canal cut slopes and levee embankment slopes would be protected with grass. Experience of others with canal-type hydroelectric projects has shown that the canal width in unprotected canals increases up to 40 percent during the project life due to drawdown-induced bank sloughing and localized velocity scour. A 90-foot wide attrition berm has been provided at elevation 26 msl wherever the canal slopes are not riprapped. The berm allows for a 20 percent increase in canal width due to operational sloughing on each side of the canal without affecting the levee structures. The berm also allows room for possible slope failure under 0.15 g earthquake accelerations without destroying the levees. The crest of the river-side levee slopes uniformly from elevation 45 msl at the SCL railroad crossing (Station 419+00) to elevation 35 at the exit end (Station 595+00) of the tailrace canal. This crest gradient is designed to permit progressive backwater filling of the tailrace canal and to mitigate erosive levee overflows for Santee River floods that exceed design conditions.

66. Stone slope protection (riprap). The stone slope protection below the powerhouse on the tailrace canal slopes (shown on

COOPER RIVER REDIVERSION

EXCAVATION & DISPOSAL QUANTITIES

MATERIAL USE CHART

INTAKE CANAL

CUT NO. STATION	EXCAVATION SOIL	EXCAVATION ROCK	DITCH EXCAV.	TOTAL (1) EXCAVATION	FILL (+10%)	EXPANDED EXCESS (+25%)	TOTAL AVAIL. SPOIL CAP.	LEFT DISP. AREA CAP.	RIGHT DISP. AREA CAP.	TO BE HAULED HAULED IN	TO BE HAULED HAULED OUT
(R-1)-135+34 to 170+00	1,174,630	0	6,420	1,217,620	177,710	1,299,380	1,417,550	810,368	607,184	-	-
(R-2)-170+00 to 195+00	939,120	0	23,920	989,420	88,690	1,125,910	1,150,480	638,943	511,531	-	-
(R-3)-195+00 to 225+00	1,196,270	0	10,620	1,238,550	77,680	1,451,080	1,451,500	849,890	601,610	-	-
(R-4)-225+00 to 255+00	1,369,870	21,720	8,420	1,431,670	26,670	1,756,250	1,764,280	366,878	1,397,350	-	-
(R-5)-255+00 to 280+00	989,890	47,290	6,650	1,070,210	51,920	1,272,850	916,170	582,221	333,952	-	356,680
(R-6)-280+00 to 305+00	780,340	63,500	29,110	899,340	124,340	968,750	1,710,380	752,424	957,951	720,570	
(R-7)-305+00 to 339+60	975,870	2,970	25,260	1,040,610	240,650	999,950	636,060	247,700	388,360	-	363,890
TAILRACE CANAL											
(R-8)-367+62 to 420+00	1,675,669	483,497	-	2,214,479	467,907	2,183,215	1,902,048	1,184,846	717,202	-	413,766
(R-9)-420+00 to 450+00	783,458	85,195	-	900,333	417,021	604,140	1,017,906	0	1,017,906	413,766	
(R-10)450+00 to 495+00	1,028,088	65,018	-	1,140,626	884,173	320,980	701,733	0	701,733	-	-
(R-11)495+00 to 520+00	620,226	60,784	-	707,410	434,690	340,900	366,971	0	366,971	-	-
(R-12)520+00 to 545+00	540,974	73,637	-	641,011	481,643	199,211	231,485	0	231,485	-	-
(R-13)545+00 to 570+00	573,505	3,349	-	603,254	501,371	127,355	327,550	0	327,550	-	-
(R-14)570+00 to 595+00	343,757	56,311	58,388	484,856	556,455	(-) 66,062	346,310	0	346,310	371,027	
(R-15)595+00 to 634+00	302,320	117,021	47,539	508,064	211,243	371,027	0	0	0	-	371,027
TOTALS	13,293,987	1,080,292	216,327	15,087,453	4,880,383						

(1) Including 1 foot overdepth excavation.

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Plate 9) was designed according to ETL 1110-2-120, May 1971. Design velocities range from 6 to 7.6 feet per second. Details of riprap gradation design are presented in Appendix A, Geology and Soils. Protection for canal slopes against velocity scour below the powerhouse consists of an 18-inch thick layer of riprap from elevation 0.0 msl to elevation 26 msl (both sides) from Station 367+62 to Station 385+50. From Station 385+50 to Station 395+62 slope protection consists of an 18-inch thick layer of riprap from elevation 0.0 msl to elevation 8 msl and a 12-inch thick layer of riprap above elevation 8 to elevation 15 on a one-foot thick bedding layer of sand that serves as a drain and a filter. The filter and drain design for the sand bedding layer is presented in Appendix A, Geology and Soils. Slope protection details at the Seaboard Coastline Railroad will be presented in the forthcoming railroad relocation feature design memorandum. Riprap design on interior drainage ditch slopes is discussed in section, Interior Drainage Design.

INTAKE CANAL LEVEE DESIGN

67. General. A low levee embankment has been provided on each side of the Intake Canal from Lake Moultrie to the Powerhouse. The purpose of the levees adjacent to the Intake Canal is to provide containment for canal water levels up to the existing dike crest level (elevation 86 msl) around Lake Moultrie with an embankment structure that has a stable, compacted, easily-maintained canal-side slope. The levee embankments would be constructed of suitable soils from the canal excavation, placed in thin lifts with moisture control and compacted with appropriate roller equipment. Excess excavated material would be disposed behind the levees to higher elevations than the levee crest. The levees are not necessary to provide containment for the excess excavated material unless unusually wet conditions are encountered in the excavation or the Intake Canal is excavated by dredge. The levees are shown in section on Plates 13 and 14.

68. Levee cross section. The intake canal levees would not be zoned, except for inclined drains and blanket drains between Station 334+00 and the Powerhouse. Suitable dry impervious sandy clays (CL) can be obtained from the upper few feet of the canal excavation to construct homogeneous embankments. Stability analyses (discussed below) were performed on assumed levee slopes starting with 1 vertical on 3 horizontal as recommended in the GDM. The 1V on 3H slopes were found to be stable under normal and earthquake loading. A 20-foot wide levee crest at elevation 86 msl was selected to provide room for patrol roads for maintenance access. Maximum height (35 feet) of levee in the intake canal occurs between canal stations 334+00 and 360+00 near the powerhouse.

69. Slope stability analyses. Slope stability analyses were performed on assumed levee slopes, starting with 1V on 3H slope values. The computer-aided analyses were performed according to the criteria outlined in EM1110-2-1902, Stability of Earth and Rockfill dams. Results of the stability analyses, selected design strengths and manual solutions of analyses are presented in Appendix D, Slope and Levee Stability Analyses. Required factors of safety were reduced from the EM recommended values because consequences of failure are not serious.

70. Foundation treatment. No soft organic soils were found near the ground surface during the intake canal subsurface investigations and visual reconnaissance. Foundation treatment for the levees is expected to consist of removing a thin layer (six to twelve inches) of surface topsoil and any unsuitable wet soils, scarifying the foundation surface for moisture control and compacting the surface for density. Artesian springs flowing from the ground will be encountered between Stations 334+00 and 360+00. The spring flows would be controlled during construction by selective placement of blanket drain material.

71. Seepage control. The fine grained soils from the intake canal excavation will be relatively impervious; therefore, no significant seepage is expected through or under the levees. The excess excavated material from the canal to be disposed behind the levees further inhibits through-seepage. In the vicinity of canal Stations 334+00 to 360+00 inclined interior drains and blanket drains would be provided to control embankment and foundation seepage. Details of levee seepage control design between canal Stations 339+00 and 360+00 are presented in the Powerhouse Foundation Analysis, March 1976.

72. Erosion control. Maximum water velocities in the intake canal are expected to be 3.2 feet per second. Based on velocity scour criteria in EM1110-2-1601, the canal-side levee slopes would be protected with grass.

73. Settlement. Settlement of the levee embankment crest is estimated to be in the range of six to eight inches. Settlement calculations for the intake canal levee foundations and embankments are presented in Appendix D, Slope and Levee Stability Analysis. Actual settlements are expected to be roughly one-half of the above estimates. The levees would be instrumented at a few locations to measure horizontal movement and settlement.

74. Patrol roads. Patrol roads have been provided at elevation 86 msl along the crest of each levee to furnish access for inspection and maintenance of the levee and waste disposal slopes, and the canal cut slopes. The patrol roads would be 10 feet wide, crowned to drain and surfaced with a thin layer of gravel or a suitable base course-type material.

TAILRACE CANAL LEVEE DESIGN

75. General. The tailrace canal has been provided with a 25-foot high levee (crest elevation varies) on the left descending canal bank (river-side levee) and a 12-foot high levee on the right descending canal bank (land-side levee). One of the purposes of the river-side levee would be to maintain power production during Santee River floods by lowering tailwater elevations. This is accomplished by preventing Santee River flood waters access to the tailrace. The purpose of the land-side levee is to provide a stable, maintainable slope and to contain flow down the interior drainage ditch. Both levees provide access to the tailrace canal cut slopes for inspection and maintenance. Construction of the levee embankments would require practically all of the material from the tailrace canal excavation. Suitable soils from the canal excavation would be placed in thin lifts with moisture control and compacted with appropriate roller equipment to construct the levees. Excess excavated material would be disposed behind the land-side levee between the levee and the interior drainage ditch. Small temporary retention levees would be required along the drainage ditch during construction to retain the fresh excess excavation material, which is expected to be wet. The tailrace levees are shown in section on Plates 14 and 15.

76. River-side levee cross section. The river-side tailrace canal levee would be zoned to place impervious soils on the outside (north) slope of the levee and granular soils and soft rock on the inside (south) slope. Stability analyses revealed that the levee slopes should be 1 vertical on 4 horizontal and that a 30-foot wide outside toe berm would be required up to elevation 26 msl to insure slope stability under Santee River flood drawdown conditions. The levee crest would be set at elevation 45 msl between the powerhouse and the SCL railroad bridge (canal Station 419+00) then sloped at a variable gradient to elevation 35 msl at the downstream end of the levee (canal Station 595+00) near the Santee River. Maximum height of the river-side levee would be about 25 feet. A levee crest width of 20 feet was selected to accommodate a patrol road for inspection and maintenance access. The downstream end of the levee would have a 150-foot wide area for a patrol road turnaround.

77. Land-side levee cross section. The land-side tailrace canal levee would not be zoned; instead the levee embankment would be a heterogeneous mixture of canal excavation materials. Soft rock from the bottom of the canal excavation would be placed toward the outer portions of the levee slopes. Stability analyses confirmed that the 1 vertical on 3 horizontal levee slopes assumed in the General Design Memorandum would be stable. The land-side levee crest width was set at 20 feet to provide room for a patrol road for inspection and maintenance access. Maximum height of the land-side levee would be about 12 feet.

78. Slope stability analyses. Stability analyses were performed on the river-side levee and the land-side levee slopes starting with the 1V on 3H slope values proposed in the GDM. Results of the stability analyses, soil profiles, selected design strengths and manual solutions of analyses are presented in Appendix D, Slope and Levee Stability Analysis. The river-side levee slopes were flattened to 1V on 4H to be stable under the condition of rapid drawdown from a 40-year flood.

79. Foundation treatment. Most of the tailrace canal is located in terrain classified as "swamp"; therefore, deposits of soft organic soils, unsuitable as foundation materials, would ordinarily be expected to be present. Actually, very little organic material was found during subsurface investigations and visual reconnaissance of the tailrace canal area. Foundation treatment for the levees is expected to consist of removing a thin surface layer (one to two feet thick) of organics, and any thicker organic deposits encountered, to firm inorganic soil suitable for an embankment foundation. Scarifying for moisture control and re-compaction for density would be applied as necessary. Artesian spring conditions would be encountered at several locations along the levee alignments. Flows would be controlled by small dikes or other suitable methods.

80. Seepage control. Water levels in the Tailrace Canal would remain below the toe of the levees except during flooding in the Santee River. During flood events the expected maximum differential head across the river-side and the land-side levees would be in the order of 2 feet. Durations of flood stage range from two to six weeks. Refer to tailrace rating curves in Appendix B, Hydrology and Hydraulics Design, for Santee River flood stages and durations. No provision for seepage control was made in the land-side levee embankment because of the low differential head and low height of levee. The impervious earth zone in the outer (north) slope of the river-side levee would effectively control seepage through the levee embankment under the low differential head. The sandy clay (CL) soils blanketing the flood plain would be expected to prevent significant underseepage through the foundations of either levee.

81. Erosion control. The tailrace canal levee slopes would not be riprapped. Erosion protection limits were selected based on velocity scour criteria for earth slopes outlined in EM1110-2-1602. Refer to Appendix B, Hydrology and Hydraulic Design, for the velocity profile in the tailrace canal. The tailrace canal levee slopes would be protected with grass.

82. Settlement. Settlement of the levee embankment crest is estimated to be in the range of 7 to 14 inches. Settlement calculations for the tailrace canal levee foundations and embankments are

presented in Appendix D, Slope and Levee Stability Analysis. Actual settlements expected are roughly one-half of the above estimates. The levee embankments would be instrumented at a few locations for horizontal movement and settlement.

83. Patrol roads. Patrol roads would be provided from the Powerhouse to the Santee River along the crests of both tailrace levees for inspection and maintenance of the levee slopes, the canal cut slopes and, in the case of the land-side levee, the interior drainage ditch and waste disposal area slopes. The patrol roads would be 10 feet wide, crowned to drain and surfaced with a thin layer of gravel or a suitable base course-type material. The river-side levee patrol road would have a turnaround at the downstream end. The land-side levee patrol road would connect with the Lake Mattassee access road over the interior drainage drop structure at about canal Station 596+00.

EXCAVATED MATERIAL DISPOSAL AREAS

84. Intake canal. The major portion of the intake canal excavated materials would be excess to the volume of material required to build the levees. Material usage is shown on Table 2. The excess material would be disposed behind the levees in the disposal areas shown in plan on Plates 6 through 8 and in section on Plates 13 and 14. The excavated materials would be placed directly into the disposal areas by dragline or hauled into the areas by truck or pan. Maximum haul distance required for disposal would be about one-half mile. No spreading or compaction of material is planned except for normal operation of hauling equipment. Temporary drainage and control of surface water run-off during construction would be required. Lift thicknesses and moisture content would not be controlled. The disposal fill slope against the levee was set at 1V on 5H and the top surface of fill would be sloped to drain at 1 percent away from the levee. The back (outside) slope of the disposal fill would be 1V on 10H. Suitable embankment material for the road relocation bridge crossings would be stockpiled in the disposal areas adjacent to the crossings if the road relocations are not accomplished before canal construction. All disposal fill surfaces would be trimmed and dressed and planted with brown-top millet and Pensacola bahia for erosion control and wildlife forage. The disposal areas would be forested where adjacent lands are wooded and along the road relocations to screen the disposal areas from view. Run-off drainage from the fill surfaces would be intercepted at intervals by shallow ditches (swales) perpendicular to the canal

and directed laterally to the interior drainage ditch or toe ditches to natural drainage basins. The effects of the disposal areas on existing ecosystems are discussed in paragraph, "Effects on Fish and Wildlife", in this report. Effects of project construction on archeological resources are discussed in paragraph, "Archeological Studies". The intake canal alignment, excavation and disposal areas were coordinated with ecological and archeological agencies as discussed in section, "Co-ordination with Other Agencies".

85. Tailrace canal. A large portion of the material excavated from the tailrace canal between the powerhouse and SCL railroad (canal Station 419+00) is excess to the volume requirements for construction of the tailrace levees. Materials usage is shown in Table 2. The excess material would be disposed behind the levees in the disposal areas shown in plan on Plates 9 through 12 and in section on Plates 14 and 15. Downstream from the railroad there would be only a small excess of excavation material over and above the levee embankment construction requirements. All excess material from the tailrace canal below the railroad would be disposed behind the south (land-side) levee. The materials would be placed directly into the disposal areas by dragline or hauled into the areas by truck or pan. Maximum haul distance required for disposal would be about one-half mile. No spreading or compaction of material is planned except for normal operation of hauling equipment. Lift thicknesses and moisture content would not be controlled. The disposal fill for interior drainage ditch slopes in the disposal areas would be placed and compacted in thin lifts, the same procedures as used to construct the levees. Temporary drainage and control of surface water run-off during construction would be required. The disposal fill slope against the levee would be 1V on 5H. The top surface of the fill would be sloped to drain toward the interior drainage collector ditch at 1 percent. The outside slope of the disposal fill between the powerhouse and the railroad would be 1V on 10H. All disposal fill surfaces would be trimmed and dressed by bulldozer or other suitable equipment, and planted with brown-top millet, Pensacola bahia and native trees to restore the areas to wildlife habitat and to control erosion. The amount of disposal area acreage in the Santee River floodplain has been significantly reduced over the General Design Memorandum canal design plan in keeping with recommendations of fish and wildlife interests made during co-ordination of the tailrace canal design with other agencies. The effects of the disposal areas on existing ecosystems are discussed in paragraph, "Effects on Fish and Wildlife", in this report. Effects of tailrace canal construction on archeological resources are discussed in paragraph, "Archeological Studies". The tailrace canal alignment, excavation and disposal areas were co-ordinated with interested ecological and archeological agencies as discussed in section, "Co-ordination with Other Agencies".

86. Powerhouse. Disposal of excavated materials in and around the powerhouse area (canal Station 339+60 to Station 367+60) is presented in the Powerhouse Foundation Analysis, February 1976. Excavation and backfill quantities within the above area approximately balance; however, provision has been made for excess material disposal in, or borrowing material from, the disposal areas presented in this report. Locations for temporary stockpiles of powerhouse (and stockpiles of other) construction materials have been considered in establishing project boundaries in the powerhouse area. Unsuitable materials from the powerhouse excavation will be disposed within the project limits shown on Plate 9.

87. Entrance channel. Dredged material from the Entrance Channel (canal Stations 0+00 to 135+34) will be disposed in a separate diked area immediately adjacent to the north intake canal disposal area at Lake Moultrie. Design of the disposal area, including ecological effects and restoration treatment, is presented in Entrance Channel in Lake Moultrie DM No. 3, March 1974.

INTERIOR DRAINAGE DESIGN

88. Intake canal collector ditch. The locations of the two intake canal interior drainage ditches are shown on Plates 6 through 9. The maximum height of cut slope in the two ditches would be 12 feet. Slope values for various heights of cut slope were developed by stability analysis as follows:

Cut slope height	Slope value
0-9 feet	1V on 2H
9 feet and above	1V on 3H

Ditch invert elevations are shown in profile on Plates B-29 through B-31. Maximum gradient of the ditch bottom would be 0.0048.

89. Tailrace canal collector ditch. The location of the tailrace canal collector ditch is shown on Plates 9 through 12. The ditch would be in cut and fill with the invert at elevation 22 msl. Slopes would be set at 1V on 3H to be stable in the disposal area fills. Maximum height of slope would be approximately 11 feet. The ditch is shown in profile on Plate B-32. The gradient of the ditch bottom would be virtually zero.

90. Culverts at road crossings. Culvert structures would be required where roads cross the proposed interior drainage ditches along the south side of the intake canal. These ditches would be required because the natural drainage, which is generally northward to the Santee Swamp, would be interrupted by the proposed levee on the south side of the canal. The natural drains which would be intercepted are lower than the normal water level in the intake canal. The recommended drainage system with two ditches running in opposite directions, begins from an existing divide between S. R. 35 and S. R. 45. Flows in the ditch running northeast with the intake canal would be ultimately routed to the Tailrace Canal. Flows in the second ditch would run counter to the intake canal flow and would eventually be released in a natural drain south of Russellville. The proposed ditches and culvert locations are shown on Plates 6 through 12 and on Plate B-12 in Appendix B. Descriptions of the recommended culverts are shown in Table B-16. The relatively small box culverts at S. R. 35, lower crossing, are recommended because of the limited head room. Approach and outlet channels at this crossing would have to be widened considerably to accommodate properly spaced multiple circular pipes. All culverts except two were designed to pass the 50-year frequency flood. The two culverts were designed to pass the 10-year flood because of low road elevations.

91. Winged inlet and outlet headwalls would be provided on the circular culverts and winged headwalls with concrete aprons would be used on the box culverts. Because of the flat ditch grades and resultant low velocities, no significant scour or erosion problems are expected at the culverts.

92. Structural design for the culverts was developed for this report only to the extent necessary to obtain a recommended arrangement and a proper cost estimate. Final culvert requirements will depend on the number and locations of bridge crossings over the intake canal. Recommendations for these crossings will be developed in the forthcoming Primary and Secondary Road Relocations DM. The box culverts will be designed for H20 Highway loading in accordance with procedures outlined in EM 1110-2-2902. Circular culverts will be designed to conform to the required standards of the South Carolina Highway Department.

93. Drop structure design.

a. General. This section covers the structural design for the reinforced concrete baffled chute drop structures used in the interior drainage system. Two structures would be used in the intake drainage ditch - one southeast of intake canal Station 350+00 to lower the ditch flows into the ravine southeast of the powerhouse access road and another at tailrace canal Station 409+00 to lower discharges from the ravine outlet into the tailrace. The remaining drop structure would be located at the tailrace access road and would be used to lower flows collected in the tailrace drainage ditch into the tailrace. See Plates 8, 9 and 12 for locations of drop structures.

b. Design criteria. The drainage structures are designed in accordance with procedures outlined in the following references:

a. EM 1110-1-2101 - Working Stresses for Structural Design

b. EM 1110-2-2103 - Details of Reinforcement - Hydraulic Structures

c. EM 1110-2-2400 - Structural Design of Spillways and Outlet Works

d. "Design of Small Canal Structures" - U. S. Department of the Interior, Bureau of Reclamation

e. "Building Code Requirements for Reinforced Concrete" - A.C.I. 318-71

f. EM 1110-2-2502 - Retaining Walls

c. Working stress design is used for the drainage structures. The minimum specified compressive strength for reinforced concrete is 3,000 psi at 28 days. Reinforcing steel conforming to ASTM A615, Grade 40 or Grade 60, with a basic stress of 20,000 psi is specified.

d. Water pressures, uplift, and velocity forces used in the design are based on the 50-year frequency storm with minimum coincident water levels in the tailrace where applicable.

e. Description of structures. Each drop structure consists of upstream and downstream wingwalls, a horizontal approach chute and a baffled sloping chute. The wingwalls are designed as inverted-tee retaining walls and would be separated from the spillway proper by contraction joints. The side walls of the approach and baffled chute are designed to act with the base slab as U-frames. To obtain the required stability against sliding and overturning, the horizontal and sloping portions of the structure are designed as a simple unit without contraction joints. Site Plans and design data are shown on Plate 16.

f. Plate 17 shows typical structural details for the structure below the powerhouse access road and sample computations are included in Appendix E. Borings have not been made at the immediate site but several borings in the vicinity indicate that this structure would be founded on materials varying from clayey sands to very firm, cemented fine sands. Cutoff keys along the upstream and downstream edge of the structure would be needed to develop the required sliding resistance. A portion of each wingwall would be cast monolithicly with the sidewall and cutoff key to provide a longer key and in order to reduce its required depth. To better accommodate this cutoff extension, the upstream wingwalls on this structure would be built at 90 degrees to the channel centerline. Nearby borings indicate that the lower portion of the other two drop structures would be founded in rock where adequate sliding resistance would be obtained with a nominal key along the downstream edge. Upstream wingwalls at these structures would be oriented at 45 degrees to the channel. Additional borings will be made during preparation of construction plans to confirm the design assumptions.

g. Riprap. Riprap would be provided upstream and downstream of the drop structure to protect against erosion. Riprap protection would also be provided in the area subject to splash outside the sloping chute walls. Refer to Plate 16 for locations of riprap slope protection. A depressed area would be provided in the outlet channel immediately downstream of the structure to give an increased tailwater depth and thereby help dissipate the energy of the flowing water as it leaves the sloping chute. This area would be lined with a thickened layer of riprap to insure that the sliding resistance at the downstream cutoff key is not reduced by erosion. Riprap has been designed in accordance with ETL 1110-2-120, May 1971. Filter design is in accordance with EM1110-2-1901 and TM 5-820-2 (see Appendix A, Geology and Soils).

h. Spillway bridges. A one-lane bridge would be provided across the apron section of the two lower drop structures to provide continuous access along the levee patrol road. The bridge would be designed for standard HS-15 loading. The bottom of bridge slab would be set to clear the water surface in the tailrace resulting from the 10-year flood flows in the Santee River.

i. Safety. Metal post and rails faced with chain link fence would be provided around the open top of the drop structures where a potential hazard to the public or to large animals exists.

j. Instrumentation. Lead plugs would be installed in each drop structure to allow monitoring of horizontal and vertical movement of the structure components.

EARTHQUAKE DESIGN

94. General. The project powerhouse and canals are located in a known earthquake area; therefore, consideration of future seismic activity is a necessity in the design of project structures. This section summarizes the steps taken to develop an earthquake-resistant design for project features.

95. Critical structures. After considering the consequences of failure due to a seismic event, the stability of the following structures is considered critical to the operational integrity of the project.

a. Powerhouse, including intake and tailrace retaining walls.

b. Intake canal levees from Station 339+00 to 360+00.

Critical structures are those whose failure could possibly result in loss of life or loss of pool level by partial draining of Lake Moultrie (and Lake Marion).

96. Loss of life. An examination of the Santee River floodplain reveals no human habitation either below the project or in areas adjacent to the canals below normal lake level (elevation 75 msl) except near Station 339+00. The human habitation near Station 339+00 is at elevation 72 msl; therefore, the project would not pose a significant threat to human life in the event of uncontrolled release of water from Lake Moultrie due to failure of a project structure under earthquake loading.

97. Loss of pool. Loss of pool level would be limited to pool levels above elevation 64 msl, the controlling invert of the Entrance Channel. Normal pool level would be elevation 75 msl. Minimum power pool level would be elevation 60 msl. The historic low pool level (elevation 69.6 msl) occurred in January 1956 during a severe drought. Provision would be made in the Powerhouse Foundation Analysis, February 1976, to stockpile closure rockfill near the powerhouse to close any breach that might occur in the intake canal levees at the powerhouse. The levees would be naturally buttressed downstream by a broad stable ridge with crest elevations up to elevation 72 msl. Complete failure of the massive abutments of the powerhouse is highly unlikely.

98. Non-critical structures. Non-critical structures are those whose failure would result only in increased maintenance to restore the structure to design efficiency. The following project features are considered non-critical structures:

Entrance channel

Intake canal cut slopes

Intake canal levees (except Station 339+00 to 360+00)

Tailrace canal cut slopes

Tailrace canal levees

Interior drainage ditches, drop structures and culverts

SCL Railroad bridge

Highway bridges

Powerlines

In the event of failure of non-critical structures, power production at the project may be interrupted; however, replacement power can be produced by increasing flow through the Jefferies Hydro Plant at Pinopolis. If the Jeffries plant is also damaged, replacement power can be obtained from other plants in the Authority's system, or from adjacent power systems, until repairs are made. All structures presented in this report are considered non-critical from an earthquake standpoint. The design of intake canal levees between Stations 339+00 and 360+00 is presented in the Powerhouse Foundation Analysis, February 1976.

99. Earthquake design criteria. The critical structures (Powerhouse and intake canal levees between Stations 339+00 and 360+00) have been designed according to criteria outlined in EM1110-2-2200 and EM1110-2-1902, April 1970, for Zone 3 seismic forces (see Plate 26). The seismic coefficient used in design of the critical structures except the powerhouse was 0.15. For seismic coefficients used in the powerhouse design see DM 7, Preliminary Design Report, St. Stephen Power Plant. Lesser seismic coefficients in the range of 0.05 to 0.12 were used in design of the non-critical structures. In other words, the non-critical structures would be stable under earthquakes up to Modified Mercalli magnitudes of IV or V but could fail under higher magnitudes. The seismic coefficients used to design the structures in this report, all of which are non-critical, are shown in Appendix D, Slope and Levee Stability Analyses, on Plates D-14 through D-36.

100. Earthquake design features. The following supplemental improvements would be included in the project as a result of earthquake design studies:

a. Widened berms between canal cut slopes and levees in the intake and tailrace canals.

b. Internal embankment drainage and flattened slopes on intake canal levees between Stations 339+00 and 360+00 (refer to Powerhouse Foundation Analysis, February 1976).

c. Emergency closure rockfill near the Powerhouse (refer to Powerhouse Foundation Analysis, February 1976).

101. Instrumentation. The Powerhouse and its abutments would be instrumented to record structural response to seismic activity. The instrumentation system will be presented in the forthcoming feature

design memorandum Instrumentation. Since no faults or other evidence of displacement have been detected by drilling, geologic mapping or examination of aerial imagery, no instrumentation of the canals, levees or powerhouse for earthquake displacement is planned. The above structures would be instrumented for normal settlement and horizontal movement.

CONSTRUCTION CONSIDERATIONS

102. Construction procedure. Construction of the project features is planned to proceed in the following sequence:

Access Roads and Construction Facilities

Entrance Channel and Disposal Area

SCL Railroad Relocation

Tailrace Canal

Powerhouse

Road Relocations

Intake Canal

Temporary drainage and interior drainage facilities would be constructed as required to accomplish control of surface waters during construction of major project features. Blockouts would be used for highway and railroad traffic continuation where canal construction would precede relocation construction. The last item of work in the intake canal would be removal of a canal plug and the existing dike at Lake Moultrie to open the Intake Canal to water flow from the lake.

103. Excavation methods. The cost estimate in this report is based on dragline excavation of the intake and tailrace canals. Scrapers and pans could be used to advantage in the Intake Canal to excavate material above the groundwater table for disposal in the levee embankments. Seasonal wet conditions may make scraper/pan operation difficult in the tailrace canal area since the groundwater table is very close to the ground surface. The hard rock to be excavated in the Intake Canal is expected to require pre-blasting to break it down for dragline excavation. Some short hauling (up to one-half mile) of excess excavated material is planned in the intake and tailrace canals to distribute excess material into more favorable disposal configurations. See Table 2 for material distribution plan. The configurations of the intake canal disposal areas were developed by balancing excess material disposal between natural barriers, i.e. the major highways, and setting elevation 98 msl as

the top of stable fill. Most of the rock to be excavated from the Tailrace Canal is soft enough for dragline excavation. In order to excavate the hard sandstone at the SCL railroad, the dragline may have to expose the outcrop face by excavating soil and soft rock, then lifting the hard rock at natural planes of weakness (bedding planes, joints, etc.). The largest volumes of excess material from the Tailrace Canal occur between the powerhouse and the SCL railroad (Station 419+00), and at the confluence of the Tailrace Canal with the Santee River. In these reaches the excess materials would be re-distributed by approximately one-half mile hauls to adjacent volume-deficient disposal areas in order to minimize disposal area encroachment into the floodplain timber and wildlife habitat. Hauling would be accomplished by truck, pan or other suitable equipment picking up material from temporary stockpiles or directly loaded by dragline. Excavation of the Intake Canal by dredge (with disposal in diked areas) would be possible even in the event the entrance channel had not already been dredged, since portable dredges are available. Dredging of the Tailrace Canal would not be feasible because excavated material volumes would not be sufficient to build the tailrace canal levees with flat hydraulic fill slopes between disposal dikes.

104. Construction materials.

a. Embankment. Soil and rock for embankment construction would come from the intake and tailrace canals excavations. Sufficient suitable impervious soils are available in the intake canal to construct homogeneous, impervious levee embankments. Impervious soils in the tailrace canal excavation are sufficient to construct an impervious zone in the outer (north) slope of the river-side tailrace levee. The remainder of the tailrace levee embankments would be constructed of granular soils and soft rockfill from the tailrace canal excavation.

b. Stone slope protection (riprap). Rock of suitable quality and gradation for riprap is available from Columbia, South Carolina, a distance of approximately 70 miles. Granite quarries in the Columbia area have been approved in the recent past for jetty stone production for Tybee Island Project in Georgia (Savannah District). A sampling program for approval of riprap sources and a compilation of existing data on rock quality and gradations will be presented in forthcoming feature design memorandum, Construction Materials.

c. Filter/bedding material. Sand of suitable gradation for filter/bedding material is available from commercial processing plants within 20 miles of the project. Production of the filter/bedding gradations from sand deposits at the project site would require processing of the sand as it is excavated from the tailrace canal or powerhouse.

d. Concrete. Ready-mix concrete is available from commercial producers within 20 miles of the project site. The powerhouse contractor is expected to be on-site with his concrete batch plant before the canal construction is complete. Fine and coarse aggregate are available from commercial producers within 30 miles of the project site. The aggregate sources currently meet South Carolina State Highway Department standards for concrete aggregate. A sampling program for approval of aggregate sources and a compilation of existing data on quality and gradations will be presented in forthcoming feature design memorandum Construction Materials. Cement is available from commercial plants near Harleyville, South Carolina, a distance of about 30 miles.

e. Reinforcing steel. Re-steel is available from commercial firms within 20 miles of the project site.

f. Concrete pipe. Concrete pipe for interior drainage system culverts is available from manufacturers in Charleston, South Carolina or Columbia, South Carolina, within a maximum distance of 70 miles.

TEST FILLS AND TEST EXCAVATIONS

105. Test fills. No formal test fills are planned. Crude test fills would be constructed to simulate disposal area construction operations during the test excavation program described below.

106. Test excavations. A test excavation into limestone rock is planned at approximate canal Station 280+00 in the intake canal. Two test excavations are planned in the tailrace canal - one near the existing SCL railroad (near canal Station 420+00) and another near the tailrace exit (approximate canal Station 595+00). The purpose of the excavations is to confirm estimated unit costs of small draglines excavating soil and rock from the canals. The test excavations would be performed during preparation of plans and specifications after submittal of detailed test program plans for review and approval.

COORDINATION WITH OTHER AGENCIES

107. Effects on fish and wildlife. The effects of project construction on fish and wildlife resources were given significant

consideration in selecting a canals alignment and selecting disposal areas. Alignment locations and disposal plans were co-ordinated with the U. S. Fish and Wildlife Service (USFWS), the National Marine Fisheries Service (NMFS), the Environmental Protection Agency (EPA), and the South Carolina Wildlife and Marine Resources Department (SCWMRD). A letter to the USFWS dated 10 March 1975 and a letter dated 30 September 1975 solicited their comments on alternative alignment locations and disposal plans. Alternative alignments are shown in plan on Plate C-1 in Appendix C, Alternative Studies. In a letter reply dated 22 July 1975 the U. S. Fish and Wildlife Service recommended the direct route (alignment A) for the intake canal, and in a subsequent letter dated 12 January 1976 USFWS recommended the meandering route following the natural drainage features through Lake Mattassee (alignment D) for the tailrace canal USFWS co-ordinated their comments and recommendations with NMFS, EPA and SCWMRD. The proposed intake canal and tailrace canal alignments essentially reflect the preferences of the concerned State and Federal fish and wildlife agencies. Co-ordination letters are presented in Exhibit B, Co-ordination Correspondence. Effects of project construction on fish and wildlife resources are detailed in the correspondence.

108. Archeological studies. The canals alignment proposed in the General Design Memorandum was surveyed for archeological sites by the University of South Carolina Institute of Archeology and Anthropology in 1974. In a letter dated 7 July 1975 the Institute described two sites they consider to be of historical importance (see sites 38BK83 and 38BK74 on Figure 2 in Exhibit B and on Plate C-1 in Appendix C) within project limits and informed the Corps of a proposal they had made to the National Park Service for survey and preservation funds. In a letter dated 17 October 1975 the Institute submitted their proposal to the Corps. In a letter dated 30 March 1976 the Institute was informed by the Corps that the proposal did not meet the requirements of the Corps outlined in 33 FR 41636-41641, Cultural Resources Identification and Administration, and was furnished a recommended revised scope of work for a cultural resources survey. The Corps recommended in the letter that further survey work be concentrated in those segments of the proposed canals which vary from the GDM alignment covered in previous survey work. Co-ordination letters are presented in Exhibit B, Co-ordination Correspondence. The excavated material disposal plan for the Tailrace Canal at the confluence with the Santee River was altered from the recommended plan in the GDM in order to preserve the nearby Indian relic site (38BK83).

109. Groundwater monitoring program. A program of continuous monitoring of groundwater levels at the project site has been co-ordinated with U. S. Geological Survey. The program began in September 1973 immediately after installation of the system of 10-inch diameter observation wells (see Plate 18 for well locations). It provides annual funds to USGS for collecting and evaluating monthly observation well readings at the project site. USGS has reduced the number of continuous recording wells from twenty to thirteen and digitized the recorded

data. The monitoring program is planned to continue until three years to five years after project completion. An annual report will be submitted by USGS containing groundwater level records and an evaluation of the effect of significant groundwater changes on the project area. The annual cost of the monitoring program in FY 76 was $15,000, including the annual USGS report.

MAINTENANCE

110. Intake canal maintenance. Maintenance of the intake canal would consist of (a) mowing the berms and the levee slopes and (b) minor filling and shaping of areas eroded by surface water runoff. The excess material disposal areas would be planted with wildlife forage; therefore, they would not be mowed. Minor erosion of the canal cut slopes would be tolerated without repair. Design flow velocities in the intake canal were set at 3.5 feet per second to minimize cut slope erosion and shoaling in the canal. Maintenance dredging would not be required in the intake canal. Bridge clearances at the road crossings will be made sufficient for barge-mounted crane access to the Powerhouse.

111. Tailrace canal maintenance. Mowing of the berms and the levee cut slopes, and minor filling and shaping of areas eroded by surface water runoff would be the primary maintenance items in the tailrace canal. Minor erosion of the canal cut slopes (including interior drainage ditch exits) would be tolerated without repair. Re-shaping and/or minor repair of the riprap/drainage layer system below the Powerhouse may be necessary during the first year or two of operation since the slope drainage and the riprap layers would be adjusting to the fluctuating tailwater levels and velocities. Minor shoaling is anticipated in the exit reach (Station 595+00 to end of construction) of the tailrace canal; however, canal flow volumes and velocities are considered sufficient to prevent build-up of shoaling to an extent that maintenance dredging would be required.

112. Interior drainage system maintenance. Mowing, brush cutting and debris removal would be required maintenance in the intake canal and tailrace canal interior drainage ditches, culvert crossings and at the three concrete drop structures. Minor filling and shaping of areas eroded by surface water runoff would also be required along the ditches and around the culverts and drop structures.

WELL INVENTORY

113. Well inventory. An inventory of private and municipal wells within the influence of the project construction has been assembled by Charleston District personnel. Well data (including location, depth, pump setting, type of pump, water level and well capacity) was obtained for deep and shallow wells within the project limits (as well as for wells outside project limits) that could be affected by canal and powerhouse construction. Construction effect distances were estimated from pump test data presented in the Site Selection and Geology DM No. 6, May 1975. Wells were inventoried up to a distance of 9,000 feet from the project centerline. In many instances complete well data was not available from the well owner or the well driller and some data was estimated from measurements made during the inventory. The complete well inventory is presented in Appendix F and will be recorded on computer data cards for U. S. Geological Survey's information retrieval system.

RELATED FEATURES

114. Real Estate. Project right-of-way requirements and associated real estate acquisition for the Entrance Channel in Lake Moultrie and the two project access roads were presented in Real Estate Design Memorandum, Area 1, DM No. 5 dated September 1974. The right-of-way requirements for the proposed Intake and Tailrace Canals in this report are shown on Plates 6 through 12. The total acreage within the proposed project right-of-way lines for the Intake and Tailrace Canals and Powerhouse is estimated to be 2,175 acres. Property identification and associated real estate acquisition costs will be presented in forthcoming Real Estate Design Memorandum, Area 2, DM No. 12.

115. Transmission line along tailrace canal. The route of the proposed transmission line (as presented in the GDM) connecting the Powerhouse with the existing SCPSA Kingstree transmission line lies outside the project right-of-way limits proposed in this report. Consideration has been given to routing this connector line along the Tailrace Canal within the proposed project right-of-way limits. A detailed study of such routes along the tailrace canal will be prepared and the results presented in forthcoming Powerhouse Feature Design Memorandum No. 7.

116. Road relocations. Road relocation bridge design and design of associated road embankments will be presented in the forthcoming Primary and Secondary Road Relocation DM No. 10. Disposition of

county roads and provisions for mitigation of severed access to the Santee River floodplain lands will also be presented in the above DM. Construction of the road bridges over the Intake Canal is planned to precede intake canal construction. If the bridges cannot be constructed first, then blockouts would be provided in the canal at the bridge locations. Suitable road embankment fill material can be obtained from the upper portion of required canal excavation in the vicinity of each bridge site. If the canal is constructed before the bridges, suitable road embankment fill material would be stockpiled for road embankment construction. The three road relocation bridge crossing sites (at S.C. 35, S.C. 45 and U.S. 52) proposed in the General Design Memorandum No. 1, January 1972, are shown on Plates 6, 7 and 8.

117. <u>SCL Railroad relocation</u>. Design of the SCL railroad bridge crossing over the Tailrace Canal will be presented in the forthcoming Seaboard Coastline Railroad Relocation DM No. 8. Construction of the detour track embankment and the bridge are planned to precede construction of the Tailrace Canal. Approximately 50 feet of length of the canal would be excavated under the bridge as part of the railroad relocation contract. The excavated materials from the canal would be too wet to be suitable for detour embankment fill. The relocation site and the proposed borrow area for suitable embankment fill material are shown on Plate 9. If the Tailrace Canal is constructed before the railroad relocation, approximately 300 feet of canal length would be reserved for the relocation construction.

118. <u>Utility relocations</u>. Details of utility relocations will be presented in the forthcoming Utilities Relocation DM No. 11. The powerline relocation sites are shown on Plates 6 and 11.

119. <u>Fish hatchery</u>. Design of the fish hatchery will be presented in the forthcoming Fish Hatchery DM. A tentative location for the hatchery has been selected approximately 800 feet east of the Powerhouse (see Plate 1). The hatchery site location will be coordinated with the State of South Carolina Wildlife and Marine Resources Department after this report and the Powerhouse Site Plan Feature Design Memorandum have been approved.

120. <u>Fish barrier</u>. A tentative site location and a conceptual design of a fish barrier in the Santee River at the tailrace canal exit were presented in GDM Supplement No. 1, Comparison of Alternative Plans, October 1973. As proposed in the above report, final design and construction of the fish barrier will await completion of post-project fish and wildlife studies; therefore, the tailrace canal design presented in this report does not include the fish barrier. The barrier structure concept presented in the above-referenced report can be incorporated into the project without modification to the proposed tailrace canal design.

ESTIMATED COSTS

121. Cost estimate. The total estimated cost to the Federal Government for the project features presented in this design memorandum is $24,770,000. This is the total initial cost for all of the project features outlined in the detailed cost estimate in Exhibit A. The costs used in the estimate are based on the drawings presented in this design memorandum and on the assumptions outlined in the foregoing paragraphs. The unit prices applied are current contract prices. The estimated acreage for "foresting" was based on re-foresting all of the tailrace canal disposal areas with seedlings plus 25% of the intake canal disposal area. "Excavation" was split into "common" and "rock" for this report; however, the contract bid item would be labeled "Unclassified Excavation". The term "rock" includes only rock excavation requiring blasting. No blasting is anticipated in the Tailrace Canal. Fifteen percent has been added to the contract cost for contingencies and 14.5 percent for Government costs (8.5 percent for engineering and design and 6.0 percent for supervision and administration).

DEPARTURE FROM GENERAL DESIGN MEMORANDUM

122. Intake and tailrace canals alignment. The proposed alignments for the intake and tailrace canals presented in this report are modified in location from the canal alignments presented in the GDM, January 1972. The changes in alignment location resulted in lower cost and more environmentally acceptable canal alignments. The changes in alignment required a 26 degree clockwise rotation of the Powerhouse about its center from the powerhouse orientation recommended in Site Selection and Geology, DM No. 6, May 1975. Based on tentative preliminary approval of the alignment changes in SADEN-GK 1st Ind. dated 9 June 1975, subject "Intake and Tailrace Canals DM, Cooper River Rediversion Project", the powerhouse rotation has been included in the Powerhouse Preliminary Design Report, January 1976.

CONCLUSIONS

123. Conclusions. The following conclusions were developed during the design studies for this report:

a. Studies performed for this report have adequately considered canal and interior drainage design alternatives.

b. The intake and tailrace canals alignment proposed in this report is the most economical alignment and has the least adverse environmental impact of alternatives considered;

c. The proposed interior drainage system is the most economical and feasible method of handling intercepted drainage of the alternatives considered;

d. The canal design adequately considers earthquake effects on the canal and levee structures.

RECOMMENDATIONS

124. It is recommended that the proposed intake and tailrace canals alignment, disposal plan and interior drainage plan be approved as the basis of preparation of plans and specifications for these project features.

EXHIBIT A

COST ESTIMATES

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

DETAILED COST ESTIMATE
(June 1976 Price Levels)

Cost Account	Feature	Unit	Quantity	Unit Price	Total Cost
08.	ROADS (IN-PART)				
	Patrol Roads	LS	Job		$52,000
	Contingencies, 15%				8,000
	Account 08. Total				$60,000
09.	CHANNELS AND CANALS (IN-PART)				
	Clearing	Acre	1,750	$ 900	$1,575,000
	Clearing and Grubbing	Acre	200	1600	320,000
	Canal Excavation				
	Common	CY	14,300,000	0.77	11,011,000
	Rock (Requiring Blasting)	CY	140,000	3.50	490,000
	Levee Embankment				
	Stripping	CY	300,000	0.50	150,000
	Compaction	CY	4,730,000	0.22	1,041,000
	Stone Protection	Ton	35,500	25.00	888,000
	Sand Filter	CY	30,500	15.00	458,000
	Pervious Fill	Ton	57,000	23.00	1,311,000
	Interior Drainage Ditches	CY	270,000	0.60	162,000
	Exterior Drainage Ditches	CY	72,000	0.60	43,000
	Drop Structures	Ea.	3	125,000	375,000
	Culverts	LS	Job		140,000
	Instrumentation	LS	Job		10,000
	Grassing	Acre	1,250	600.00	750,000
	Foresting	Acre	600	60.00	36,000
	Account 09. Sub-Total				$18,760,000
	Contingencies, 15%				2,814,000
	Account 09. Total				$21,574,000
	Sub-Total (Accounts 08. and 09.)				$21,634,000
30.	Engineering and Design (8.5%)				1,838,000
31.	Supervision and Administration (6.0%)				1,298,000
	TOTAL COST				$24,770,000

EXHIBIT B

CO-ORDINATION CORRESPONDENCE

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

SUMMARY PROJECT COST ESTIMATE
(June 1976 Price Levels)

Cost Account No.	Item or Feature	Current Cost Estimate
08.	Roads (In-Part)	$ 60,000
09.	Channels and Canals (In-Part)	21,574,000
	Sub-Total	$21,634,000
30.	Engineering and Design (8.5%)	1,838,000
31.	Supervision and Administration (6.0%)	1,298,000
	Total Cost	$24,770,000

SANGR 10 March 1975

Regional Director
U. S. Fish & Wildlife Service
17 Executive Park Drive, N. E.
Atlanta, Georgia 30329

Dear Sir:

The purpose of this letter is to provide you with updated information on the Cooper River Rediversion Project and to solicit your views on the alignment of the Tailrace Canal. Additionally, it is requested that you provide certain information on the status of your studies.

Your office was advised late in FY 1974 that construction funds were not expected to be available in FY 1975. However, construction funds were later appropriated in FY 1975, and we anticipate an increased rate of funding during succeeding years. Construction is expected to begin in June 1975, and rediversion is planned for CY 1980.

On the basis of preliminary design studies, alternative alignments are now being considered for the Intake Canal and the Tailrace as shown on the attached map. Your previous comments on the disposal of excavated material from the entrance channel have been incorporated into the selection of the locations of the channel and the dredged material disposal area. Please evaluate the alignments in the following paragraph and rank them according to their effect on fish and wildlife resources.

Intake Canal Route A follows a direct route from Lake Moultrie to the powerhouse near St. Stephen, while Intake Canal Route B follows lowest ground between the lake and the powerhouse. Material excavated from either Intake Canal route will be placed immediately adjacent to the canal to form wide (up to 1,000 feet) levees on each side with flat side slopes. This area will be graded and vegetated and will be available for wildlife management, lumber production, agriculture, or other beneficial uses. Tailrace Route A follows a direct route from the powerhouse to the Santee River and would require widening of the existing run of the Santee River to Lake Mattassee. Tailrace Route B follows a direct route from the powerhouse to the Santee River at Lake Mattassee, and Tailrace Route C follows existing natural drainage features close to the natural bluff line. Material excavated from Tailrace Routes A and B will be placed immediately adjacent to the canal excavation to form wide levees on each side with flat

SANGR
Regional Director

10 March 1975

side slopes. Material from Tailrace Route C will be placed between the canal and the bluff line except for a portion that would be needed to construct a narrow levee along the flood plain (north) side of the canal to keep floodwaters out of the Tailrace Canal. The material placed on the south side will be spread out and transitioned into the existing highland so as to be well drained. This area will also be vegetated and available for selected beneficial uses as indicated above.

During the public hearing held on 4 April 1974, we were informed by a local representative of the Bassmasters Organization that the spring herring run in 1974 was small. It is requested that you give us an estimate of the size of the 1974 herring run and furnish a comparison with runs in other years. If there was a significant reduction in this run, is there any authoritative explanation for the reduction?

Reference is made to your letter of 20 March 1974 in which you submitted a preliminary outline for a 10-year study to begin in FY 1975. We would appreciate a copy of the final outline and any revisions to the study schedule. Also, we need to be informed as soon as possible of your proposed study plan and estimated costs for FY 1976 in order to be able to recommend transfer of funds by the Office, Chief of Engineers. It is understood from telecon with Mr. Jim Brown on 27 February 1975 that you received approximately $60,000 transfer funds for the Cooper River Project in FY 1975 and that almost all of this total was applied to purchase of fish counting equipment and services for counting fish at Pinopolis Dam during the 1975 fish runs. Confirmation of this information as well as an indication of your current work plans will be appreciated.

Sincerely,

1 Incl (dupe)
As stated

HARRY S. WILSON JR.
Colonel, Corps of Engineers
District Engineer

United States Department of the Interior
FISH AND WILDLIFE SERVICE
17 EXECUTIVE PARK DRIVE, N. E.
ATLANTA, GEORGIA 30329

July 22, 1975

District Engineer
U.S. Army Corps of Engineers
P.O. Box 919
Charleston, South Carolina 29402

Dear Sir:

Subject: Cooper River Rediversion Project, South Carolina

We have enclosed for your advance information a copy of our draft report on the subject project. Please note that this draft is preliminary and subject to revision after review by agencies that are concerned. Review is presently underway, and we plan to make final release of this report as soon as coordination is completed.

Sincerely yours,

John D. Green
Regional Supervisor
Division of Ecological Services

Enclosure

PRELIMINARY DRAFT
SUBJECT TO REVISION
NOT FOR PUBLIC RELEASE

DRAFT

District Engineer
U.S. Army Corps of Engineers
Charleston, South Carolina

Dear Sir:

This is in response to your letter of March 10, 1975, concerning alternative alignments of the intake and tailrace canals for the Cooper River Rediversion project. Our comments are submitted in accordance with provisions of the Fish and Wildlife Coordination Act (48 Stat. 401, as amended; 16 U.S.C. 661 et seq.).

Intake Canal Route A, as described in the final environmental impact statement, would follow a direct route 4 miles from Lake Moultrie to the proposed powerhouse site near St. Stephin. The canal would have a bottom width of 375 feet, and approximately 7,777,000 cubic yards of spoil material would be placed on each side of the canal to form levees up to 1,000 feet wide. Intake Canal Route B follows a longer route of approximately 5 miles along a natural low-lying drainage area. This route may require less excavation but would also require levees of up to 1,000 feet on each side of the canal and would incur the destruction of a large acreage of productive wooded habitat along the natural drainage area. For these reasons intake alignment A would be the least damaging of the suggested alternatives.

Three alignments are presented for the proposed tailrace canal consisting of a short canal (route A), a canal as described in the environmental impact statement (route B), and a meandering canal (route C) that follows natural drainage features to the Santee River. Although route A, the shortest route, would require less destruction of bottom-land hardwood wetland habitat along the overland route, the required widening and future maintenance of approximately 2.5 miles of the Santee River would be extremely deleterious to high-quality fishery habitat. In addition, it is apparent that the spoil resulting from widening of the river would probably be placed on adjoining bottom-land hardwood and wooded swamp habitat. Tailrace route C is the longest of the alternative routes (approximately 6 miles) and would follow a natural drainage feature that leads into Lake Mattassee. Construction of the canal along this route would eliminate most of the valuable wooded swamp habitat in the drainage and Lake Mattassee.

Tailrace route B would be the least damaging alternative provided no dredging or filling is conducted in the waters of Lake Mattassee. Lake Mattassee, with its adjoining wooded swamp composed of mature bald cypress, tupelo gum, and other associated vegetation, provides a diverse wetland habitat for various fish and wildlife species. Anadromous fishes, such as American shad and blueback herring, utilize the flooded swamp for spawning areas. In addition, Lake Mattassee is reported to be inhabitated by the American alligator, currently listed by the U.S. Department of the Interior as an endangered species. Therefore, any construction measurably affecting this area may not be in compliance with the Endangered Species Act of 1973.

In summary, to minimize the impacts of canal construction to fish and wildlife resources, this Service recommends the following:

1. the intake canal be constructed along route A; and,

2. the tailrace canal be constructed along route B with the necessary design modifications to insure that no dredging, filling, construction, or maintenance activities will affect Lake Mattassee or adjoining wetlands.

However, it should be emphasized that any of the tailrace canal alternatives would be destructive to the rich bottom-land hardwood and wooded swamp habitat in the area. From the dimensions given in the environmental impact statement, tailrace route B will revert a minimum of 970 acres of this wetland habitat into open water and spoil area. In view of these significant irreversible losses, it is also recommended that compensation for the loss of this valuable wetland habitat be provided at project expense. To this end, our future studies will provide a more concise description of the losses to be incurred by construction of the canal and other aspects of the project and recommendations for compensation of these losses.

We appreciate the opportunity to comment on this aspect of the Cooper River Rediversion project. It is hoped that these recommendations will aid in your selection of the least environmentally damaging rediversion canal route. We emphasize, however, that these comments should not be interpreted as endorsement of the ecological soundness of constructing the diversion canal or any other aspect of the Cooper River Rediversion project.

(Paragraph citing State review and concurrence)

Please advise us of action taken by the Corps of Engineers in this matter.

Sincerely yours,

Regional Director

SANGE 1 August 1975

Mr. John D. Green
Regional Supervisor
Division of Ecological Services
Fish and Wildlife Service
17 Executive Park Drive, N.E.
Atlanta, Georgia 30329

Dear Mr. Green:

This is in regard to your letter of 22 July which inclosed a draft report expressing your views of the alternative canal alignments for the Cooper River Rediversion Project.

We appreciate receiving your comments in advance of your final report; however, we note several areas which we believe need to be further discussed in detail prior to the preparation of your final report.

For instance, the statement concerning "significant irreversible losses" and recommendation that "compensation for the loss of this valuable wetland habitat be provided at project expense" is not in accordance with the findings of the U. S. Fish and Wildlife Service at the conclusion of their detailed study of the project in 1966. At that time, the report stated, "As a result, restoration of permanent swamps will occur downstream to U. S. Highway 17, thus providing improved wood duck breeding habitat, sanctuary for deer and turkey, and wintering areas for migratory waterfowl. In addition, hardwood timber growth rates and mast production will be improved and winter flooding will make these areas available to wintering waterfowl. As a result, wildlife values for these swamp and bottomland hardwood areas will be increased." Also, the report included the statement that "--- significant benefits will be provided for wildlife resources of the Santee River flood plain downstream from St. Stephen Canal, including the estuary waterfowl marshes."

We also do not believe that it is feasible to construct the project without effecting "Lake Mattassee" and believe that alignment "C" will have the least adverse effect on swamp hardwoods. The disposal area will be moved adjacent to the highland which will result in a narrower right-of-way and will move some of the disposal area out of the swamp-timber area and onto the pineland area.

I believe it would be mutually beneficial for our staff representatives to meet and discuss the proposed canal construction plans prior to completion of your report. I suggest that we meet during the early part of August and would appreciate your representative contacting our Mr. John Carothers, FTS 803, 577-4258 in order that we can schedule a meeting as soon as possible.

Sincerely,

HARRY S. WILSON, JR.
Colonel, Corps of Engineers
District Engineer

Copy furnished:
Dr. James A. Timmerman, Jr.
Executive Director
S. C. Wildlife & Marine Resources Dept.
P. O. Box 167
Columbia, South Carolina 29202

SANGE-S 30 September 1975

Regional Director
U. S. Fish and Wildlife Service
17 Executive Park Drive, N. E.
Atlanta, Georgia 30329

Dear Sir:

Reference is made to the 16 September 1975 meeting in our office with members of your agency concerning the Cooper River Rediversion Project. This letter and inclosures provide you the information you requested concerning the Tailrace and Intake Canal alignment and solicits your views with respect to the alignments presented herein.

The two tailrace alignments studied are "B" Modified and "C" Modified, shown on inclosures 1 and 2, respectively. These modified alignments, except for minor variations, are the same as furnished with our letter of 10 March 1975. Alignment "C" Modified is the same as the alignment "D" discussed at the 16 September meeting. Tailrace alignment "B" Modified (Inclosure 1) follows a direct route to the Santee River just above Lake Mattassee. This alignment requires widening and deepening about 2,800 feet of the river. This work would be done without disturbing the north bank of the river or Lake Mattassee (See Inclosure 1). Tailrace alignment "C" Modified (Inclosure 2) follows the existing natural drainage features close to the natural bluff line. This alignment is believed to be the most desirable from an esthetic standpoint and it also involves the minimum use of low land containing swamp timber; and since it follows the lower area through the swamp, the amount of excavation is less resulting in a smaller amount of excavated material to be disposed of. All excavated material will be placed on the highland side of the canal, except for the amount required to build a parallel dike of sufficient height to prevent flood waters from flowing into the tailrace canal.

As you requested, the two alignments have been compared on the basis of total number of acres of right-of-way required below the SCL railroad. This is further broken down into acres of high and low land within the right-of-way. Alignment "B" Modified requires 730 acres of right-of-way all of which is considered to be low land. Alignment "C" Modified requires 630 acres of right-of-way, of which 470 acres is low land. The water level in Lake Mattassee would be raised about 10 feet above the present normal stage during normal releases. For each alignment, areas which are above the water surface for the average steady state discharge condition (El.18.5 M.S.L.) are outlined in red on the inclosed maps. Those areas which would be above the water surface for the maximum steady state discharge from the powerhouse are outlined in green (El.21.5). Both elevations assume a 500 cubic feet per second discharge at Wilson Dam. The above elevations are the water surface elevations in that portion of the river shown on the inclosed maps. Inundation of the river flood plain above its junction with the tail-race will occur to a limited extent through low points in the river banks. We do not believe this inundation will be extensive since the river banks are steadily rising in elevation upstream and there would be fewer low points in the banks. The depth and duration of this flood plain inundation will vary depending upon the power generation requirements; i.e. peaking operations versus sustained generation for extensive periods.

In our letter to your office of 10 March 1975 we presented two intake canal alignments; Alignment "A" which is the direct route to the power-house and Alignment "B" which follows the lowest ground between the lake and the powerhouse. Alignment "D" was proposed by my staff at the 16 September 1975 meeting. All three alignments are shown on Inclosure 3. Alignment "D" avoids a major portion of the low lying habitat as was suggested by your office. It also requires slightly less excava-tion than Alignments "A" or "B" and avoids conflict with a new residen-tial area adjacent to S. C. Highway 45. Alignment "D" is not as favorable as Alignment "B" from a road relocation standpoint, however, considering all of the above factors, Alignment "D" appears to be the most acceptable and is presently our recommended alignment.

It is requested that your final comments concerning the proposed align-ments be furnished by 20 October 1975 in order to insure that they be given full consideration during preparation of the Detailed Design Report for the Rediversion Canal.

Sincerely,

3 Incls

HARRY S. WILSON, JR.
Colonel, Corps of Engineers
District Engineer

United States Department of the Interior
FISH AND WILDLIFE SERVICE
17 EXECUTIVE PARK DRIVE, N. E.
ATLANTA, GEORGIA 30329

Airmail

JAN 1976

District Engineer
U.S. Army Corps of Engineers
P.O. Box 919
Charleston, South Carolina 29402

Dear Sir:

This is in response to your letters of March 10, August 1, and September 30, 1975, and the meeting held September 16, 1975, with your staff concerning the alternative diversion canal alignments for the Cooper River Rediversion project. Our comments are submitted in accordance with provisions of the Fish and Wildlife Coordination Act (48 Stat. 401, as amended; 16 U.S.C. 661 et seq.).

With the additional new alignment D described to us at the September 16 meeting and your letter of September 30 providing additional information, we have reevaluated the alignments. Assessment of the different alternate routes has been narrowed to two basic tailrace canal routings: (1) a direct route to the Santee River intersecting above Lake Mattassee and, (2) a meandering route following the natural drainage features adjoining highlands and then through Lake Mattassee. According to your data the direct route, alignment B, would require 730 acres of right-of-way area consisting of river bottom land and the excavation of 2,300 feet of the Santee River. Alignment D would be confined along the highland route and its 630 acres of right-of-way would encompass approximately 430 acres of swamp and bottom land. Your data also indicate that whichever route is used the Lake Mattassee area will be inundated by 10 feet during normal discharge releases from the new powerhouse.

After meeting with your staff and reviewing the additional data, we agree that alignment D for the diversion canal would be the least damaging alternative. Therefore, to minimize the impacts of canal construction on present and future fish and wildlife resources, this Service recommends:

1. The Rediversion be constructed along Route D.

AMERICAN REVOLUTION BICENTENNIAL 1776-1976

2. All dredged material be placed on the highland side of the canal, except the minimal amount needed for construction of a dike to prevent flood waters from flowing into the tailrace canal.

3. Assurances be given that after construction and transfer of operations to the South Carolina Public Service Authority, the right-of-way be revegetated in natural vegetation and preserved in this state to aid in offsetting the losses, as stated in the final Environmental Impact Statement.

It should be emphasized that any of the tailrace canal alternatives would be destructive to the rich bottom-land hardwood and wooded swamp habitat in the area. From the data provided in your September 30, 1975, letter, alignment D will encompass a minimum of 470 acres of this wetland habitat. In view of these significant irreversible losses, it is also recommended that compensation for the loss of this valuable wetland habitat be provided at project expense. To this end, our future studies will provide a more concise description of the losses to be incurred by construction of the canal and other aspects of the project and recommendations for compensation of these losses.

We appreciate the opportunity to comment on this aspect of the Cooper River Rediversion project. It is hoped that these recommendations will aid in your selection of the least environmentally damaging rediversion canal route. We emphasize, however, that these comments should not be interpreted as endorsement of the ecological soundness of constructing the diversion canal or any other aspect of the Cooper River Rediversion project.

This report has been coordinated with the National Marine Fisheries Service, Environmental Protection Agency, and South Carolina Wildlife and Marine Resources Department.

Please advise us of action taken by the Corps of Engineers in this matter.

Sincerely yours,

Kenneth E. Black

Regional Director

UNIVERSITY OF SOUTH CAROLINA

COLUMBIA, S.C. 29208

INSTITUTE OF ARCHEOLOGY AND ANTHROPOLOGY

July 7, 1975

District Engineer
United States Army
Corps of Engineers
Charleston, South Carolina 29407

Dear Sir:

Recently Dr. Leland G. Ferguson of the Institute has submitted a proposal to the National Park Ser ice for funds to mitigate the impending damage to archeological resources resulting from the construction of the proposed Cooper River Rediversion Canal. We have received word from the Park Service that the proposal is under consideration.

There are two areas mentioned in this proposal that may eventually prove to be of enough importance to be nominated for the National Register of Historic Places. One of these sites is near the end of the tailrace of the proposed canal and the other is in the vicinity of the proposed power house.

The first of these sites is stratified and may well produce strata recording several thousand years of Indian occupation on the coastal plain. The second site is the location of the historic Peyre Plantation. This eighteenth century plantation was near the center of historically important St. Stephens Parish -- a settlement of French Hugenots. One of General Francis Marion's Revolutionary War hideouts is reported to have been on the Peyre Plantation below Murray's Ferry in St. Stephen's Parish. As a result, this plantation may have the double importance of being a typical French occupation as well as a "lost" Revolutionary War landmark.

In addition to these two sites there is a potential for finding other important sites within the impact area. Approximately five miles of the canal, as it is now proposed, has not been surveyed for archeological sites. Survey of this area may reveal sites of exceptional importance.

We appreciate the interest the Corps of Engineers has shown in helping locate and protect archeological and historical resources. South Carolina has lost a significant portion of the archeological record to riparian construction projects. We all have the responsibility of carefully protecting our remaining information about the past.

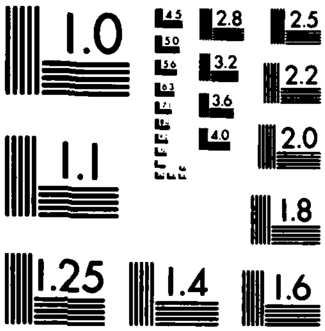
Sincerely yours,

Robert L. Stephenson
Director and State Archeologist

RLS:mls

cc: Advisory Council, National Register of Historic Places
Dr. Donald Crusoe, National Park Service, Atlanta
Mr. Charles E. Lee, S.C. Department of Archives & History
Dr. Leland G. Ferguson, Institute of Archeology

AD-A149 941 COOPER RIVER REDIVERSION PROJECT LAKE MOULTRIE AND 2/5
SANTEE RIVER SOUTH CAROLINA INTAKE AND TAILRACE CANALS
(U) CORPS OF ENGINEERS CHARLESTON SC CHARLESTON
UNCLASSIFIED DISTRICT JUN 76 F/G 13/2 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

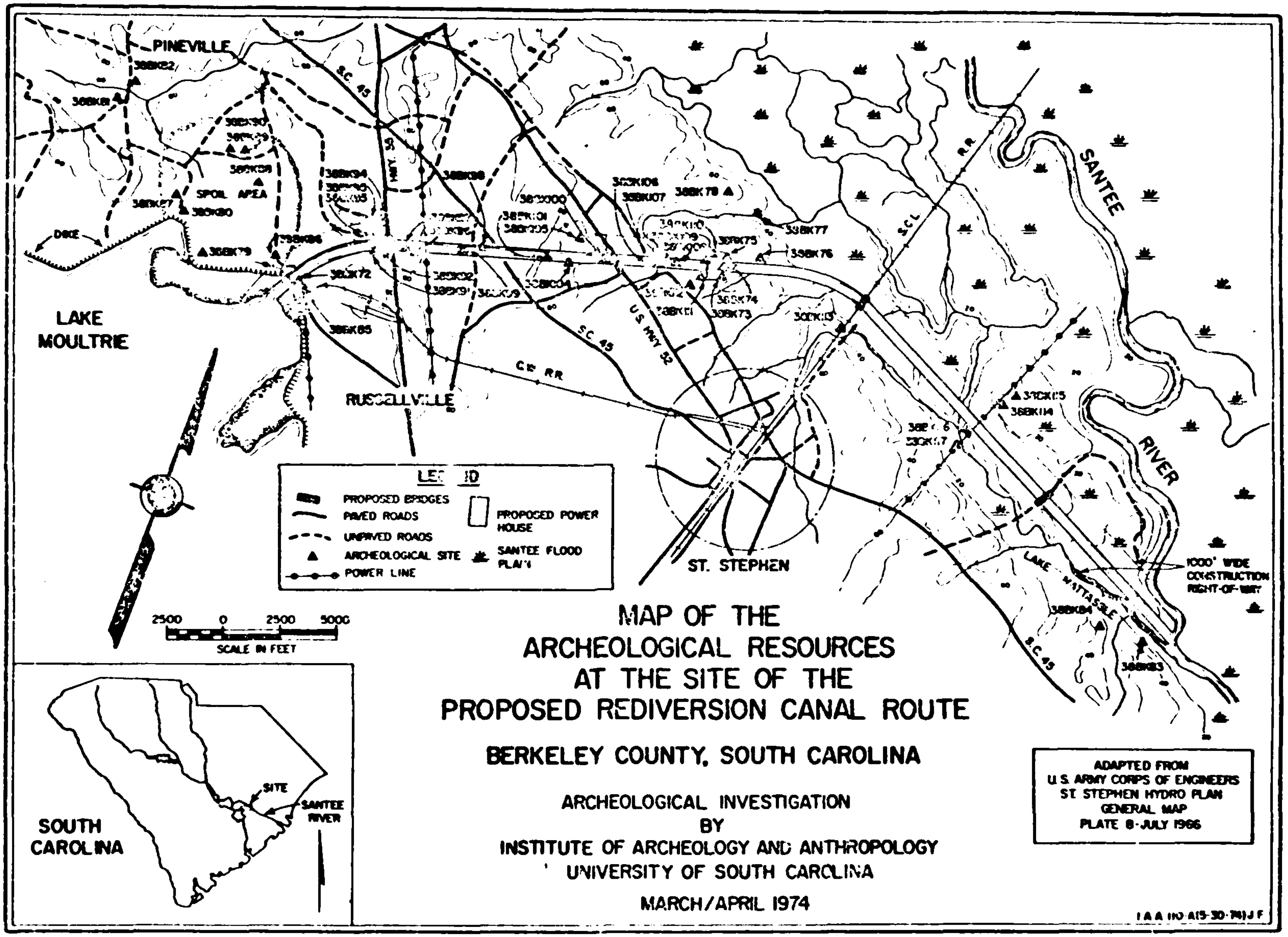

MAP OF THE
ARCHEOLOGICAL RESOURCES
AT THE SITE OF THE
PROPOSED REDIVERSION CANAL ROUTE
BERKELEY COUNTY, SOUTH CAROLINA
ARCHEOLOGICAL INVESTIGATION
BY
INSTITUTE OF ARCHEOLOGY AND ANTHROPOLOGY
UNIVERSITY OF SOUTH CAROLINA
MARCH/APRIL 1974
ADAPTED FROM
U.S. ARMY CORPS OF ENGINEERS
ST. STEPHEN HYDRO PLAN
GENERAL MAP
PLATE 8-JULY 1966
PROPOSED BRIDGES
PAVED ROADS
UNPAVED ROADS
ARCHEOLOGICAL SITE
POWER LINE
PROPOSED POWER HOUSE
SANTEE FLOOD PLAIN
2500 0 2500 5000
SCALE IN FEET
PINEVILLE
LAKE MOULTRIE
RUSSELLVILLE
ST. STEPHEN
SANTEE
RIVER
LAKE MATTASSEE
1000' WIDE CONSTRUCTION RIGHT-OF-WAY
SPOIL AREA
DIKE
S.C. 45
U.S. HWY 52
S.C.L. R.R.
SOUTH CAROLINA
SITE
SANTEE RIVER

FIGURE 2

United States Department of the Interior

NATIONAL PARK SERVICE
Office of Archeology and Historic Preservation
Interagency Archeological Services - Atlanta
730 Peachtree Street, Room 1010
Atlanta, Georgia 30308

IN REPLY REFER TO:

H2219-PI(A)

NOV 28 1975

Colonel Harry S. Wilson, Jr.
District Engineer
Charleston District, Corps of
Engineers
P.O. Box 919
Charleston, South Carolina 29402

Dear Col. Wilson:

Thank you for your recent letter requesting our evaluation of the proposal for archeological investigations in the areas to be affected by the Cooper River Rediversion Project. We have reviewed the proposal and offer the following comments for your consideration.

The potential contractor does not seem to be aware of the Corps' responsibilities under the National Historic Preservation Act of 1966 and Executive Order 11593 as delineated in the Advisory Council on Historic Preservation's Procedures for the Protection of Cultural Properties (36 CFR 800). These procedures require that the Corps identify and evaluate all potentially significant properties with respect to National Register criteria, and then proceed with the compliance process in consultation with the Advisory Council. An acceptable proposal should address itself not only to the problem of insuring an adequate treatment of the resources but also should include provisions for services which will satisfy the Corps' legal compliance requirements. We are enclosing a sample scope of work which we have been providing to other Corps districts as a guideline to the contractor services which should be specified. We feel that the approach outlined in the enclosure satisfies the Corps' needs and also serves the interests of the resources.

The amount of work which is proposed seems to be in excess of what the Corps needs at this time. The areas which have not been surveyed should be the primary focus of the survey effort. Once these surveys have been completed the data can be combined with the results of the previous survey and an evaluation of the total potential can then be made. Using

AMERICAN REVOLUTION BICENTENNIAL 1776-1976 this combined data base the contractor ought to be able to devise a comprehensive mitigation plan. The work currently proposed at both the Kellar site and the sites in the Power House area should more properly be considered part of the mitigation effort, should salvage be decided upon as the most satisfactory alternative after consultation with the Advisory Council and other parties involved.

From the technical point of view we find the proposal too general in its approach and are unable to fairly assess it for technical sufficiency. Details of methodology, approach, data catagories, etc. are lacking. We realize that these are matters that would be covered in the final research design for the mitigation phase, however, this proposal was originally submitted to us as a plan for mitigation and has been resubmitted to the Corps virtually unchanged. As such, these criticisms are valid. A major example of the proposal's shortcomings is the fact that a plan which purportedly is designed to evaluate coastal plain adaptations in the last 10-12,000 years does not mention a single paleoecological reference.

Finally, under FACILITIES (page 27) there is a reference to National Park Service support for operating costs as well as the cost of moving and setting up trailers in the research area. We have not been approached on this matter and this may be an oversight in the resubmission of the proposal.

We would like to emphasize that we will gladly assist the Corps with archeological expertise in all phases of the compliance process. We appreciate this opportunity to comment and we thank you for your efforts on behalf of the cultural resources in the Charleston District. If you have any questions please do not hesitate to call at 404-526-2611. After December 1st our new FTS number will be 285-2611.

Sincerely yours,

Wilfred M. Husted
Acting Chief, Interagency
Archeological Services-Atlanta

Enclosure:sample scope of work

SACEN-E 30 March 1976

Dr. Leland G. Ferguson
Archaeologist
Institute of Archaeology and Anthropology
University of South Carolina
Columbia, South Carolina 29208

Dear Dr. Ferguson:

We have reviewed your proposal of 17 October 1975 for archaeological survey and testing of cultural resources within the right-of-way of the proposed Cooper River Rediversion Canal. Your proposal was also evaluated by the National Park Service Office of Archaeology and Historic Preservation whose comments are inclosed. Our review and that of the National Park Service indicate two areas of major concern.

1. The proposed work does not satisfy the requirements of the Corps of Engineers as outlined in the inclosed 33 FR 41636-41641 Cultural Resources, Identification and Administration. At this time, the Corps is required to conduct a cultural resources survey which is defined in the above-referenced Federal Register as being sufficient to permit determination of the number and extent of the resources present, their scientific importance, and the time factors and cost of preserving, recovering or otherwise mitigating adverse effects on them. The criteria to be applied in determining the importance of a site are those for determining eligibility in the National Register for Historic Places. At this time, the Corps must know which, if any, of the sites in the project area qualify for the National Register in order to meet its legal responsibilities for further consultation with the National Park Service and the Advisory Council on Historic Preservation. The 17 October proposal does not specify an evaluation of each site by the National Register criteria.

2. The work outlined in the proposal goes beyond that needed by the Corps at the present. Any detailed excavations and other major recovery efforts would follow the evaluation of sites for inclusion in the National Register. While cost estimates and time estimates for mitigation are requested as part of the cultural resources survey report, actual steps

toward mitigation could be funded only after study of the report by the Corps and consultation with the necessary agencies. The 17 October proposal is little changed from the proposal of 19 June 1975 submitted to the National Park Service. Considerable effort is directed toward gathering data of a broad and general nature.

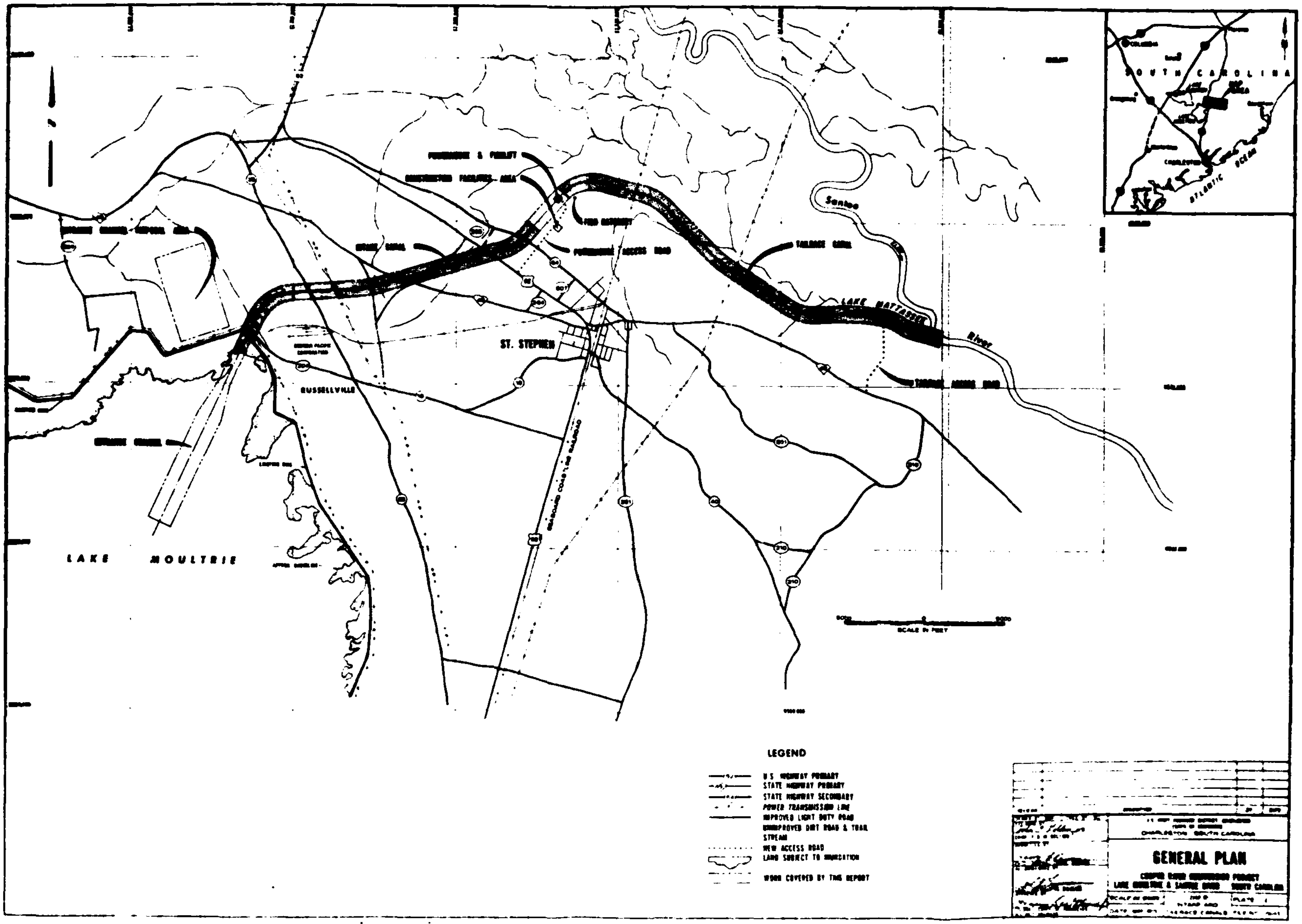
Based on the content of the 17 October proposal, much of the work needed to satisfy the requirements of a cultural resources survey has already been performed. Further survey efforts should be concentrated on areas which have not yet been examined. Inclosed for your consideration is a scope of work which we believe provides for the required archaeological surveys of unexamined areas.

Please submit a cost estimate for the work described in the inclosed scope of work. The inclosed scope of work will form the basis for any contractural agreement between the contractor and the Corps of Engineers.

Sincerely

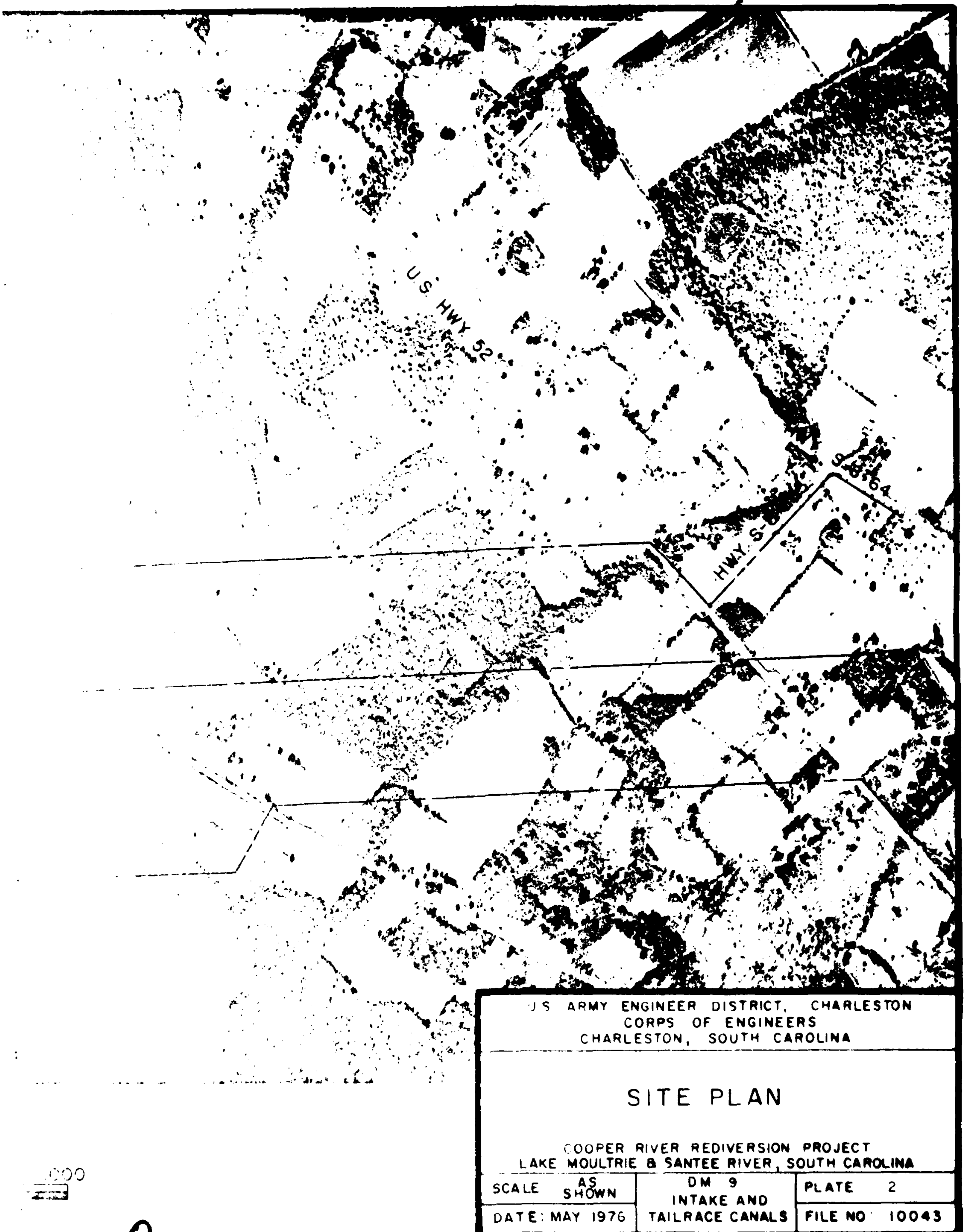
3 Incls
As stated

HARRY S. WILSON, JR.
Colonel, Corps of Engineers
District Engineer


ST. STEPHEN
RUSSELLVILLE
LAKE MOULTRIE
LAKE MARION
Santee
River
SOUTH CAROLINA
ATLANTIC OCEAN
SCALE IN FEET
LEGEND
U.S. HIGHWAY PRIMARY
STATE HIGHWAY PRIMARY
STATE HIGHWAY SECONDARY
POWER TRANSMISSION LINE
IMPROVED LIGHT DUTY ROAD
UNIMPROVED DIRT ROAD & TRAIL
STREAM
NEW ACCESS ROAD
LAND SUBJECT TO INUNDATION
WORK COVERED BY THIS REPORT
CHARLESTON, SOUTH CAROLINA
GENERAL PLAN
SOUTH CAROLINA


HWY S-8-35
NAL CONSTRUCTION
34
ANNEL
LAKE
MOULTRIE
HWY S-8-204

2



U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

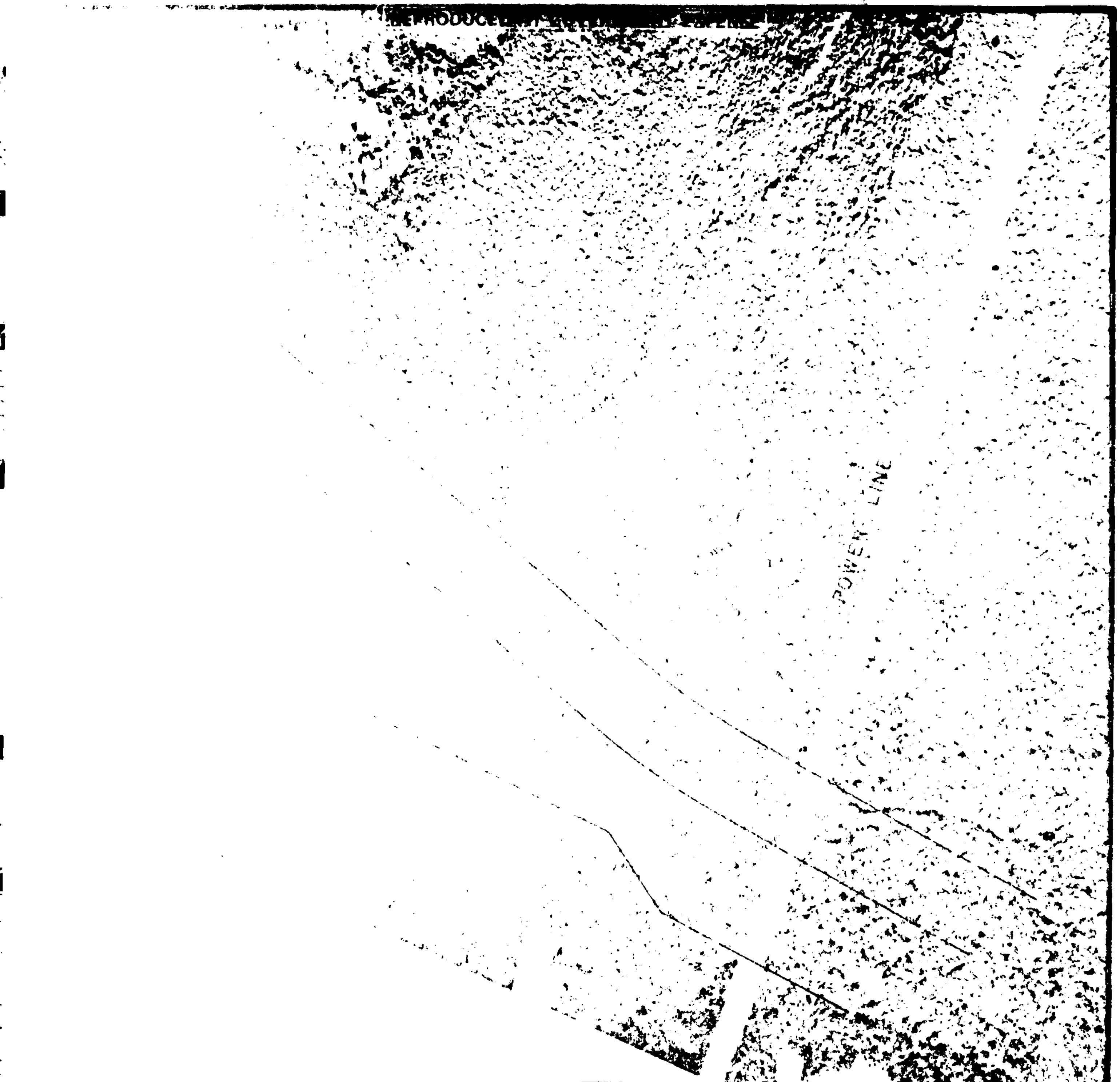
SITE PLAN

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE 2
DATE: MAY 1976		FILE NO 10043


POWERHOUSE
FISH HATCHERY
CONSTRUCTION FACILIT
ACCESS RD
HWY 52
HWY S.C. 45
N

2



U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

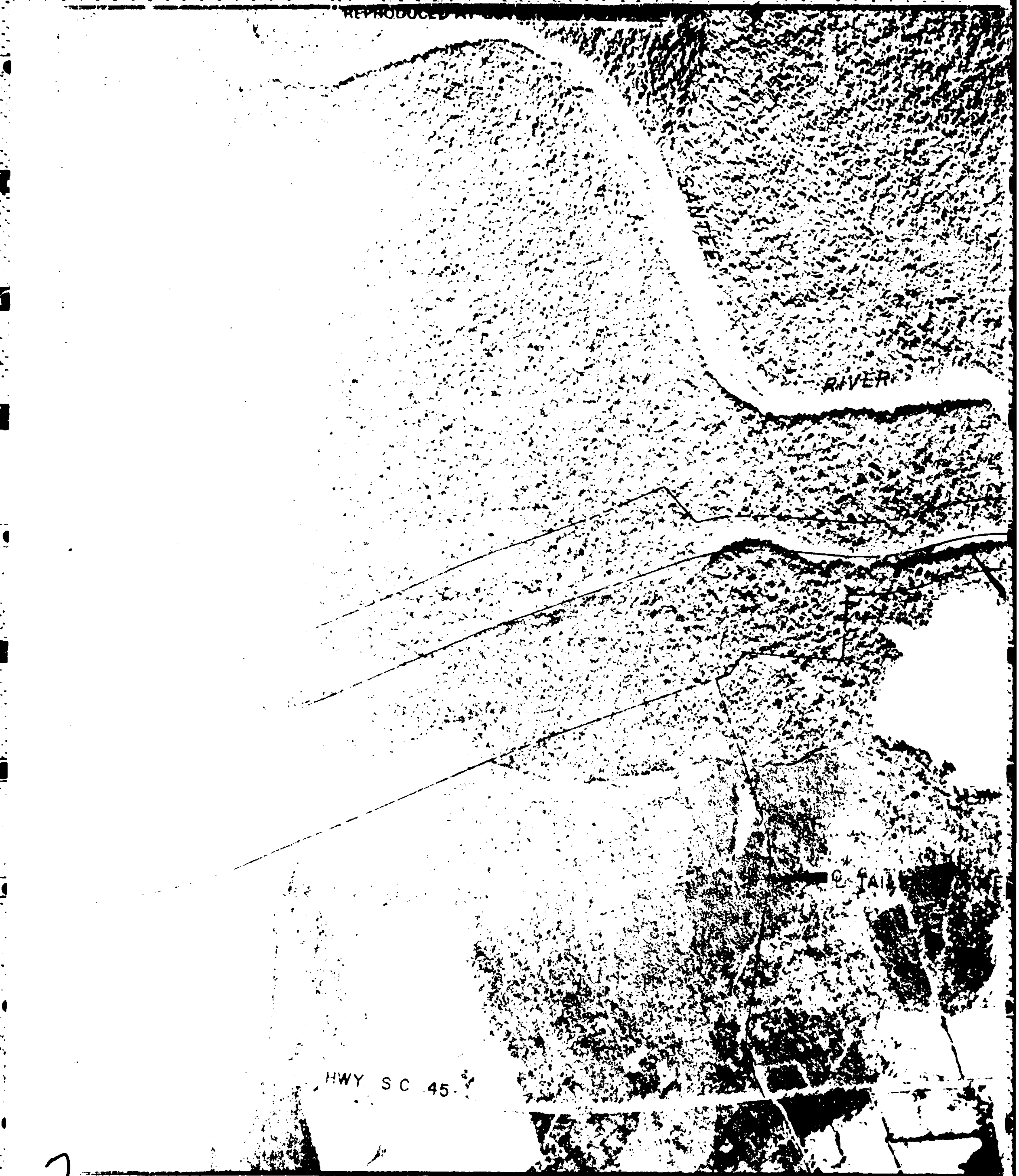
SITE PLAN

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE 3
DATE: MAY 1976		FILE NO. 10043

3


℄ CANAL
1000 0 1000
SCALE IN FEET



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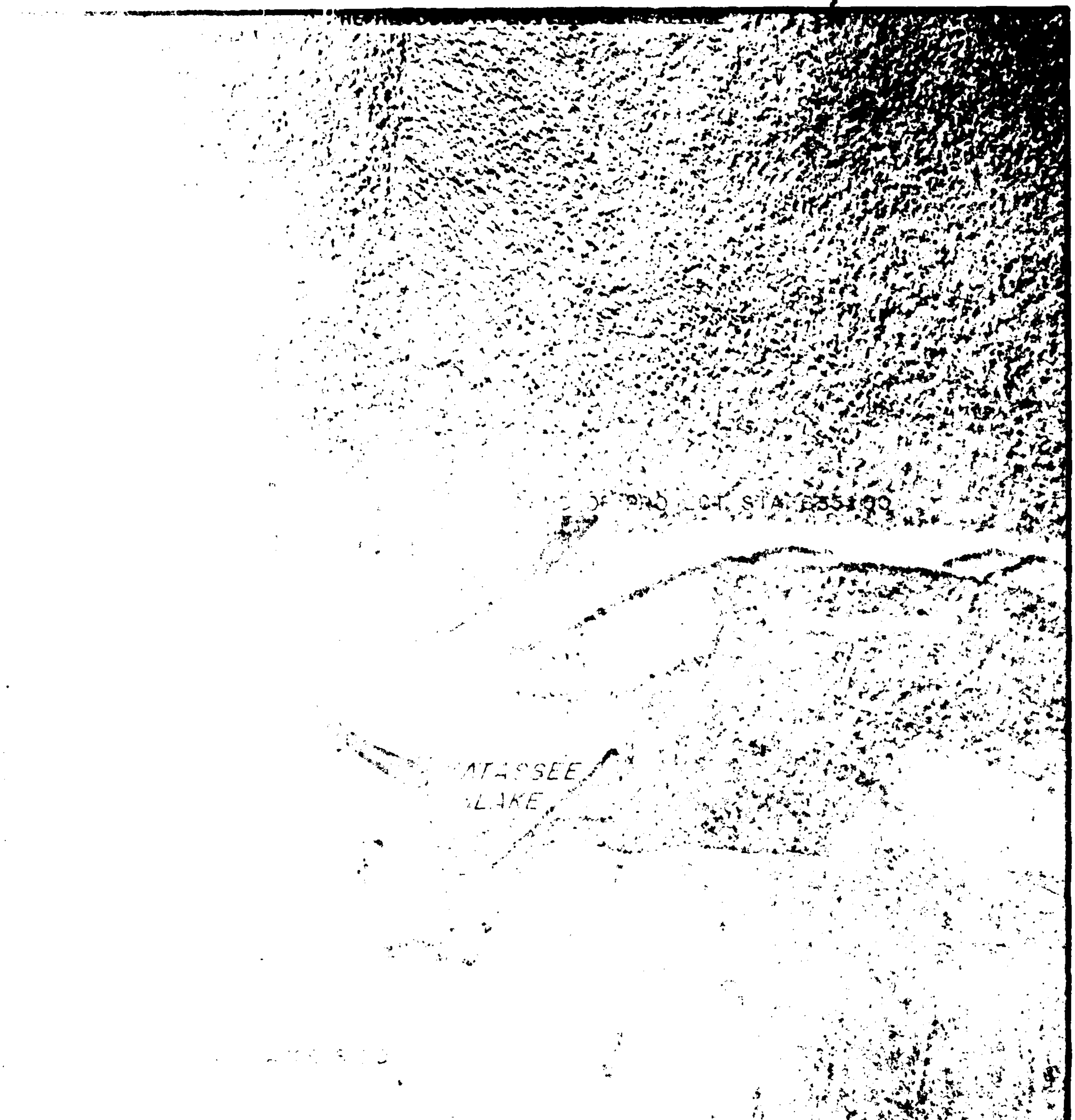


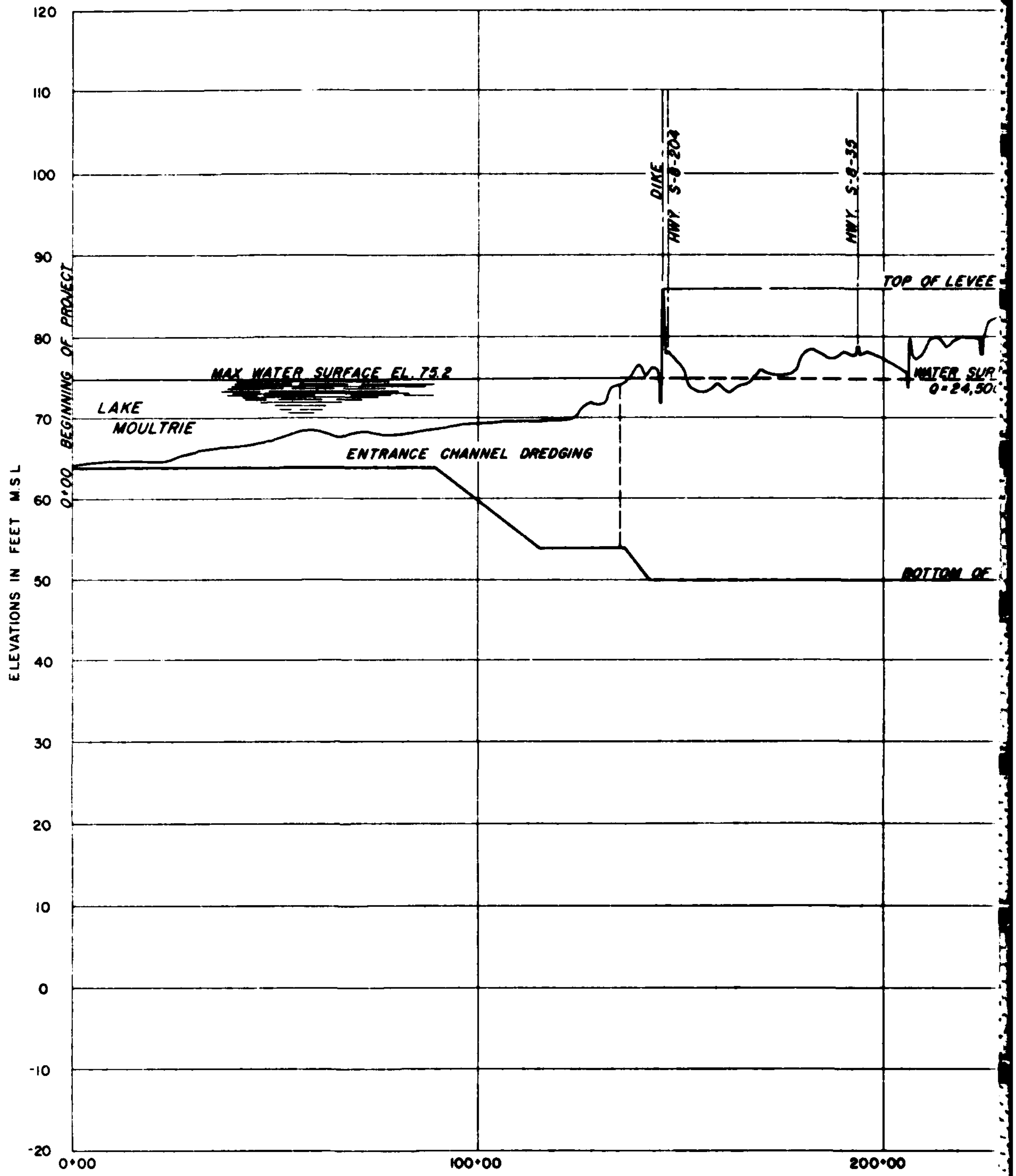
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

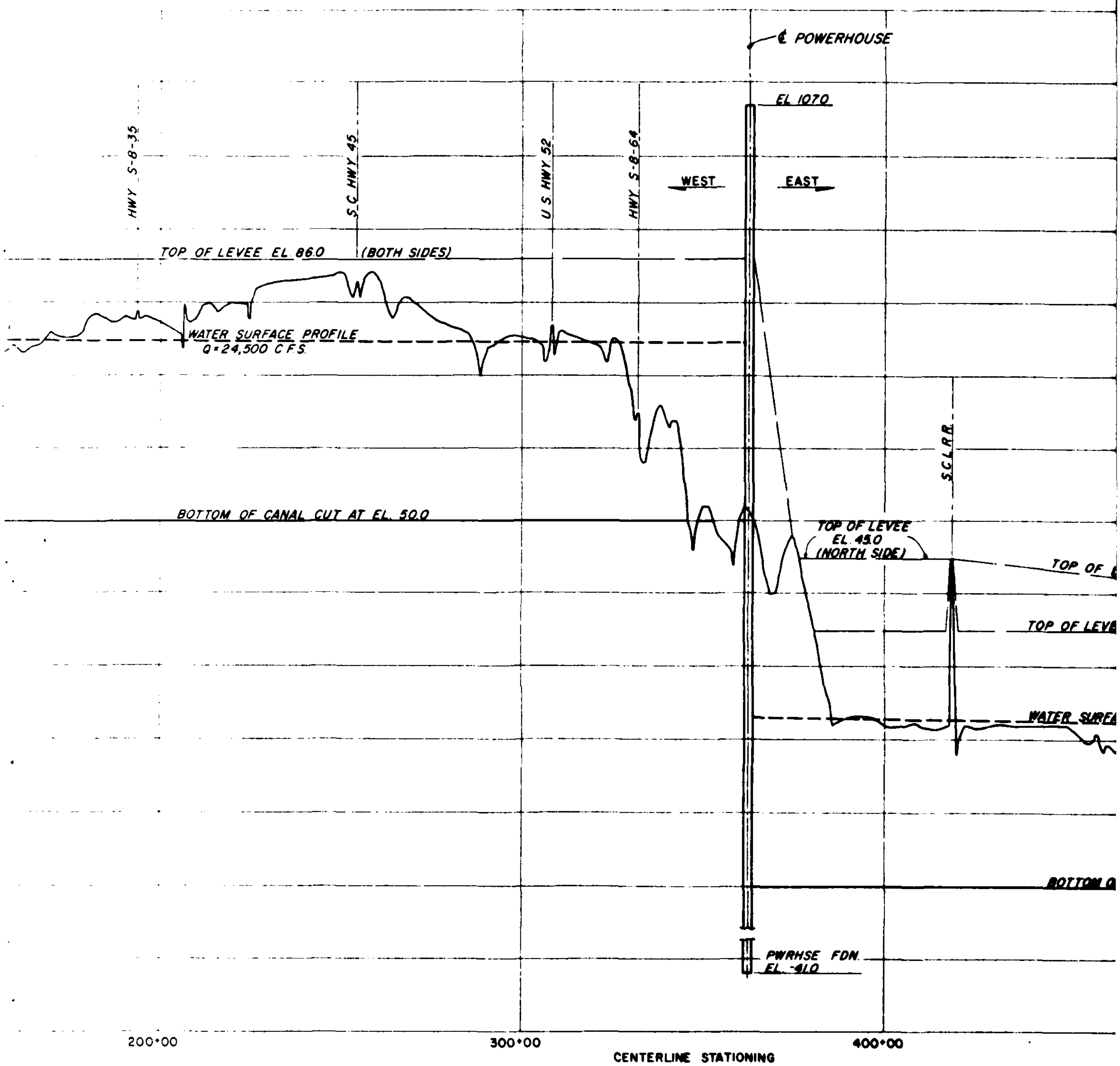
SITE PLAN

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE 4
DATE MAY 1976		FILE NO 10043

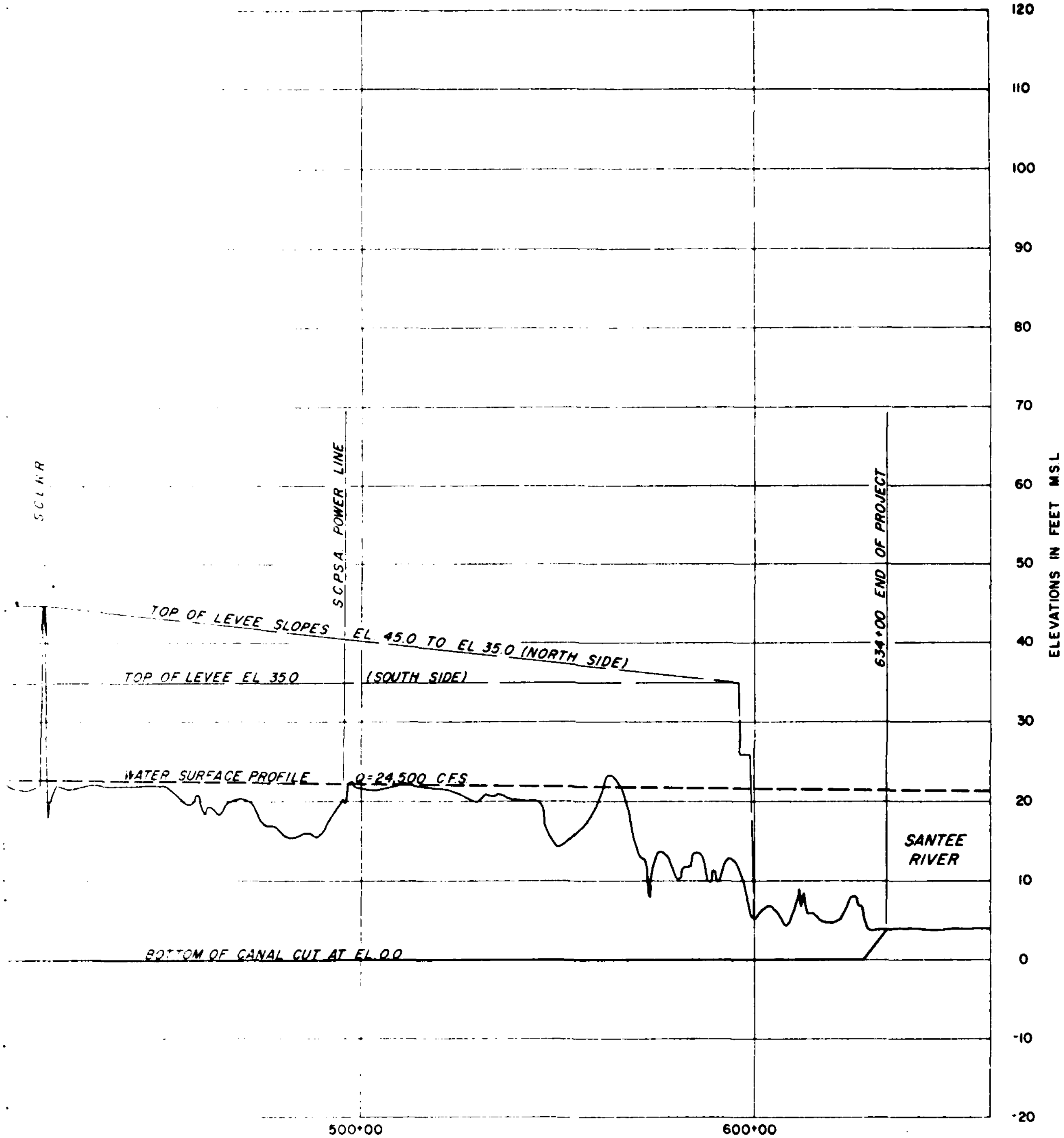
3


120
110
100
90
80
70
60
50
40
30
20
10
0
-10
-20
ELEVATIONS IN FEET M.S.L
0+00 BEGINNING OF PROJECT
LAKE
MOULTRIE
MAX WATER SURFACE EL. 75.2
ENTRANCE CHANNEL DREDGING
DIKE
HWY S-8-204
HWY. S-8-35
TOP OF LEVEE
WATER SUR
Q=24,50
BOTTOM OF
0+00
100+00
200+00



℄ POWERHOUSE
EL 107.0
HWY S-8-35
S.C. HWY 45
U.S. HWY 52
HWY S-8-64
WEST
EAST
TOP OF LEVEE EL 86.0
(BOTH SIDES)
WATER SURFACE PROFILE
Q=24,500 C.F.S.
S.C.L.R.R.
BOTTOM OF CANAL CUT AT EL. 50.0
TOP OF LEVEE
EL. 45.0
(NORTH SIDE)
TOP OF
TOP OF LEVE
WATER SURF
BOTTOM O
PWRHSE FDN.
EL. -41.0
200+00
300+00
400+00
CENTERLINE STATIONING

2



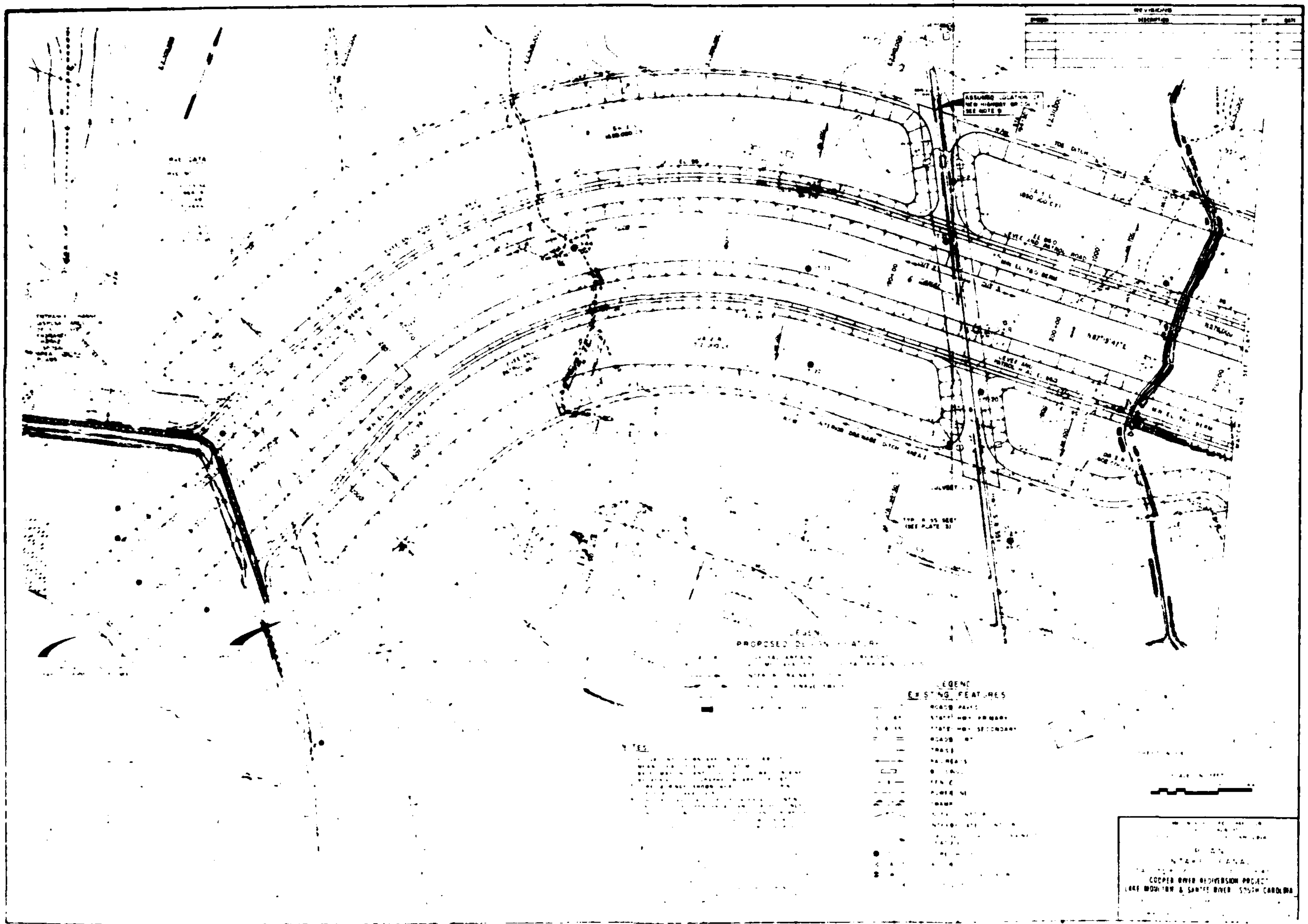
U S ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

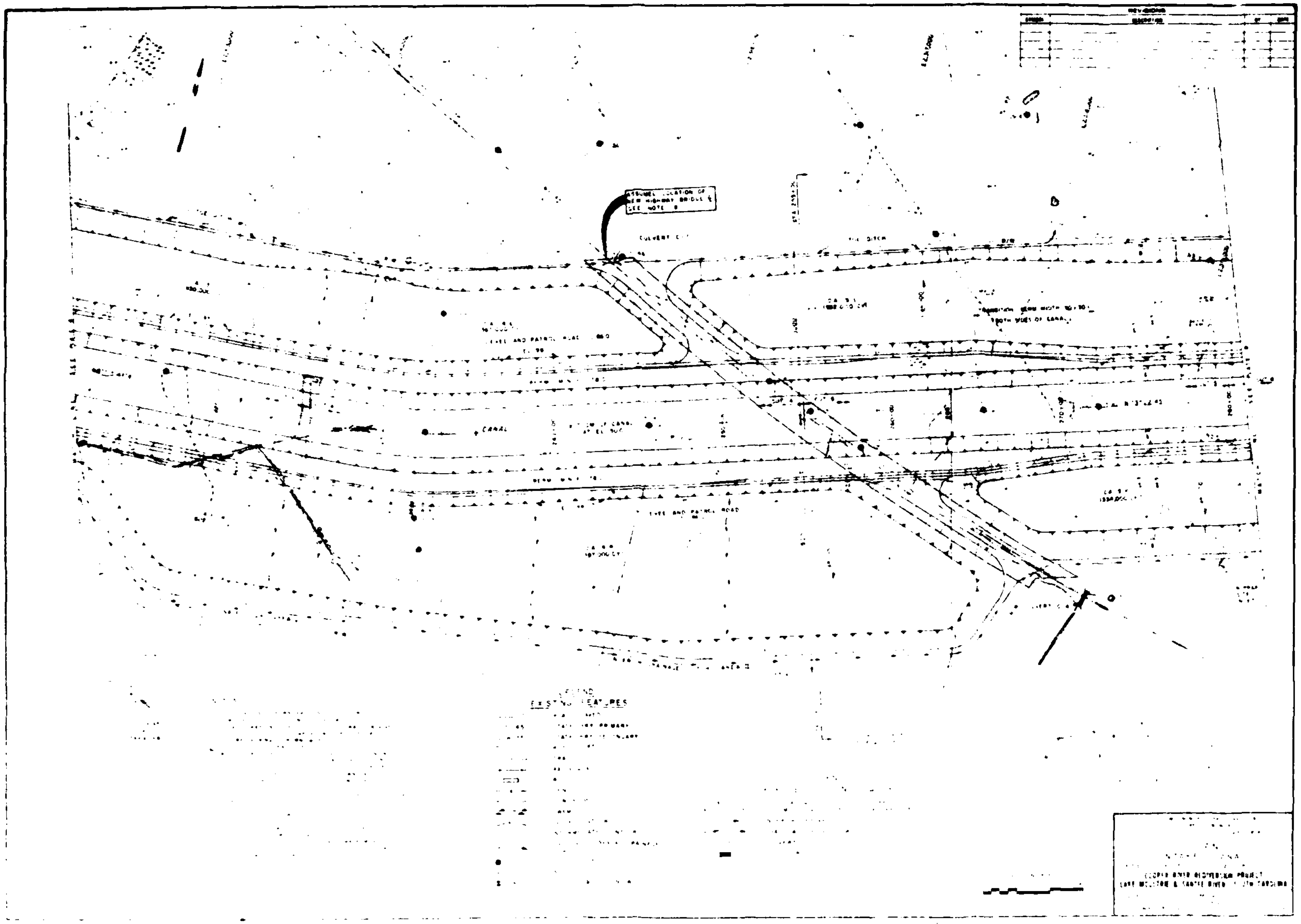
CENTERLINE PROFILE
INTAKE AND TAILRACE CANALS

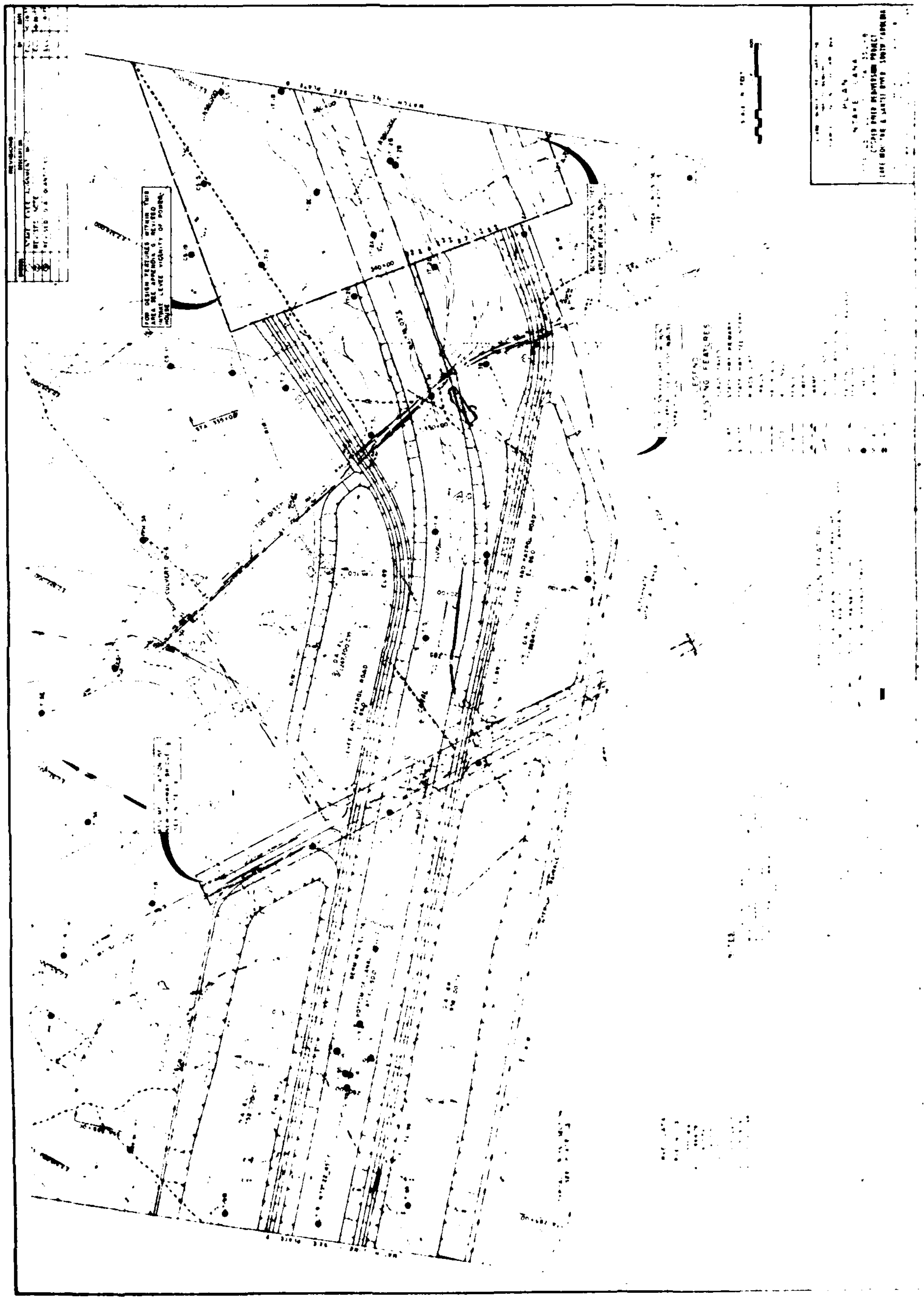
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

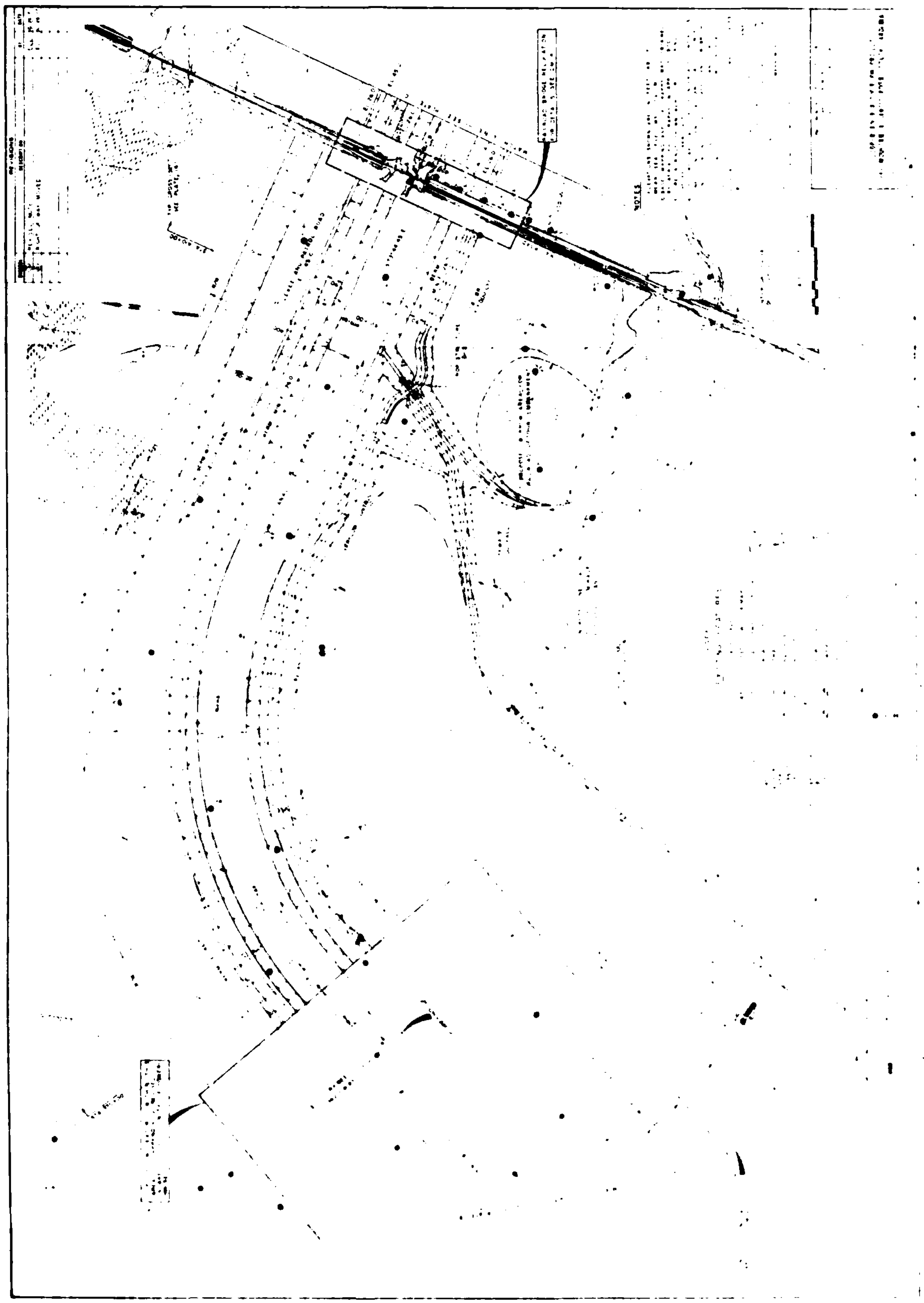
SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE 5
DATE MAY, 1976		FILE NO.: 10043

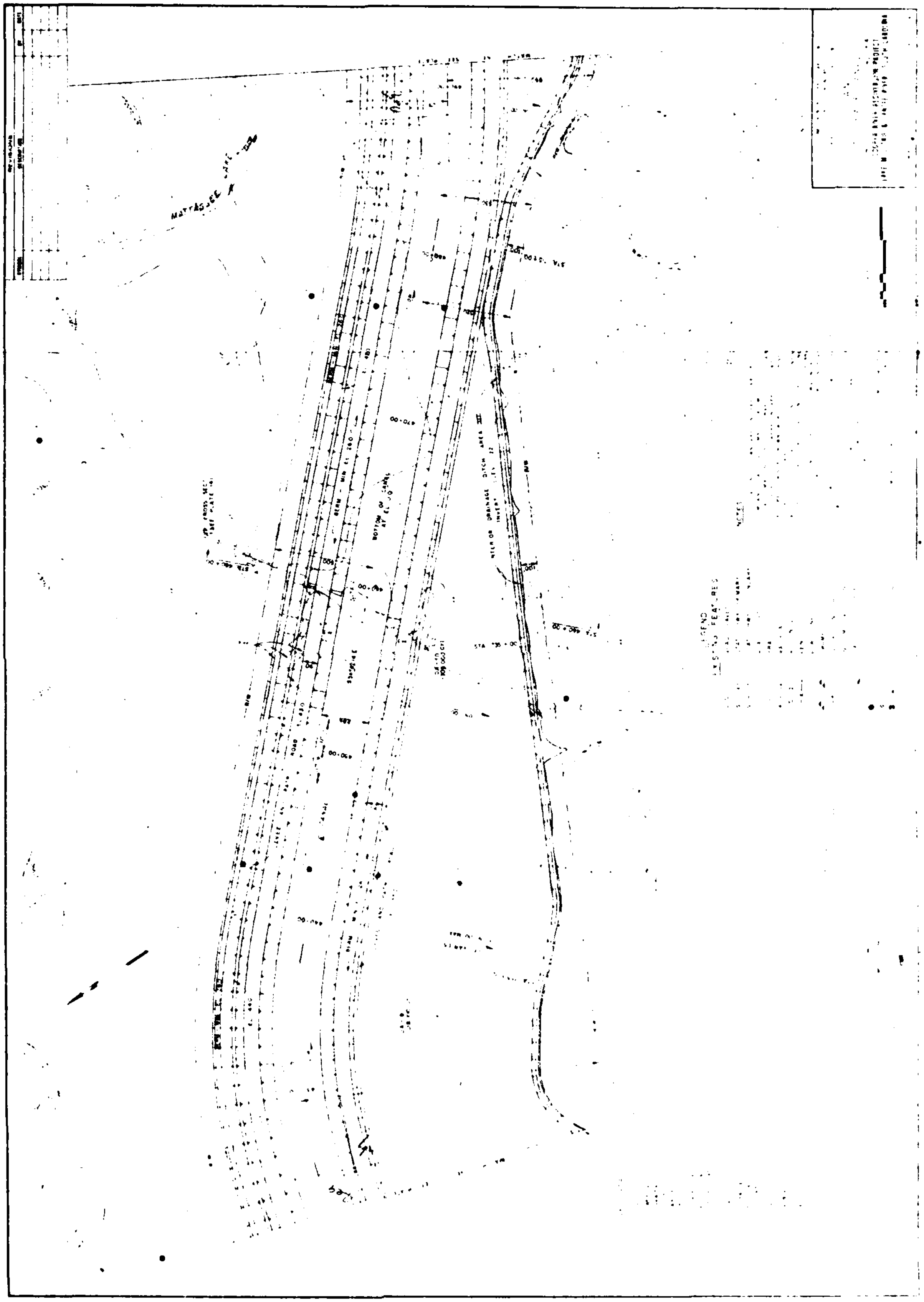
3

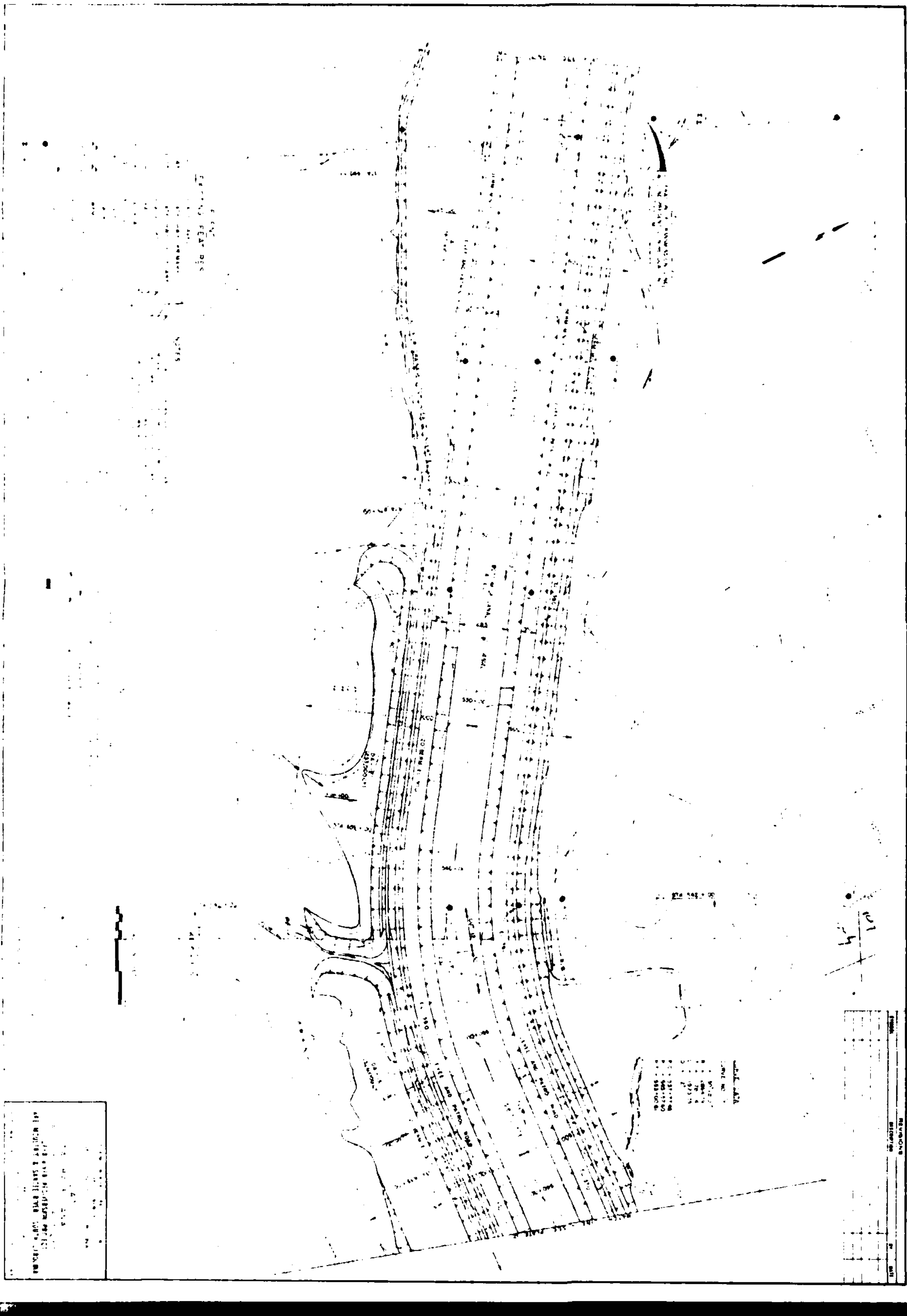

REVISIONS
LEGEND
EXISTING FEATURES
NOTES
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

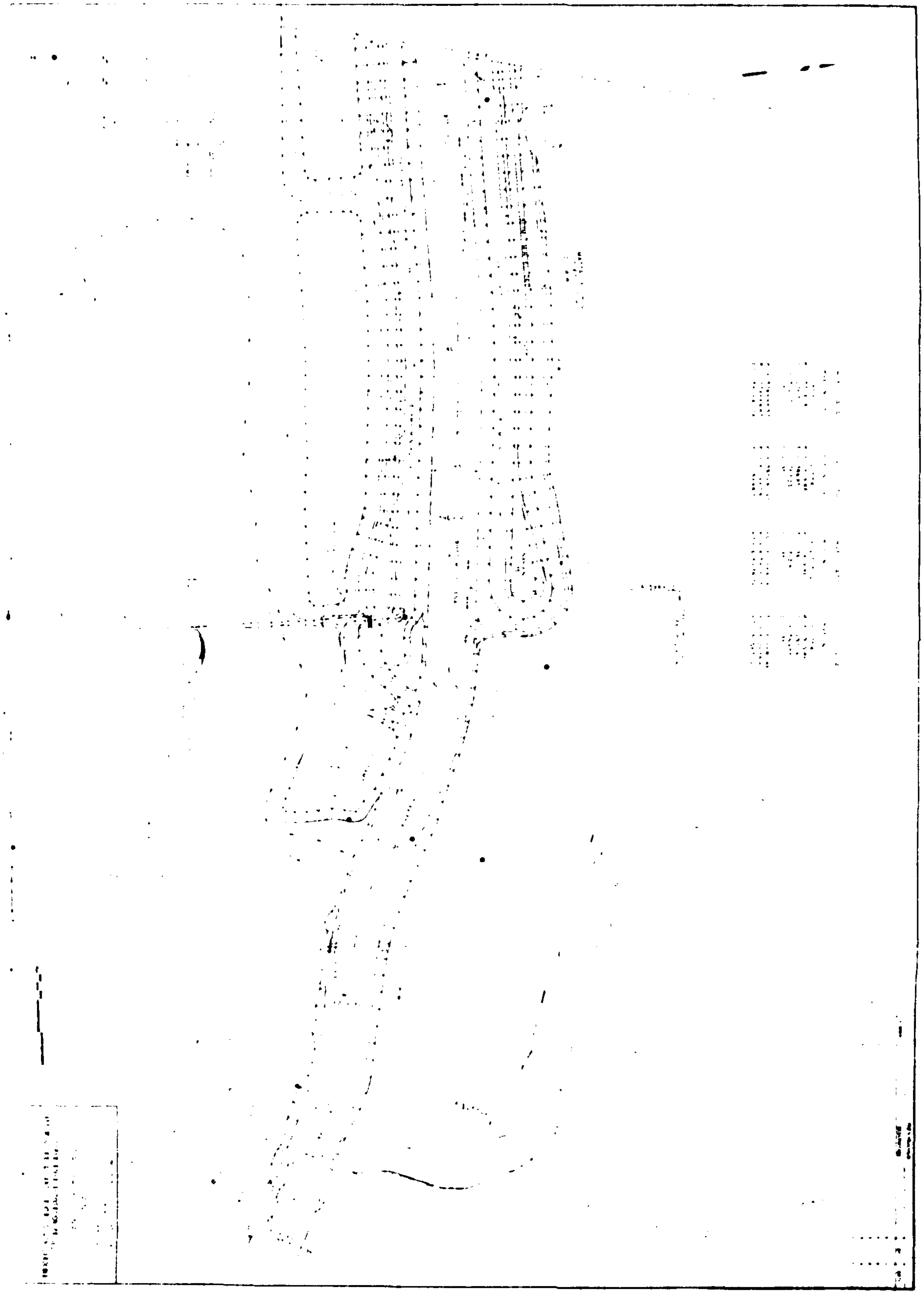

REVISIONS
ASSUMED LOCATION OF NEW HIGHWAY BRIDGE SEE NOTE 9
CULVERT
CANAL
LEVEE AND PATROL ROAD
LEGEND
EXISTING FEATURES
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

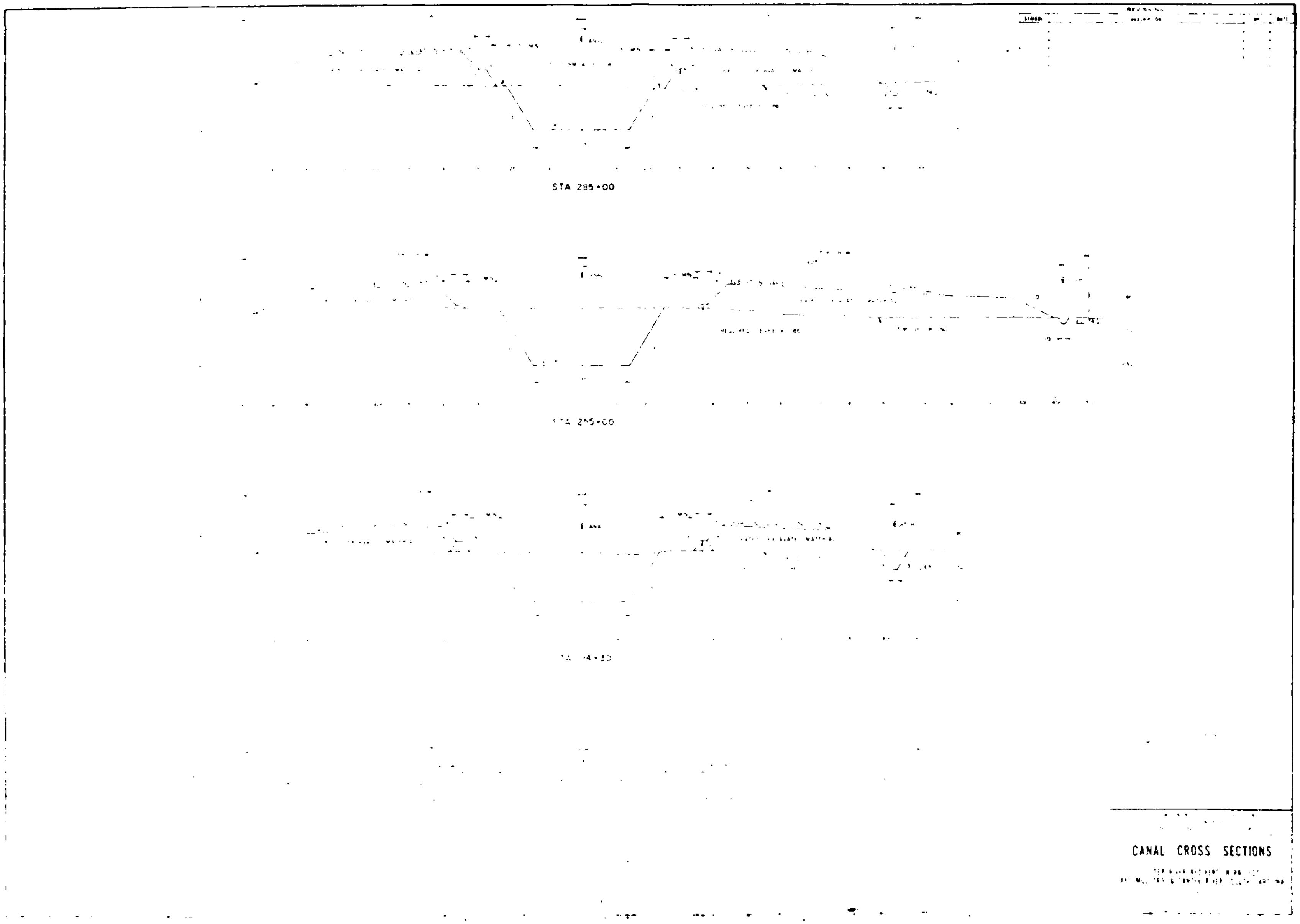


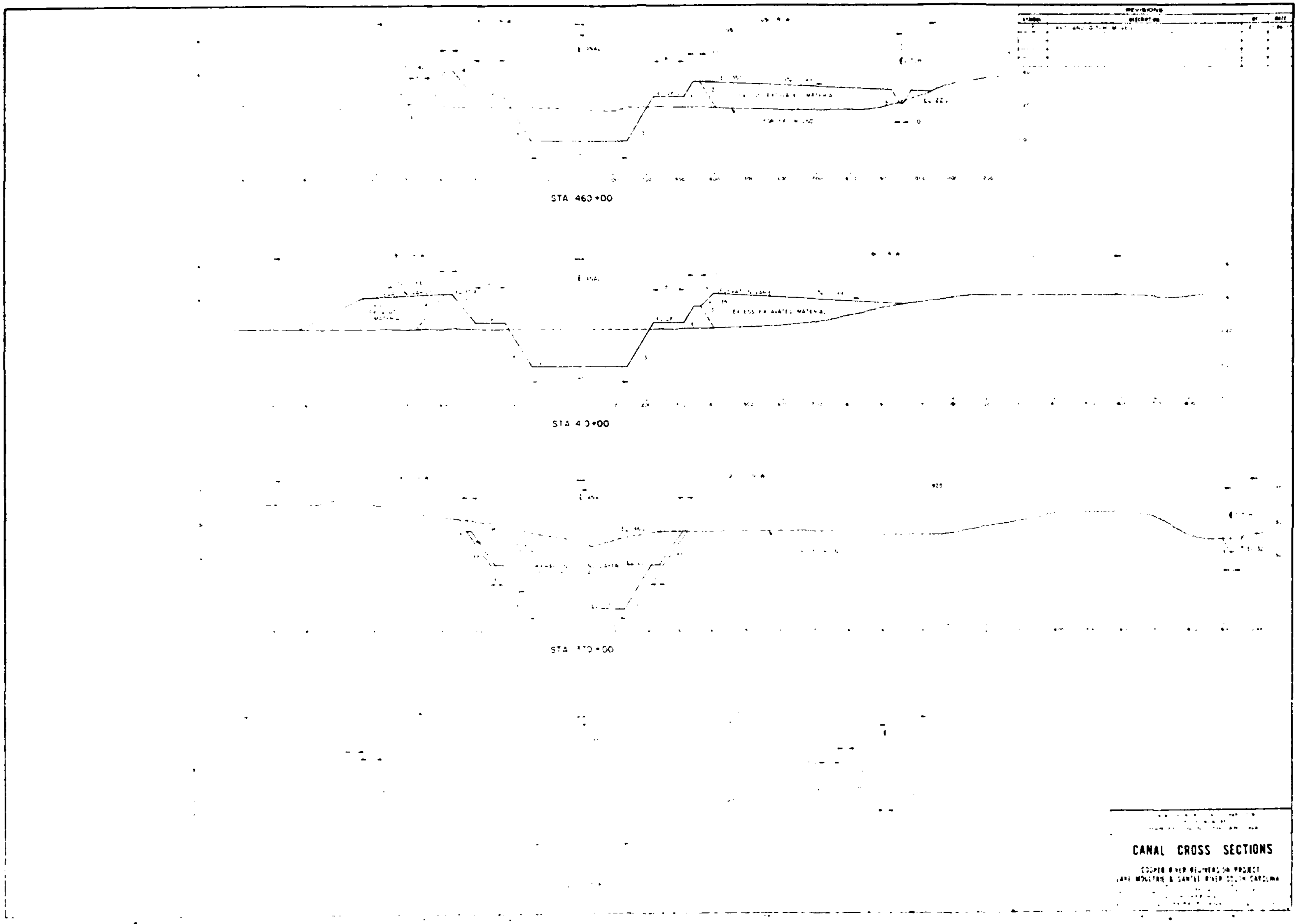


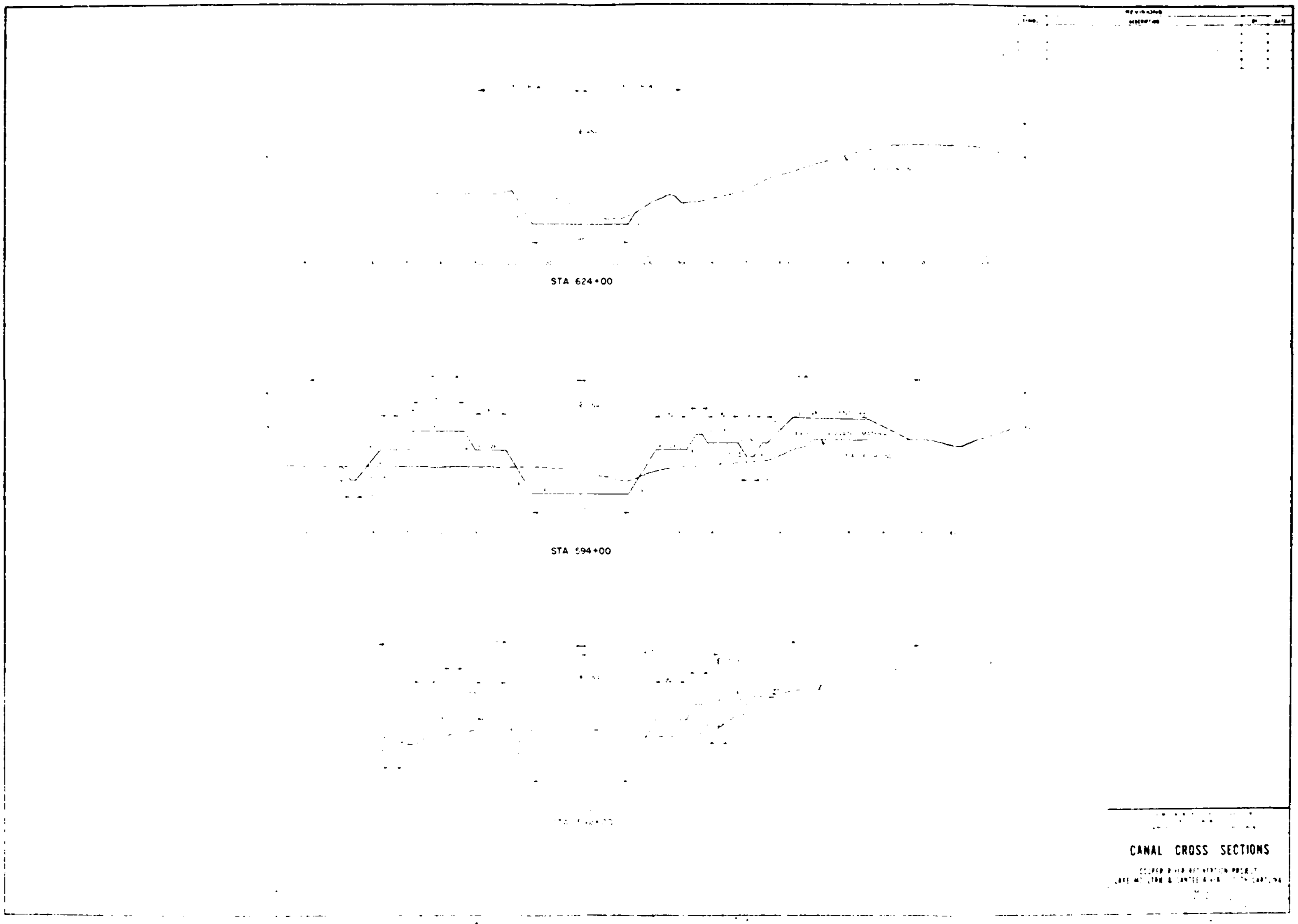







STA 285+00
CANAL CROSS SECTIONS


REVISIONS
STA 460+00
EXCESS EXCAVATED MATERIAL
STA 470+00
CANAL CROSS SECTIONS
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA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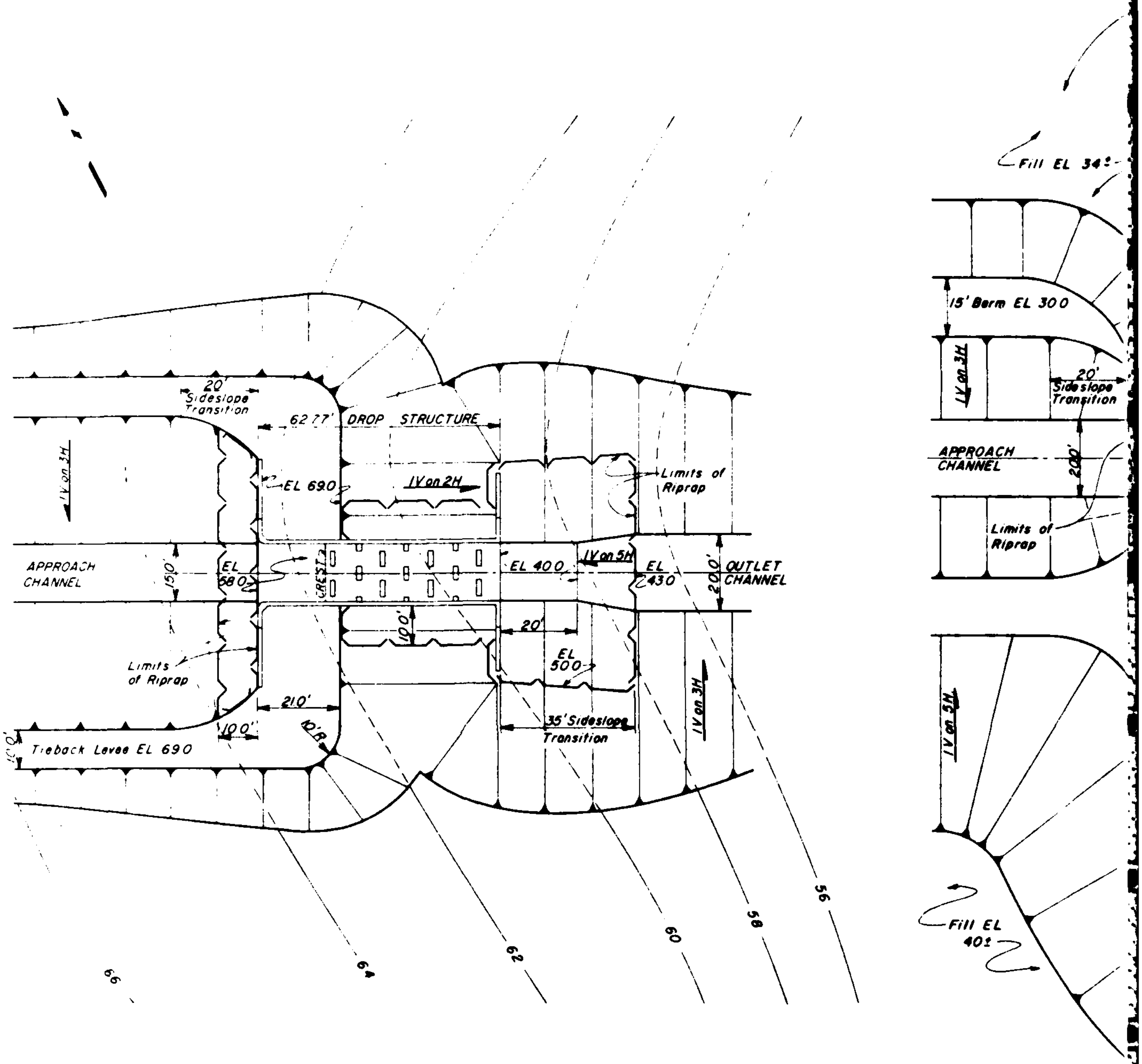
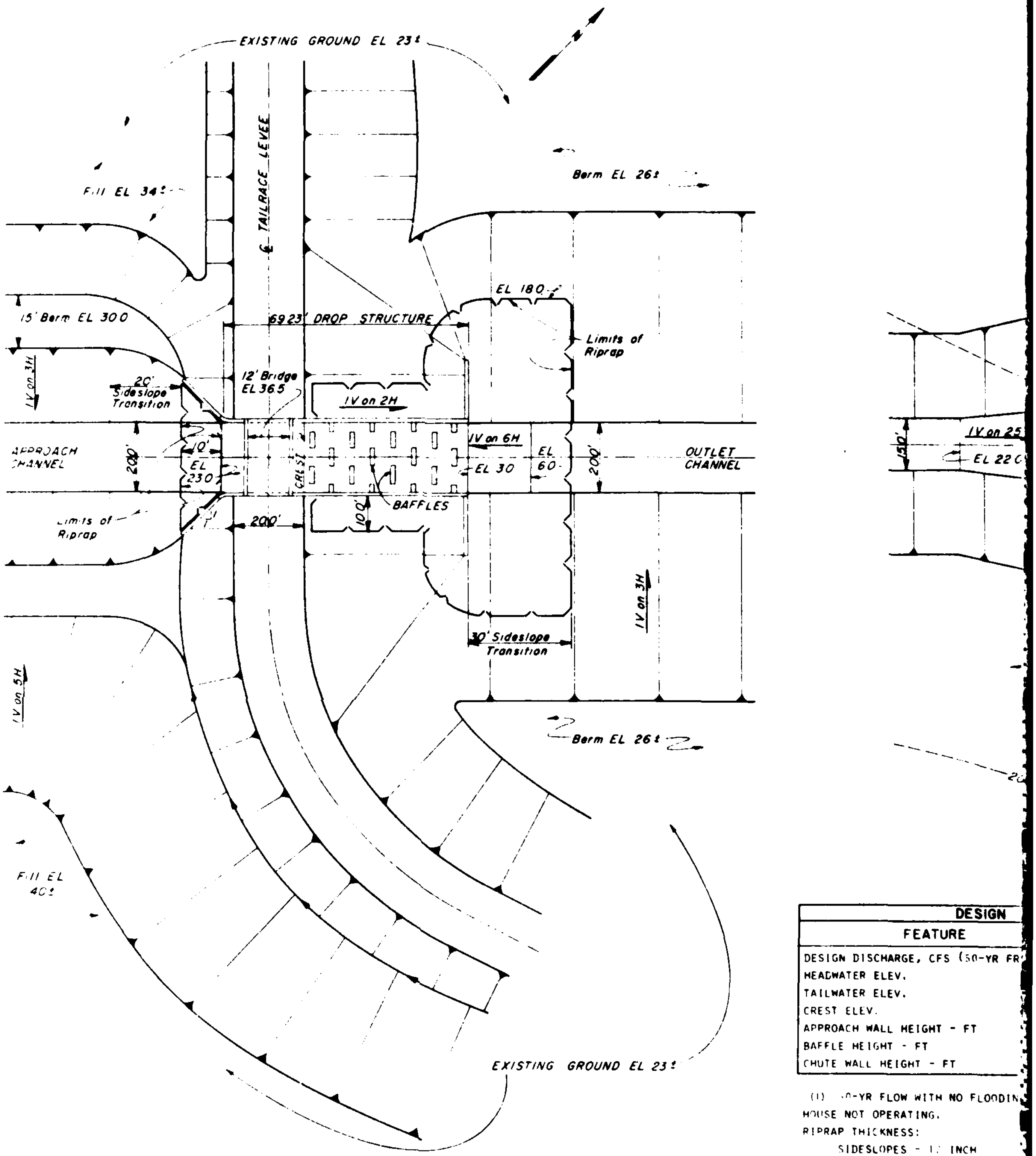
STA 594+00
CANAL CROSS SECTIONS



SITE PLAN

STRUCTURE D-1

SCALE 1" = 20'

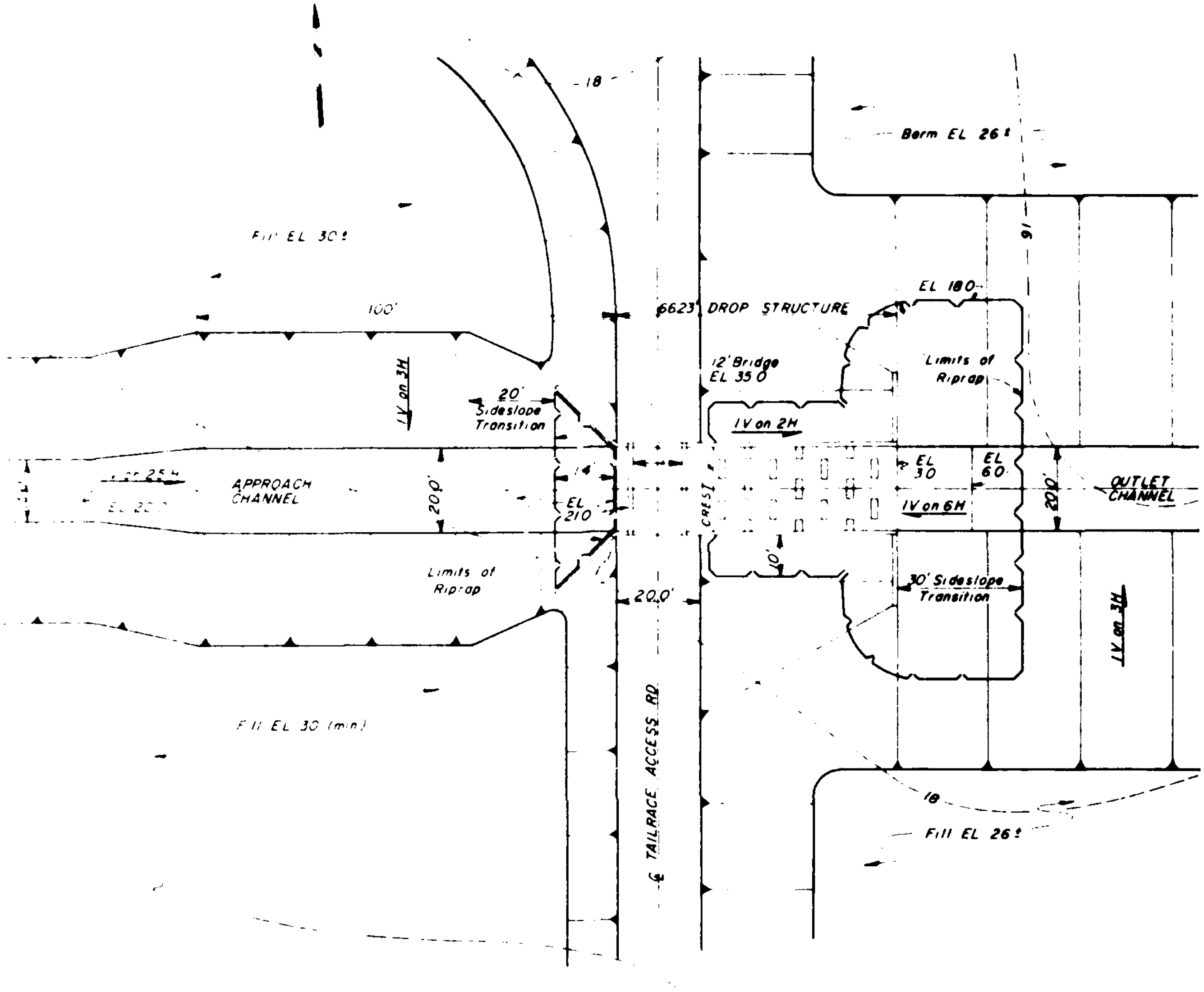


DESIGN
FEATURE
DESIGN DISCHARGE, CFS (50-YR FR
HEADWATER ELEV.
TAILWATER ELEV.
CREST ELEV.
APPROACH WALL HEIGHT - FT
BAFFLE HEIGHT - FT
CHUTE WALL HEIGHT - FT

(1) 50-YR FLOW WITH NO FLOODIN
HOUSE NOT OPERATING.
RIPRAP THICKNESS:
SIDESLOPES - 12 INCH
CHANNEL BOTTOM - 18 INCH

SITE PLAN
STRUCTURE D-2
SCALE 1" = 20'

2



DESIGN DATA

FEATURE	D-1	D-2	D-3
[illegible]	[illegible]	[illegible]	[illegible]

SITE PLAN
STRUCTURE D-3
SCALE 1" = 20'

SCALE IN FEET
20 0 20 40

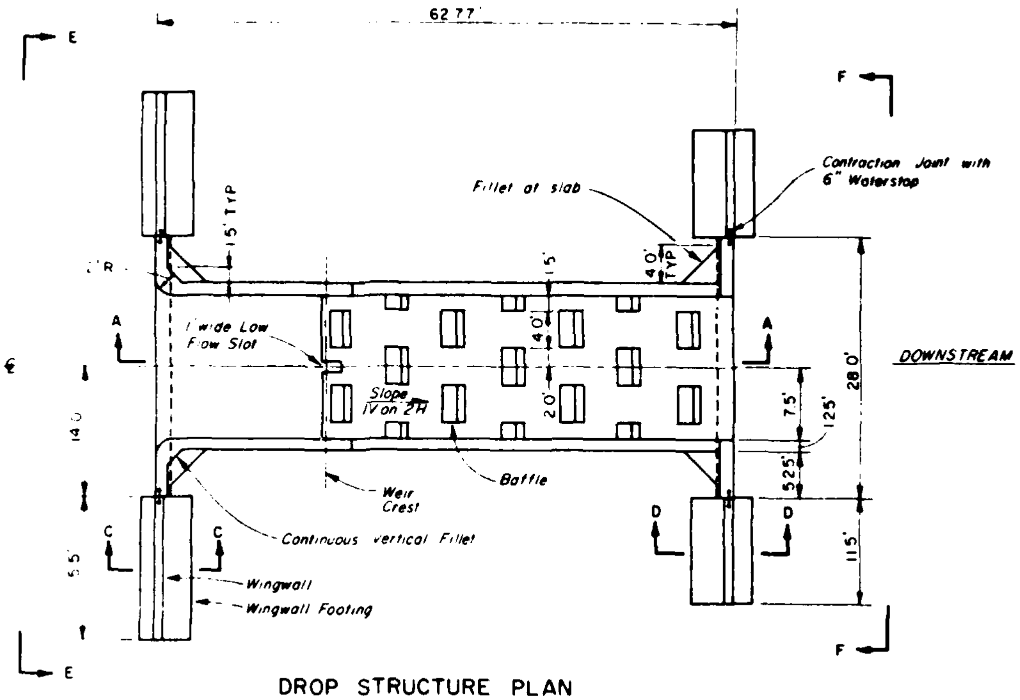
3

U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

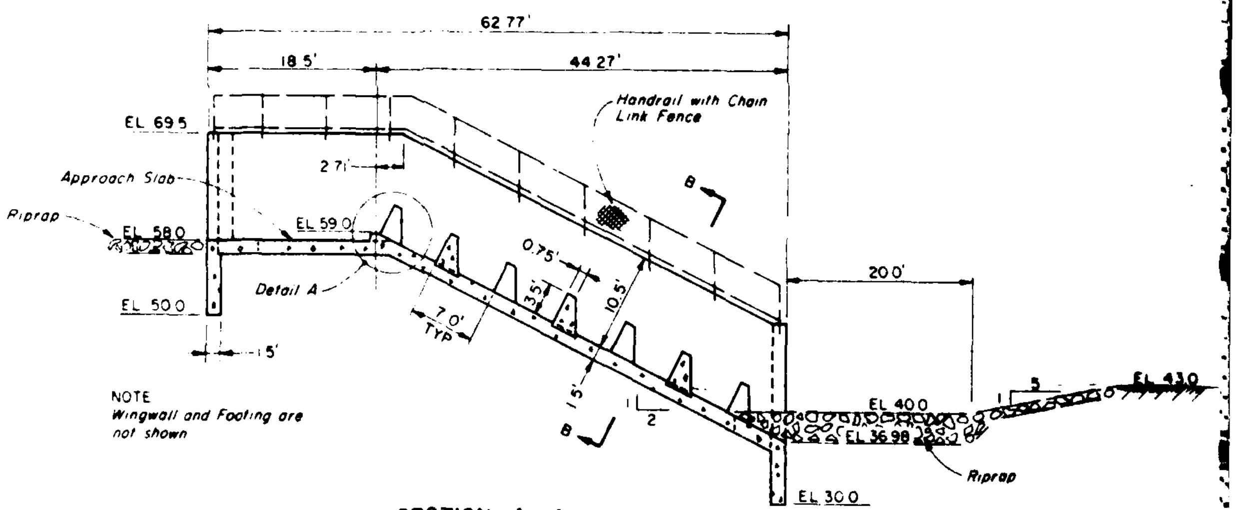
INTERIOR DRAINAGE
BAFFLED CHUTE DROP STRUCTURES
SITE PLANS

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

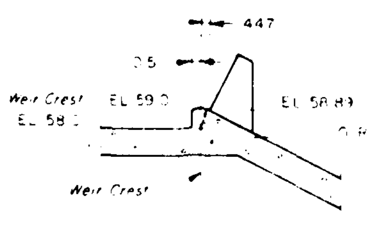
SCALE AS SHOWN	DM 9	PLATE 16
DATE MAY 1976	INTAKE AND TAILRACE CANALS	FILE NO 10043



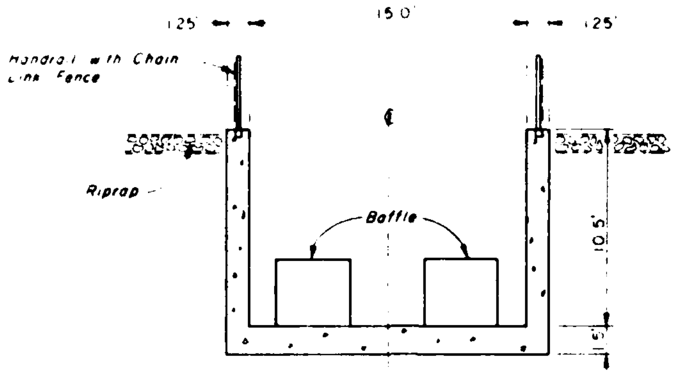
DROP STRUCTURE PLAN
SCALE 1" = 10'



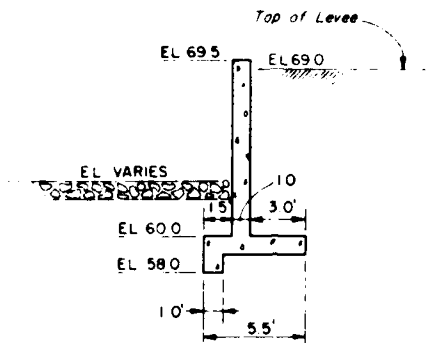
SECTION A - A
SCALE 1" = 10'



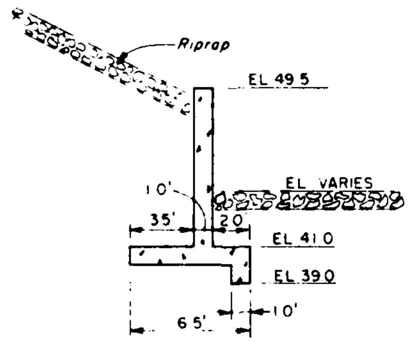
DETAIL A SCALE



SECTION B - B
SCALE 1" = 5'

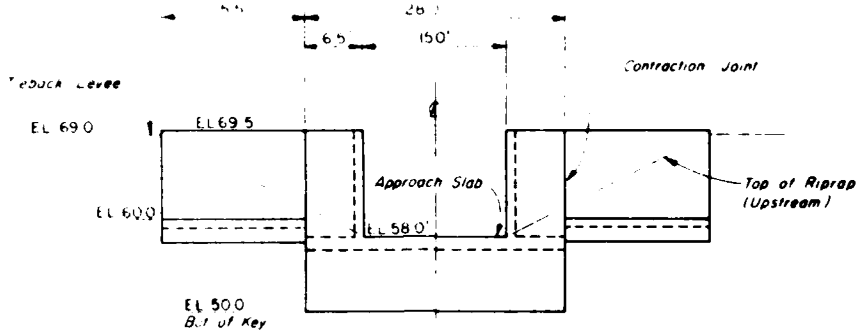


UPSTREAM WINGWALL
SECTION C - C
SCALE 1" = 5'



DOWNSTREAM WINGWALL
SECTION D - D
SCALE 1" = 5'

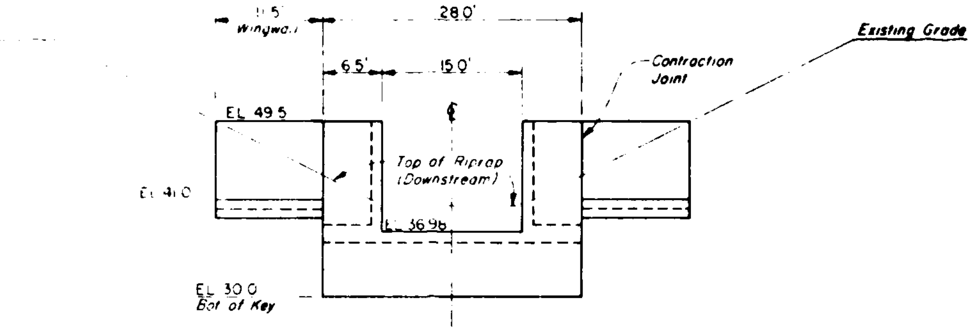
2



UPSTREAM ELEVATION

E - E

SCALE 1" = 10'

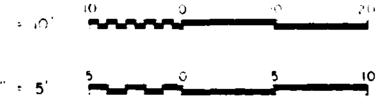


DOWNSTREAM ELEVATION

F - F

SCALE 1" = 10'

SCALES IN FEET



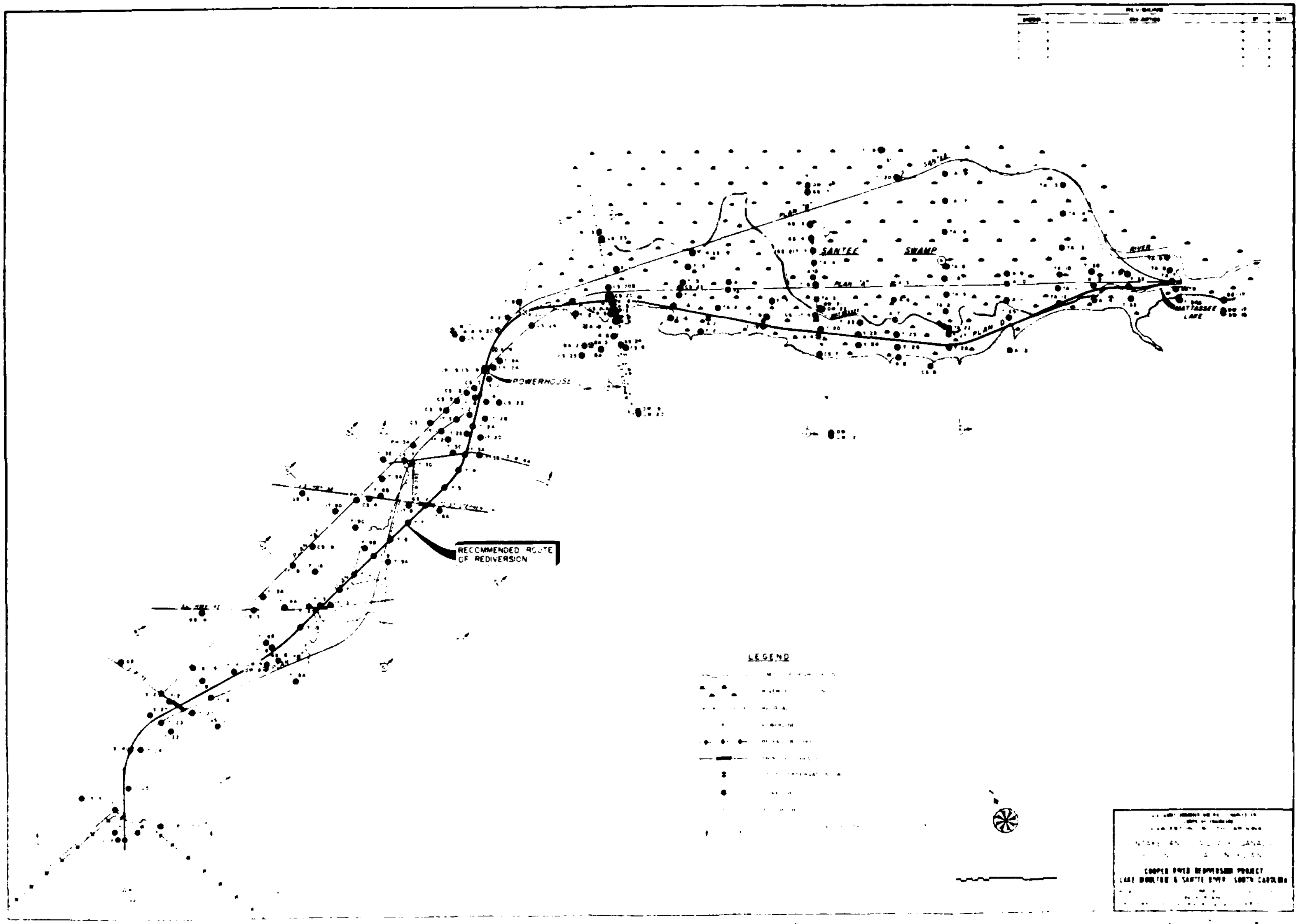
U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

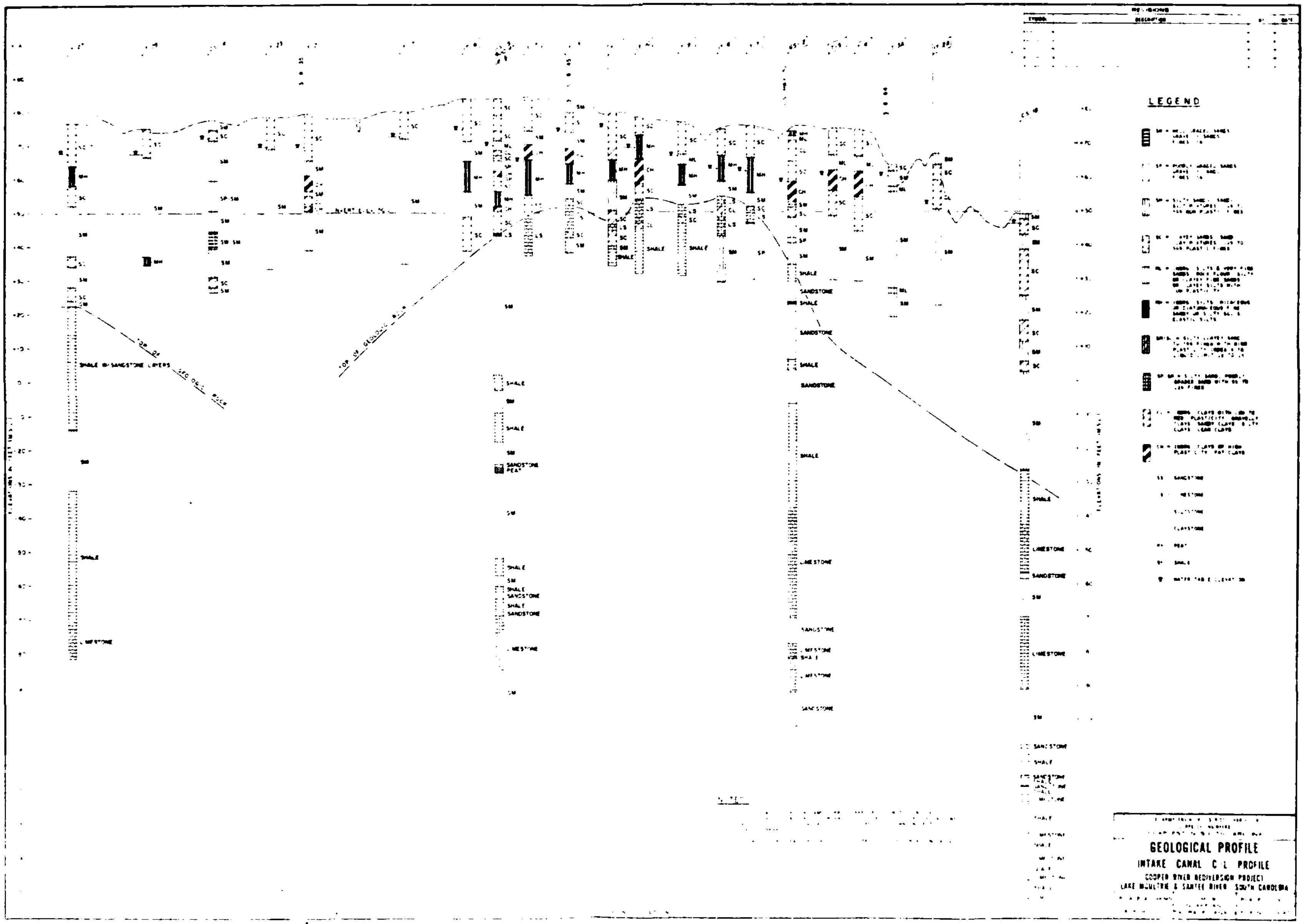
INTERIOR DRAINAGE
BAFFLED CHUTE DROP STRUCTURE
STRUCTURE D-1

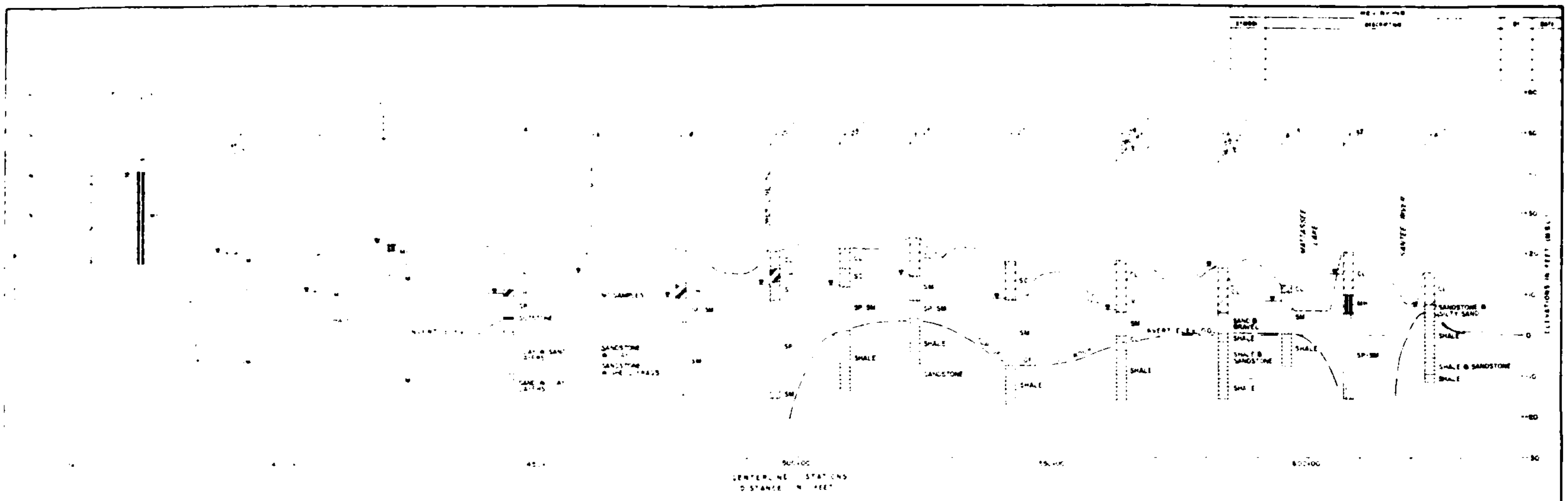
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE 17
DATE MAY 1976		FILE NO 10043

3


SANTEE
SWAMP
PLAN "A"
PLAN "B"
RIVER
POWERHOUSE
RECOMMENDED ROUTE
OF REDIVERSION
LEGEND
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA


LEGEND
SANDSTONE
SHALE
LIMESTONE
TOP OF GEOLOGIC ROCK
GEOLOGICAL PROFILE
INTAKE CANAL C.L. PROFILE
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

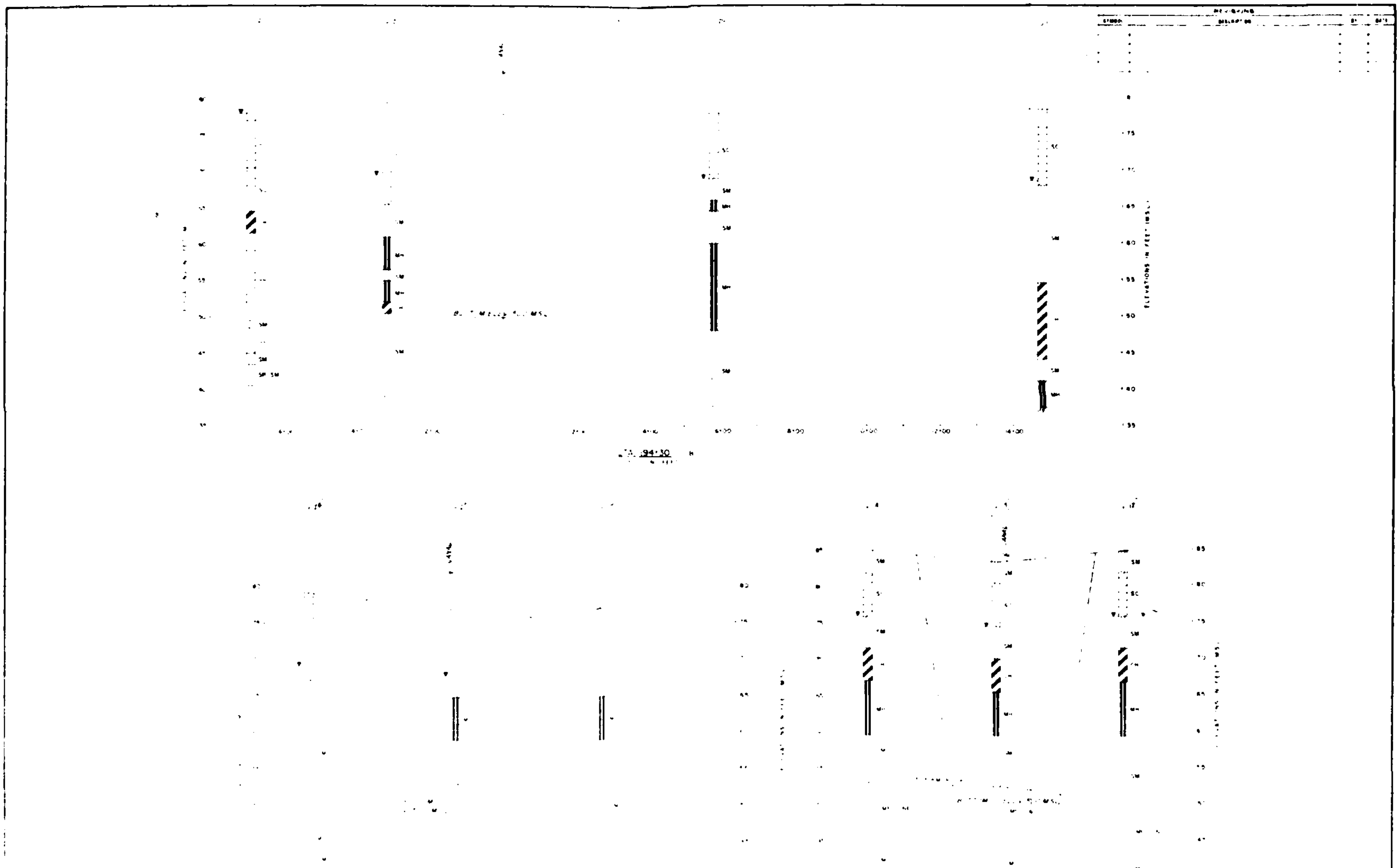


GEOLOGICAL PROFILE

TAILRACE CANAL C.L. PROFILE

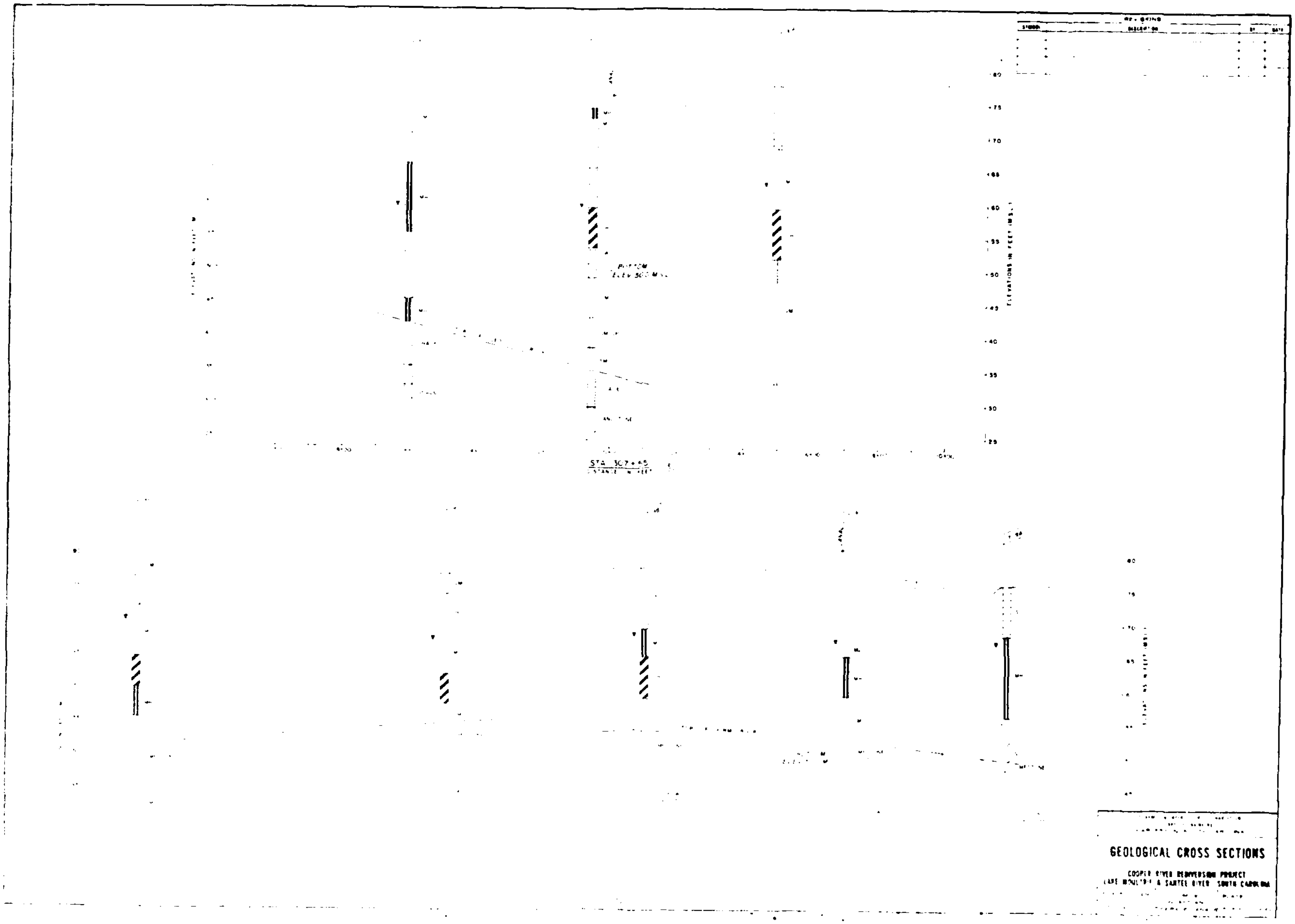
COOPER RIVER REDIVERSION PROJECT

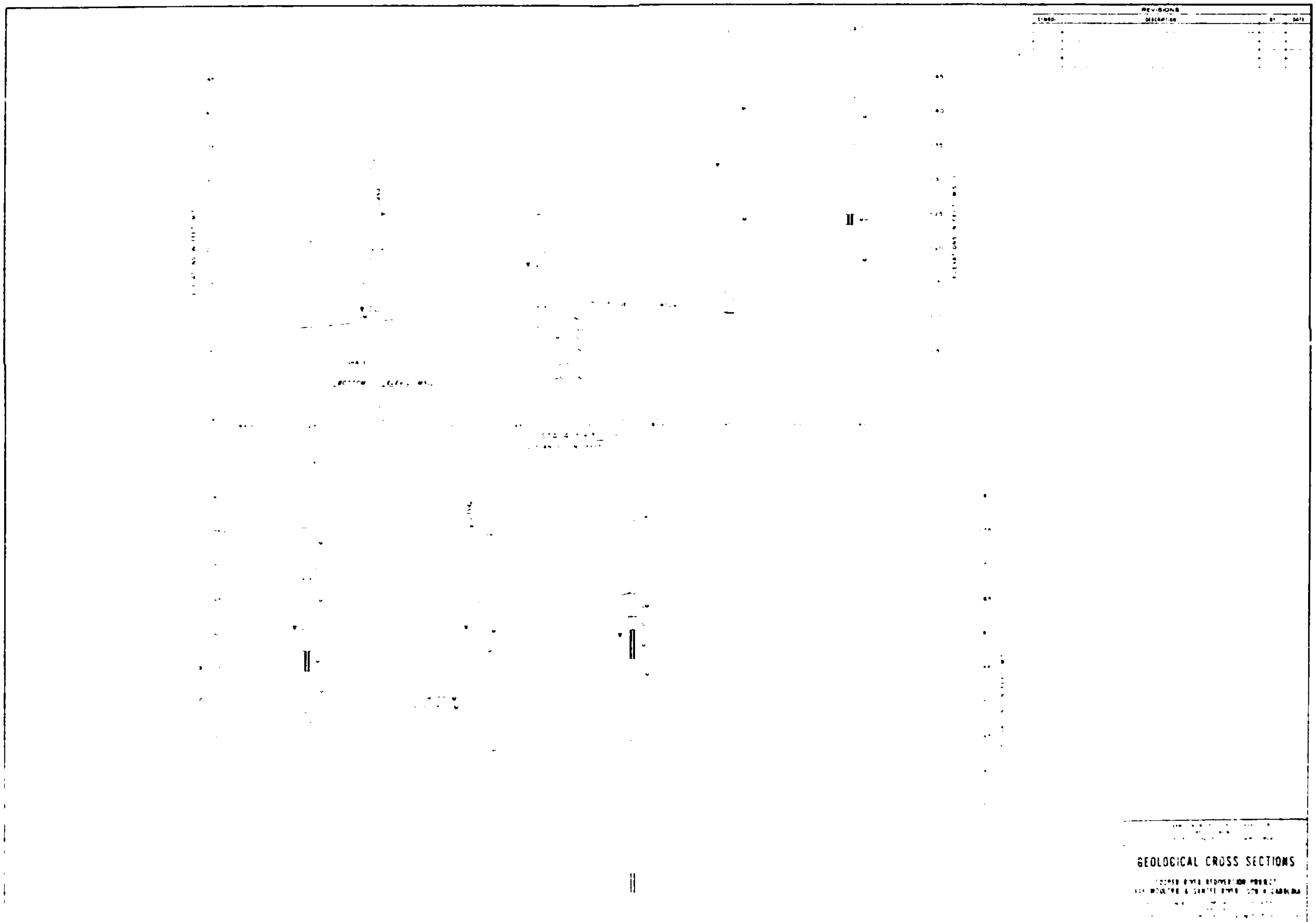
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

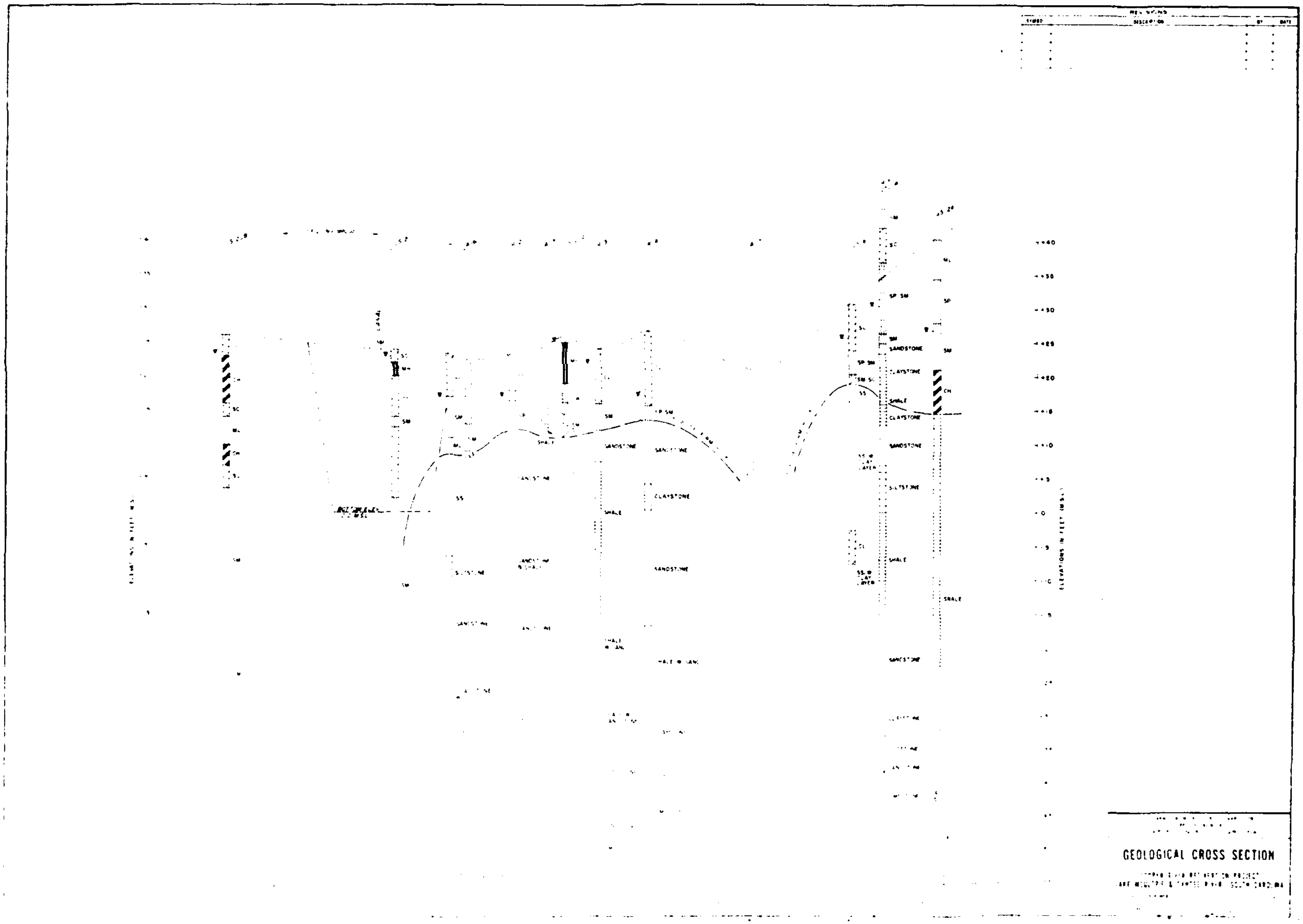


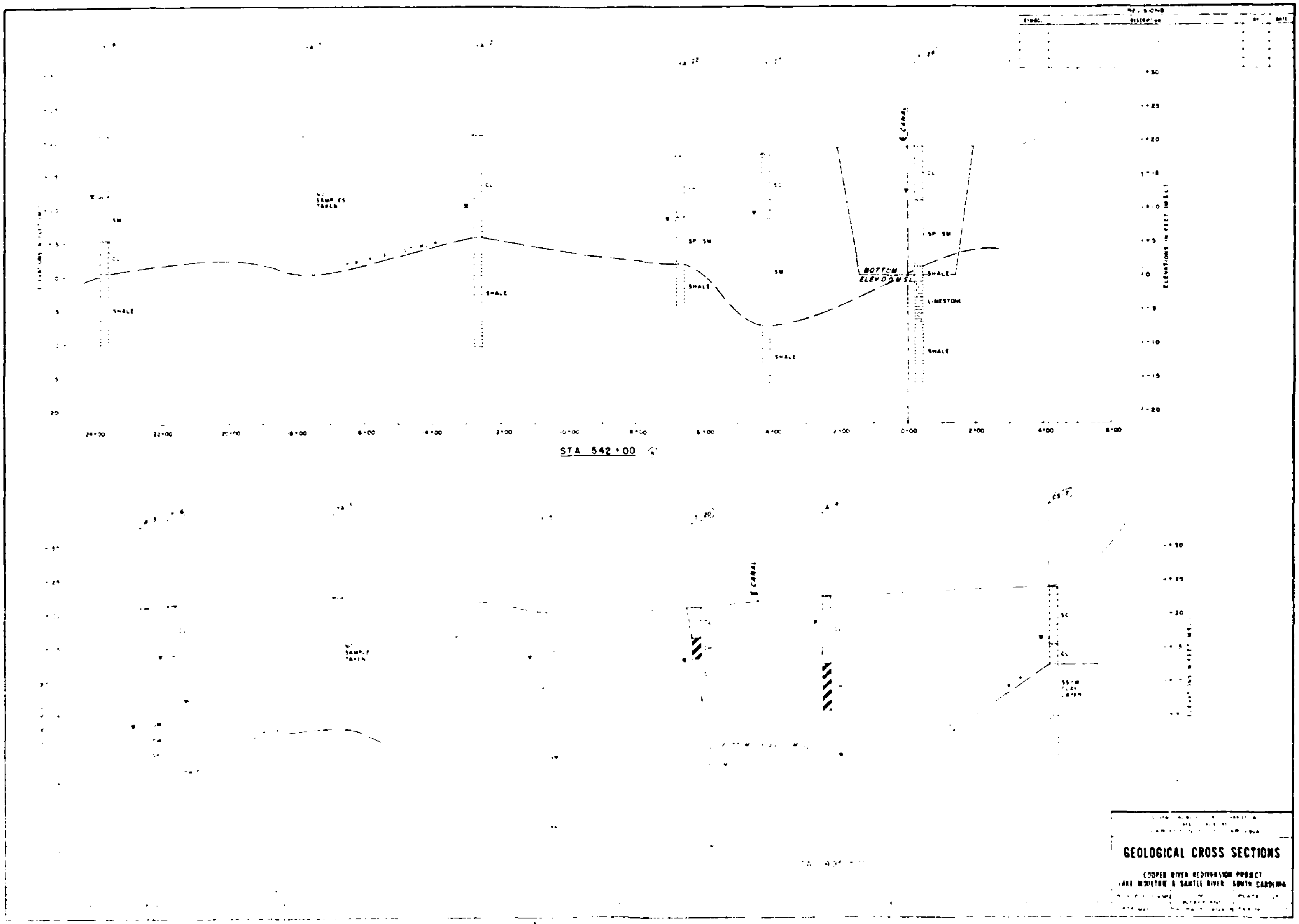
GEOLOGICAL CROSS SECTIONS

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

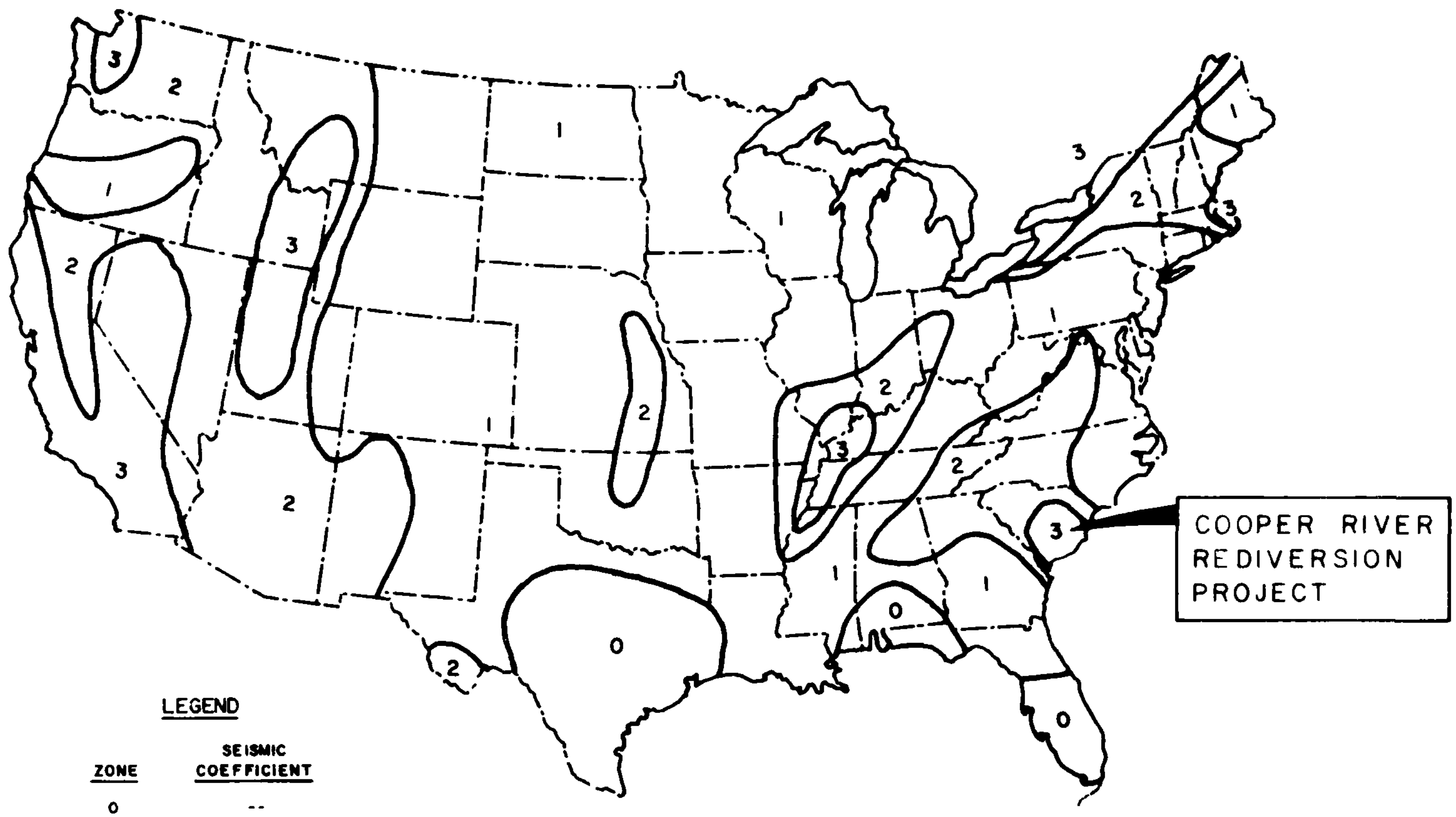

REVISIONS
ELEVATIONS IN FEET (MSL)
STA. 307+45
GEOLOGICAL CROSS SECTIONS
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA


REVISIONS
GEOLOGICAL CROSS SECTIONS


REVISIONS
ELEVATIONS IN FEET (MSL)
SANDSTONE
CLAYSTONE
SHALE
SILTSTONE
GEOLOGICAL CROSS SECTION



BOTTOM
ELEV 0 M.S.L.
SHALE
LIMESTONE
SP SM
SM
CL
SC
SAMPLES TAKEN
SAMPLE TAKEN
ELEVATIONS IN FEET (M.S.L.)
STA 542+00
GEOLOGICAL CROSS SECTIONS
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA



LEGEND

ZONE	SEISMIC COEFFICIENT
0	--
1	0.05
2	0.10
3	0.15

ZONES OF APPROXIMATELY EQUAL SEISMIC PROBABILITY AND SEISMIC COEFFICIENTS (ALGERMISSEN)

SEISMIC RISK MAP
(FROM FIGURE 6 IN EM 1110-2-1902, APRIL 1970)

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX A

GEOLOGY AND SOILS

U. S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

APPENDIX A

GEOLOGY AND SOILS

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GEOLOGY AND SOILS

GENERAL GEOLOGY

GENERAL GEOLOGY

01 REGIONAL GEOLOGY

The Cooper River Rediversion Project lies within the Atlantic Coastal Plain Physiographic Province. Prior to the Upper Cretaceous time, the geologic history of the South Carolina Coastal Plain is obscure. The basement and Triassic rocks have been slightly investigated because of the Charleston earthquake of 1886 and subsequent tremors in and around Summerville. Cooke (1936) has suggested recent movement along Triassic faults as being responsible for the historic earthquakes. Presently, the U. S. Geologic Survey is investigating the basement and regional geology around Charleston to Orangeburg area. The USGS is studying the basic crystalline rocks and attempting to identify offsets in the Tertiary to Mesozoic strata of the area. General stratigraphic relationships are shown on Plate 1.

At the end of Lower and Middle Cretaceous time, the continent warped in the region now occupied by the Appalachian Mountains, causing doming of the Appalachian System, and downward tilting of the lands lying to the east, south and southwest. . The sea then transgressed upon the margin of the continent, possibly as far as the present Fall Line. Basic rocks, thought to represent basalt shields, were tectonically emplaced beneath the Charleston area. After this event, the history of the area was marked by periodic recessions and transgression of the sea. Upper Cretaceous through Tertiary strata underlying Cooper River Rediversion Project are characterized by cyclical sedimentation with intervals of erosion. General stratigraphic time relationships utilized in this report are developed in the works listed in selected references.

02 LOCAL GEOLOGY

The Cooper River Rediversion Project lies within the lower South Carolina physiographic subprovince. The intake canal and the tailrace occupy an eight mile strip of topography trending east-west and lying within the upper third of the Bonneau S. C. 15 minute series quadrangle (U. S. Geological Survey).

The general surface topography of the project is dominated by Pleistocene terrace deposits and the Santee River floodplain. Four stratigraphic units underlie the subsurface. Overburden deposits vary from loose to dense quartzose to argillaceous sands, and unconsolidated clays. Tertiary and older subsurface strata contain shale, limestone, consolidated sands and indurated sandstone.

Geology of the project is also presented in: Powerhouse and Foundation Analysis, February 1976; Entrance Channel in Lake Moultrie DM 3; Site Selection and Geology, DM 6; and The Effects of The Cooper River Rediversion Canal on the Groundwater Regimen of the Saint Stephens Area, South Carolina, October 1975.

03 PRESENT STUDY

This study presents subsurface stratigraphy and discusses the engineering geology significant to the construction of the intake and tailrace canals. Demonstration of the above utilizes cross section and plan view drawings shown on Plates 1 thru 4. Consolidated subsurface deposits and rock units are defined by time-stratigraphy while the overburden units are defined by their engineering considerations. (Such unit definitions are restricted entirely to the geology discussion.)

03.1 TOPOGRAPHY

The canals siting is on a broad terraced plain sloping towards the east. The maximum ground elevation occurs at the upstream end of the project and is 87 feet msl. The minimum relief of 8 feet occurs along the downstream end of the tailrace at its entrance into the Santee River.

A northeasterly trending scarp of a relict beach lies on the eastern shore of Lake Moultrie. This scarp is found in aeolian dune deposits of Pleistocene age and is thought to be an extension of the Pinopolis Peninsula. (The Pinopolis Peninsula lies along the southern shores of Lake Moultrie, trends north by northeast and geologically is a Penholoway terrace dune.)

U. S. Geological Survey noted a linear feature on ERTS imagery of the Lake Moultrie area of South Carolina which he postulated as a fault that may have caused the Charleston Earthquake. The projection of this linear across Lake Moultrie would cross the western extreme of the intake canal. Withington's linear is coincident with the location of the Penholoway dune and its projection into the canal section. No evidence of faulting was found in the tertiary stratigraphy across the canal section. A cross section of the canal entrance is shown on Plate 3. A geologic study of the area, to be presented in Supplement No. 1, DM 6, "Site Selection and Geology," concluded that the Penholoway terrace, Carolina Barrier Island System and the aeolian dune material of the Pinopolis Peninsula is responsible for the Withington lineament.

A near rectangular drainage network occurs along the northeast limits of the terrace and is tributary to the Santee River. Headward erosion by these tributaries is resulting in encroachment on the upland terrace. Consequent erosion features, such as the Santee River flood plain, dominate the tailrace canal landscape. Topography of the undisected upland terrace is gently rolling with low hills of 5 to 10 feet relief. The maximum project elevation of 87 msl occurs northeast of Russelville on the uplands along the intake canal.

03.2 PHOTOGEOLOGY

Photogeologic analysis depicts two distinct zones of minor solution topography. These solution zones are of areas in the subsurface where limestone lies at shallow depth. See Plate 4 for photogeologic map.

03.3 STRATIGRAPHY

The oldest formation underlying the canal is the Black Mingo Formation of Paleocene age. It consists of shale, sands, sandstone and limestone. (See Plate 1). There appears to be a slight structural doming in the Black Mingo. Although the Black Mingo's subsurface rises to elevation 12' msl along the intake canal, invert elevations of neither the intake nor tailrace canal penetrate this formation. The Black Mingo shale facies acts as an aquiclude; thus, it defines the base of the aquifer through which the canal excavation will penetrate. (See Geologic Section A-A on Plate 2).

The Santee Formation overlies the Black Mingo Formation in the subsurface and is a formation of some consequence to excavation considerations. The Santee consists of clays, claystones, sands, sandstones and limestone. As noted in DM 3, "Entrance Channel in Lake Moultrie", the invert elevation of the entrance canal is for the most part 64 feet msl between Stations 0+00 and 89+34. This is due to the presence of a Santee limestone below that elevation which would require blasting for excavation. The easterly trending surface of the formation vertically deepens below the invert elevation of 64 feet msl at the edge of the lake. The canal intercepts the Santee Formation in the vicinity of South Carolina 45 roadway. Invert elevations lower than 54 feet msl can be expected to require blasting from roadway No. 45 to 2500 feet east of highway 52 along the canal sections. In this section the limestone varies from leached fossil hash to competent vuggy rock with an irregular, pinnacled upper surface. This limestone cannot be excavated by dragline without either ripping or blasting. It is anticipated that Santee sandstones may be encountered in sections of the entrance, intake, and tailrace canals. For the most part the sandstones are friable, often laminated with clays and where encountered can probably be excavated by dragline. However, in some area such as to the south of the SCL Railroad bridge the upper portion of sandstone strata are well cemented and hard. Although exploratory borings to date have not indicated such conditions, if such strata occurs at other locations within canal excavation limits, blasting will be required. Shales commonly are very sandy, soft and fissile and probably can be excavated by dragline. Extensive areas of such shale occur within the excavation limits of the tailrace. Shallow remnants of the Dupline-Waccamaw Formations (undifferentiated) overlie the Santee Formation and mark the top of the Tertiary strata. The remnants, occurring within the excavation limits of the canals, consist of limestone, consolidated shell-layered sands, sandstones, and siltstones. The limestones vary from fossil shell hash to well cemented rock and are of limited lateral extent with a thickness generally less than 10 feet. The sandsto . 'ltstones are soft, friable and not well consolidated.

03.4 SURFICIAL DEPOSITS

Pleistocene deposits commonly mark the landward limit or shoreline of the sea and its estuaries at corresponding stages of Pleistocene sea oscillations. Shoreline deposits consist of calcareous sands and clays of medium density and stiffness (10 to 30 blow count material). Laminated marine estuary deposits intercalated with floodplain deposits are found beneath the present floodplain of the Santee River, and on the plateau area adjacent to the floodplain. These deposits consist of coarse sands and gravels of old stream channels and laminated, thin partings of silt, fine sand and clay (all high blow count material, 50 to 80 blows). They are laterally and vertically intermixed with plastic clays and silts containing organic material derived from shallow lacustrine environments during Pleistocene time. The fat clays have a high liquid limit and contain up to 75% montmorillonite. See exhibit A-2.

The present floodplain of the Santee River is mantled by a red and yellow sandy clay and is overgrown with swamp vegetation. These recent deposits vary in thickness from a few feet to tens of feet and, when saturated, present a severe obstacle to movement of equipment.

The residual soils from solution and weathering of limestones of the Santee and Duplin Formations commonly produce a weak silt and clay of low blow count and plasticity. Denser and stiffer soils are present near the surface. This is due to the fluctuating water levels over the solution depressions. Desiccation gives apparent compaction or preloading to such soils. See Plate 4 for photogeologic map and areal extent.

04 GROUNDWATER

The tailrace and intake canals will be excavated in aquifer 1. (See Plate 2). The effect of the canal on the groundwater regimen is treated by the U.S.G.S. Water Resources Division in their report, "The Effects of the Cooper River Rediversion Canal on the Ground Water Regimen of the Saint Stephens Area, South Carolina"; October 1975.

Subsequent to the U.S.G.S. investigation, a better identity of the subsurface had been achieved through numerous borings. Due to the laminated silt, sand and clay layers, the transmissivities in these sediments may be large in the horizontal direction compared to the vertical. This might allow a larger flow rate and head equalization rate of water transmitted from the canal to surrounding sediments than is anticipated by the above mentioned report. However, the presence of substantial shale layers and the absence of highly conductive gravel channels within the tailrace excavation limits should tend to minimize the effect of head difference between aquifers. Locally, minor artesian pressures are present in the overburden, and where they occur, can contribute to slope instability. (See Plate 2 for identity of aquifers and aquicludes).

05 EARTHQUAKE HISTORY

The project is located in a zone 3 earthquake risk area. Charleston experienced a major earthquake in 1886 and has experienced over 400 recorded earthquakes within historical time. A review of the earthquake history of the project area is presented in DM 6, "Geology and Site Selection". The study of the seismic regime of the state has not progressed at the same rate as has the noteriety of its possible dangers and damage to planned structures. A review of programs in progress since 1974 leaves little to alter the available facts on Cooper River Rediversion Project. Since 1974, U. S. Geological Survey studies indicate that the fault mechanism responsible for production of this risk appears to lie to the southeast of the project area and trends away from the geographic locality of Cooper River Rediversion.

Investigative efforts of the U.S.G.S. and the University of South Carolina have shifted to areas between Charleston and Orangeburg. Their deep boring program of 1975 resulted in no definitive results; however, the early 1976 program revealed some strata offsets of up to 15 feet in the Oligiocene - Pliocene deposits. This evidence can be indicative of an existing fault approximately 40 miles southeast of the project in an area between the Edisto and Ashley Rivers. Due to the limitations of the investigations, they were not able to delineate a strike trend for the fault. No evidence of faulting was found in a study of the Moncks Corner area approximately 10 miles south of the project and investigated in 1974 by the U.S.G.S.

Recent attempts by some investigators have been made to downgrade the intensity of the Charleston earthquake of 1886 Dr. Bollinger, Professor of Geology and Seismology at VPI believes that his studies indicate that the 1886 Charleston earthquake still rates an intensity event of X on the Modified Mercalli Scale. (Dr. Bollinger is scheduled to announce the completion and results of his study to the combined southeastern and northeastern section of the Geological Society of America in April 1976.)

In summary, studies to date have not indicated the presence of faults in the Cooper River Rediversion Project area, nor have they led to downgrading of the seismicity of the project.

EXHIBIT A-1

PALEONTOLOGY

United States Department of the Interior

GEOLOGICAL SURVEY

971 National Center
Reston, Virginia 22092

July 10, 1975

Mr. Charles G. Canning
U.S. Army Corps of Engineers
P.O. Box 889
Savannah, Georgia 31402

Dear Mr. Canning:

I recently completed a service report for Dick Inden concerning the palynologic age determination of four samples from the Corps of Engineers drillsite LM-27 on the NE shore of Lake Moultrie, South Carolina. I understand that Dick sent you a copy of the réport which you would like to quote in a report of your own.

I hereby give you permission to use any part of my report for whatever purpose you have in mind.

Sincerely,

Ray Christopher

Raymond A. Christopher

EEG 167

REPORT ON REFERRED FOSSILS

STRATIGRAPHIC RANGE	Eocene	SHIPMENT NUMBER	EEG-75-17
GENERAL LOCALITY	South Carolina	REGION	Berkeley C.
QUADRANGLE OR AREA	Bonneau 15 min. Quadrangle	DATE RECEIVED	6/2/75
KINDS OF FOSSILS	Palynomorphs	STATUS OF WORK	Incomplete
REFERRED BY	Richard Inden	DATE REPORTED	6/17/75
REPORT PREPARED BY	Ray Christopher		

Corps of Engineers drillsite IT-27 on NE shore of Lake Moultrie, South Carolina.

Four samples were submitted for palynological examination from this core at depths of 56.8 feet, 77.5 feet, 88.0 feet and 148.0 feet. The sample from 148.0 feet was barren, but a poor yield of well preserved palynomorphs were recovered from the other three. This location has been given the U.S.G.S. Paleobotanical Collection Locality number R1001, and the samples are labelled:

R1001A - sample at 88.0 ft.
R1001B - sample at 77.5 ft.
R1001C - sample at 56.8 ft.

Sample R1001A (88.0 ft.) contains the most diverse microflora, of which the following forms have been identified:

NUDOPOLLIS TERMINALIS
N. THIERGARTI
AESCULIIDITES CIRCUMSTRIATUS
CARYA cf. C. SIMPLEX
C. sp. 1 (larger, more thin-walled form)
PLATYCARYA sp. 1
TILIAEPOLLENITES sp. 1
?ENGLELHARDTIA sp. (with polar folds)
ALNUS sp. 1
TRICOLPOPOLLENITES sp. 1
T. sp. 2
Tricolporates of the Leguminosae - Sapotaceae - Santalaceae types
Restoniaceae

The assemblage recovered from sample R1001B (77.5 ft.) includes:

NUDOPOLLIS TERMINALIS
ENGELHARDTIA sp. 1
CARYA sp. 1
?ANACOLOSIDITES sp. 1
TRICOLPOPOLLENITES sp. 1
Assorted fragments and plates of dinoflagellates

STRATIGRAPHIC RANGE		SHIPMENT NUMBER	EEG-75-17
GENERAL LOCALITY		REGION	
QUADRANGLE OR AREA		DATE RECEIVED	
KINDS OF FOSSILS		STATUS OF WORK	
REFERRED BY		DATE REPORTED	
REPORT PREPARED BY			

Sample R1001C (56.8 ft.) contains:

NUDOPOLLIS TERMINALIS
TRICOLPOPOLLENITES sp. 1
INAPERTUROPOLLENITES sp. 1
LAEVIGATOSPORITES sp. 1

NUDOPOLLIS TERMINALIS, abundant in all three samples, occurs throughout the Lower Tertiary of the Gulf Coast, with its maximum abundance occurring during Claiborne time (middle Eocene). The species becomes extinct in the latest Claiborne. N. THIERGARTI ranges from the Paleocene (and older sediments) into the lower Eocene of the Gulf Coast. Both Tschudy (1973) and Fairchild and Elsik (1969) consider the species to be restricted to Wilcox (lower Eocene) and older sediments of the Gulf Coast. Only two specimens of N. THIERGARTI were recovered from sample R1001A.

AESCULIIDITES CIRCUMSTRIATUS has been reported only from Wilcox (lower Eocene) and basal Claiborne (middle Eocene) beds of the Gulf Coast. In the samples examined for this report, the species was common in R1001A (88.0 ft.), but absent in the younger samples.

CARYA cf. C. SIMPLEX is reported as occurring throughout the Eocene of the Gulf Coast by Fairchild and Elsik (1969), but Tschudy (1973) did not find it in any of the samples he examined from the Eocene of the Mississippi Embayment except from samples of Wilcox and lowermost Claiborne (lower and lowermost middle Eocene, respectively). C. sp. 1 is reported by Fairchild and Elsik (1969) to occur in Wilcox sediments, but it is more common in Claiborne and younger beds.

The stratigraphic range of PLATYCARYA appears to be restricted to the transition between the Wilcox and Claiborne Groups in the Mississippi Embayment (Tschudy, 1973), although Fairchild and Elsik (1969) and Elsik (1974) consider the genus to range throughout the Eocene.

TILIAEPOLLENITES is a rare form in the Gulf Coast, ranging throughout the Eocene.

STRATIGRAPHIC RANGE		SHIPMENT NUMBER	EEG-75-17
GENERAL LOCALITY		REGION	
QUADRANGLE OR AREA		DATE RECEIVED	
KINDS OF FOSSILS		STATUS OF WORK	
REFERRED BY		DATE REPORTED	
REPORT PREPARED BY			

ENGELHARDTIA occurs in the Wilcox (lower Eocene) and Claiborne (middle Eocene) of the Gulf Coast, and forms with variations in polar folds (as observed in the samples examined here) only rarely occur above the Wilcox in the Gulf Coast.

The genus ANALCOLOSIDITES has not been reported from sediments younger than Claiborne (middle Eocene).

On the basis of the biostratigraphic ranges of the palynomorphs discussed above, it appears that sample R1001A (88.0 ft.) is uppermost lower Eocene in age, and samples R1001B and R1001C (77.5 ft. and 56.8 ft., respectively) are middle Eocene, probably lower middle Eocene, in age.

Ray Christopher

Box 8-C,074 USC
Columbia, S. C. 29208

April 26, 1975

Mr. Charles G. Canning, Sr.
P. O. Box 889
Savannah, Ga.
31402

Dear Mr. Canning:

Enclosed are my core sample reports. From the microfossil evidence, there is no doubt that the upper and lower claystones are of drastically different ages, the lower being of Paleocene age (hence Black Mingo Formation); the upper belonging to a post-Miocene unit, either the middle Pliocene Duplin Formation or the Pleistocene Wicomico Formation.

My identifications have come from the works of Cushman, Puri, Pooser, McLean, Swain, and Schnitker, supplemented by comparison with identified suites of Paleocene, Eocene, Pliocene, and Pleistocene microfossils. I can provide a bibliography i you so desire.

My agreement was to work six samples: in fact I recieved and have worked seven. I have not attempted to identify every fossil species present, but reported those species whose age and identific. ions were certain and diagnostic. You stated earlier that I can keep the specimens; may I publish any of this information? As to the seven samples, I expect payment only for six-I should have so stated above.

If I can again be of service, please let me know.

Sincerely,

Lyle D. Campbell

Lyle D. Campbell

Core: II 27, *Approximately 200.0' East of Lake on ℄*

top of hole: 7?.7 feet

depth interval: 57.0 to 57.5

Description: Calcareous sand with common leached, very chalkey shell. Some gravel persent.

Fauna:

Macrofauna: mollusks- Turritella, Petricola, and Mulinia (preservation too poor for specific identification)

Microfauna:

Ostracoda:

Loxoconcha purisubrhomboidea Edwards	Pliocene to Pleistoce

Foraminifera:

Elphidium incertum (Williamson)	Miocene to Recent
Elphidium articulatum Orbigny	Pliocene to Recent
Nonionella concentrica (Cushman)	Pliocene to Recent

Conclusions: Locality is of Pliocene (Duplin Formation) or possibly of Pleistocene (Waccamaw Formation) age. ~~xxx~~ Exact dating within this interval can not be determined with the material at hand but it is definitely post-Miocene. Assemblage is typical of estuarine and very shallow marine conditions.

Core: IT 27

top of ground: 76.7 feet, Approximately, 200.0' East of Lake.

depth interval: 65.1 to 65.3 feet

Description: Mottled black clay with sand stringers; abundant

Foraminifera, but very few species

Fauna:

Foraminifera:

<u>Guttulina</u> <u>lactea</u> (Walker & Jacob)	Miocene to Recent
<u>Elphidium</u> <u>incertum</u> (Williamson)	Miocene to Recent
<u>Elphidium</u> <u>articulatum</u> Orbigny	Pliocene to Recent
<u>Hanzawaia</u> <u>concentrica</u> (Cushman)	Pliocene to Recent

Conclusions: Locality is of post-Miocene age, either Pliocene (Duplin Formation) or Pleistocene (Wicomico Formation) age

APPENDIX A

GEOLOGY AND SOILS

LABORATORY TESTING

CORRESPONDENCE

INTAKE AND TAILRACE CANALS TESTING

INTAKE CANAL

DISTURBED

TYPE OF TEST	NO. OF TESTS
Atterburg Limits	19
Grainsize Analysis	12
Visuals & Moisture Contents	63
Consolidation (Remolded)	1
Direct Shear (Remolded)	2
$\overline{R}$ (Remolded)	4
Q (Remolded)	6
Unconfined Compression (Remolded)	6
Compaction	2
Wet-dry Cycle Test	1

UNDISTURBED

TYPE OF TEST	NO. OF TESTS
Direct Shear	4
$\overline{R}$ Test	4
Q Test	4
Unconfined Compression	8
Consolidation	1

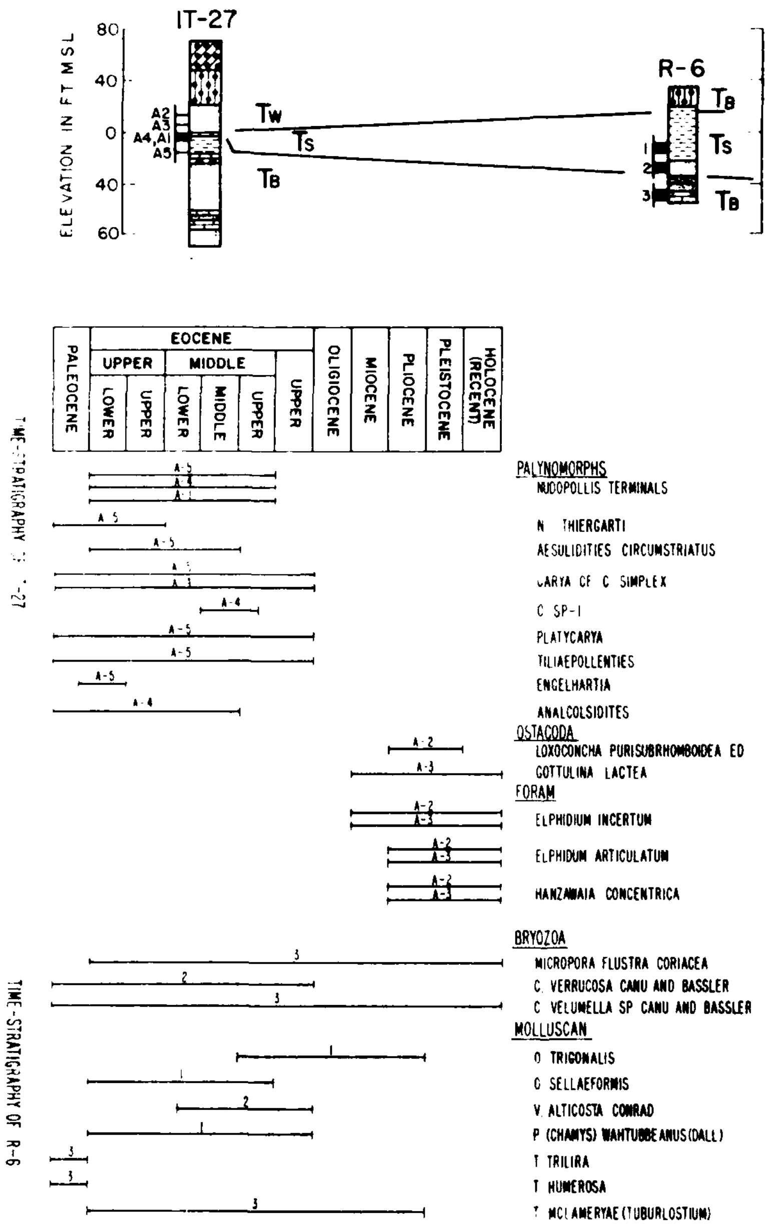
TAILRACE CANAL

DISTURBED

TYPE OF TEST	NO. OF TESTS
Atterburg Limits	23
Grainsize Analysis	17
Visuals & Moisture Contents	100
Compaction	10
Consolidation	5
Direct Shear (Remolded)	5
$\overline{R}$ (Remolded)	8
Q (Remolded)	11
Unconfined Compression (Remolded)	3

UNDISTURBED

TYPE OF TEST	NO. OF TESTS
Direct Shear	5
$\overline{R}$ Test	5
Q Test	5
Unconfined Compression	4

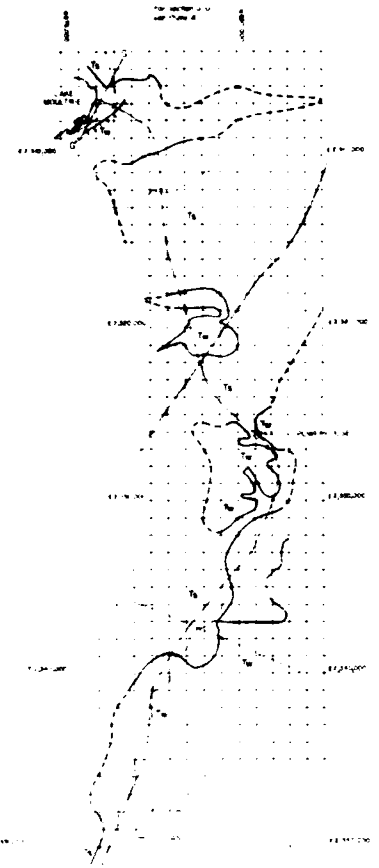


PALEOTOLOGY AND TIME STRATIGRAPHY

EXHIBIT

CORPS OF ENGINEERS

U. S. ARMY



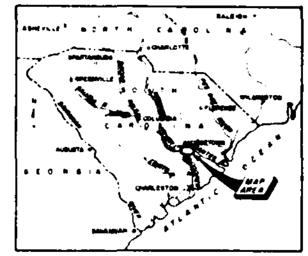
SITE GEOLOGY

GENERAL GEOLOGIC COLUMN

	SERIES	GROUP	FORMATION	SYMBOL	DESCRIPTION	THICKNESS RANGE IN FEET
[illegible]				Q_{AL}	[illegible]	[illegible]
	PLEISTOCENE		[illegible]	Q_P	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	T_W	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	SANTEE [illegible]	T_S	[illegible]	[illegible]
	[illegible]	[illegible]	BLACK MINGO			
	[illegible]	[illegible]	UNDIFFERENTIATED BLACK MINGO	T_B	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	PEE DEE	K_P	[illegible]	[illegible]

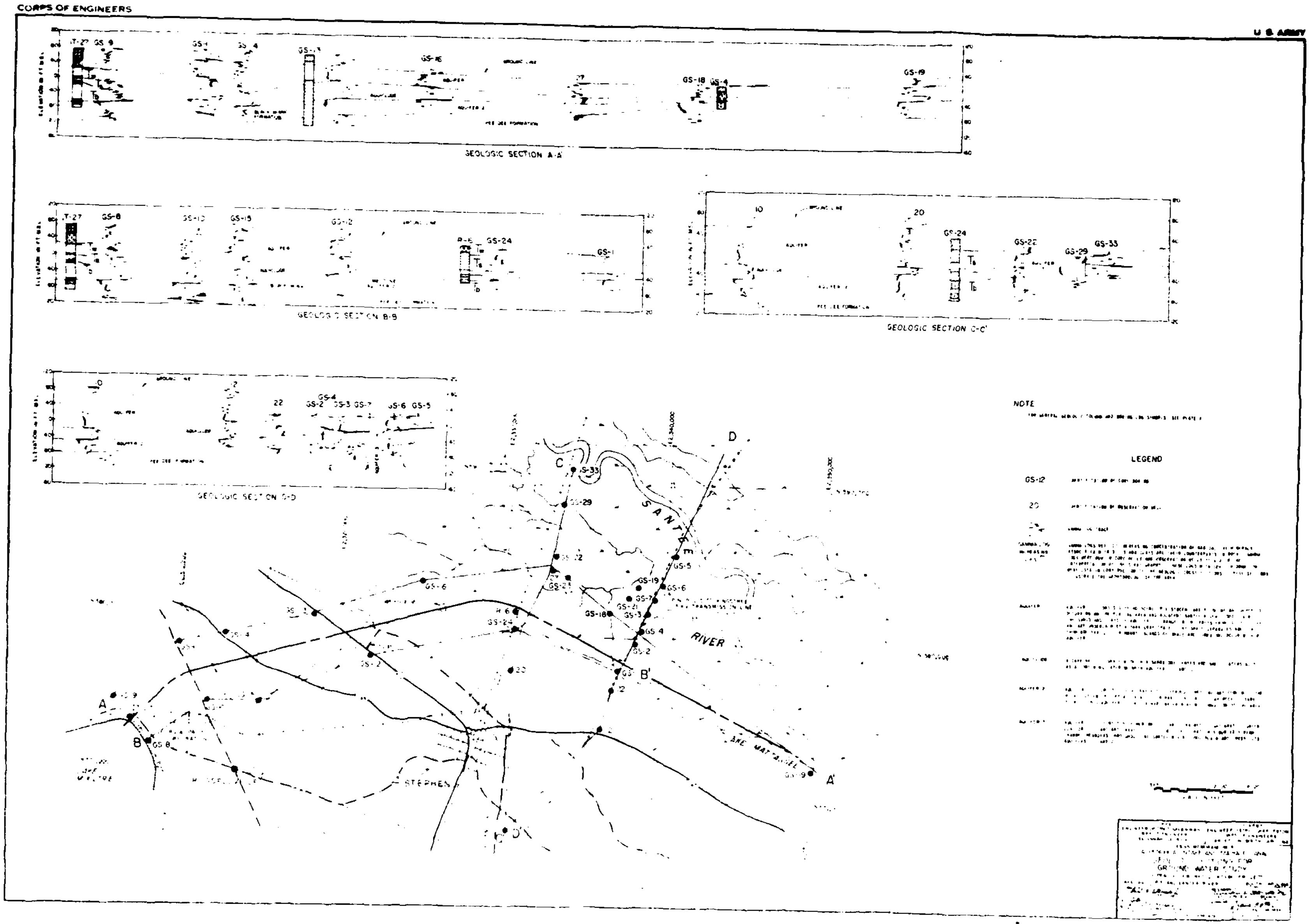
LEGEND

EXPLANATION



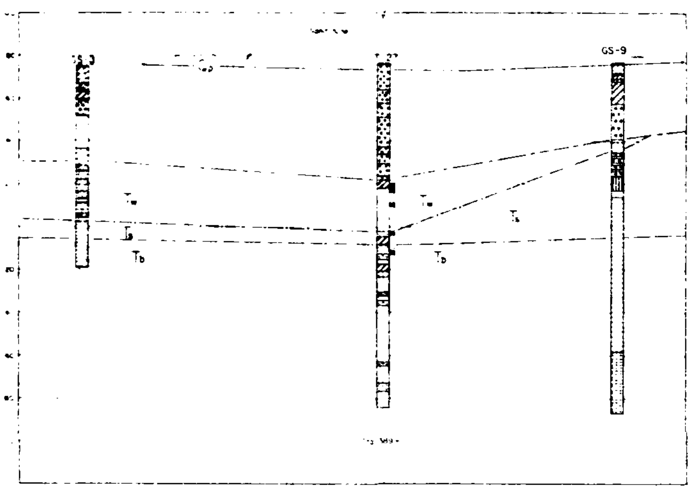
VICINITY MAP

SCALE IN MILES

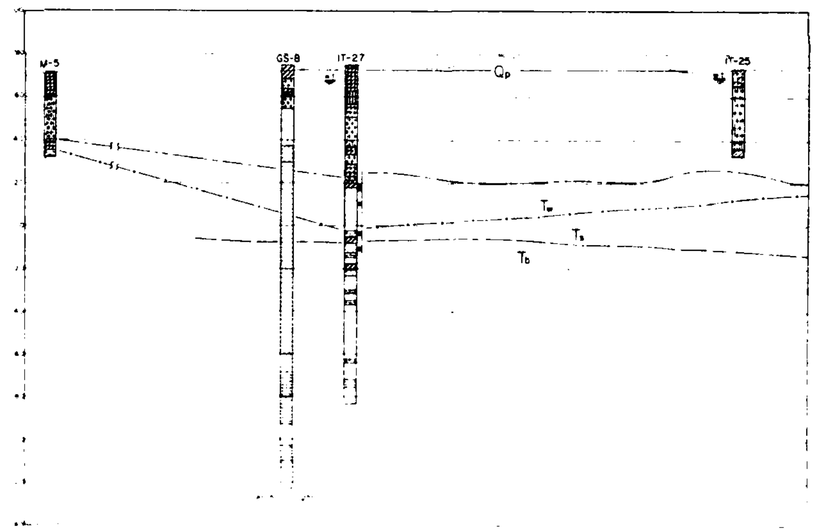

CORPS OF ENGINEERS
U.S. ARMY
GEOLOGIC SECTION A-A'
GEOLOGIC SECTION B-B'
GEOLOGIC SECTION C-C'
GEOLOGIC SECTION D-D'
PEE DEE FORMATION
NOTE
LEGEND
SANTEE
RIVER
STEPHENS
GROUND WATER STUDY

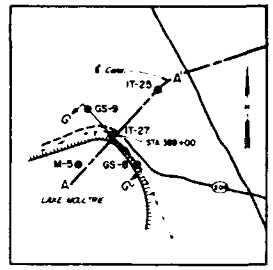
CORPS OF ENGINEERS

U. S. ARMY



GEOLOGIC SECTION LINE G-G





VICINITY MAP

SCALE IN FEET

CORPS OF ENGINEERS

U. S. ARMY

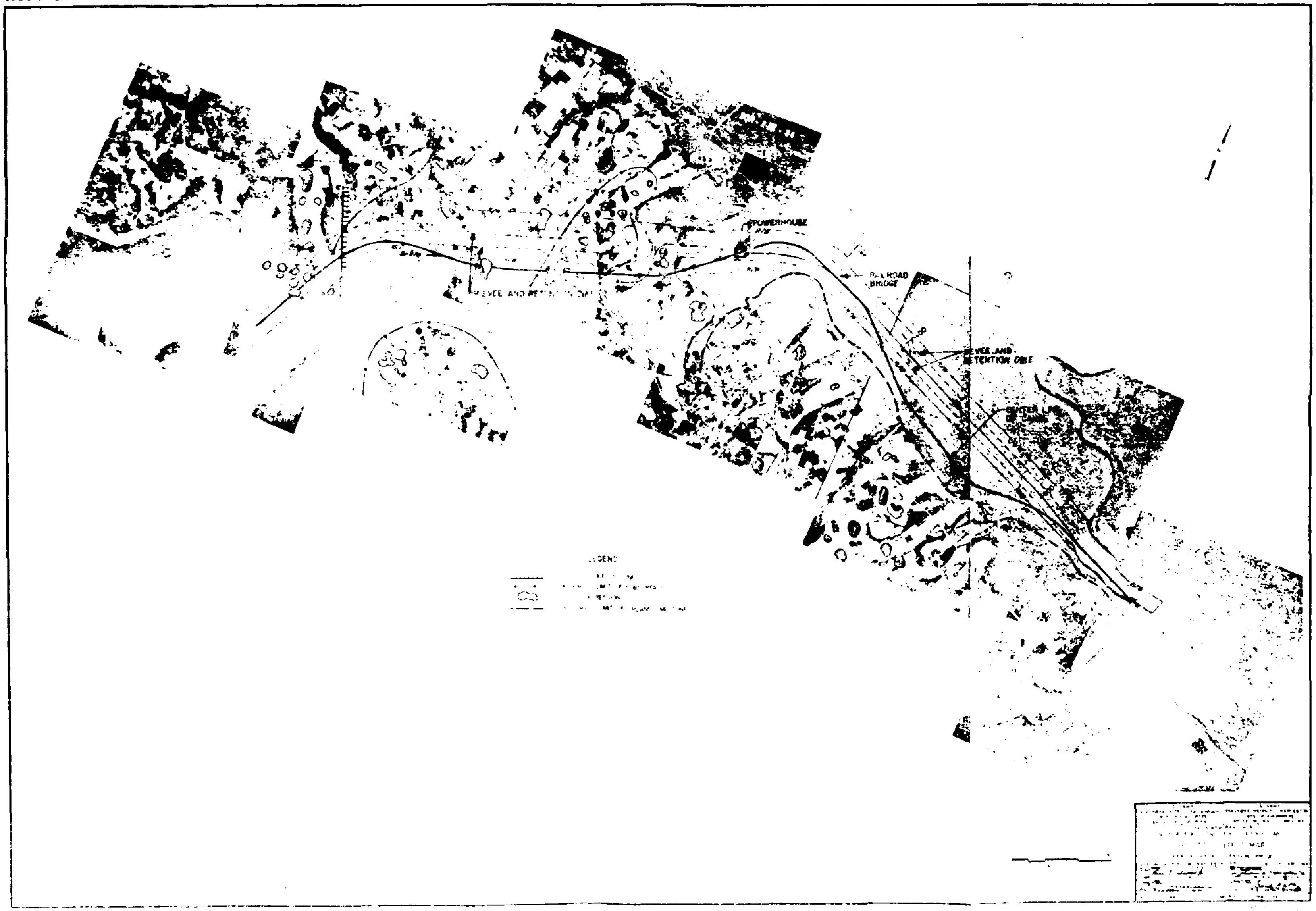


EXHIBIT A-2

PETROGRAPHIC REPORT

U.S. ARMY ENGINEER DIVISION LABORATORY, SOUTH ATLANTIC CORPS OF ENGINEERS MARIETTA, GEORGIA PETROGRAPHIC REPORT			DISTRICT Savannah PROJECT COOPER RIVER REDIVERSION CONTRACT NO.
SOURCE Cooper River Rediversion St. Stephens, S. C.		LAB NO. 235/294	DATE REPORTED 31 March 1975
DATE RECEIVED 21 February 1975	REQ. NO. SAS-ENG-CK-9		WORK ORDER NO. 9132

X-RAY DIFFRACTION ANALYSIS

Petrographic and or X-ray diffraction analyses have been made in accordance with CRD-C 127-67 and or EM 1110-2-2000. Thin section studies, petrographic oil immersion studies, and megascopic examination have been performed as necessary for evaluation procedures and photomicrographs of thin sections, where applicable, appear as figures in the report. X-ray diffraction techniques, if applicable to this testing, include ethelene glycol and heat treatment of sedimented slides as corroborative diagnostic tests to the powder press technique, and X-ray diffractograms appear as plates. Other tests necessary for this investigation are described in the report.

Detailed petrographic descriptions and pertinent remarks regarding acceptance of individual rock types, soils, or fine aggregate and other earth materials are included in the tables. The summary below presents key data resulting from the testing.

1 Incl
1 Figures
Plates
Tables

SUMMARY

X-ray diffraction and petrographic analysis has been made of mottled grey and white sediment from 28.0-29.0 ft. depth of hole 51 from the Cooper River Project. Mineral composition of the sample approximates the following:

Montmorillonite	77%
Quartz	10%
Glauconite	0
Illite (clay)	8 %
*Other	5 %

*Other includes organic matter, feldspar, zeolites, and very minor kaolinite

REPORTED BY: ☐ PHONE ☐ WIRE	TESTED BY JN	CHECKED BY RJS
DATE ____	SAMPLED BY	

SAD FORM 1417
7 May 73

Previous editions of this form are obsolete.

Sheet 1 of 1

Approximate Composition

10 % Quartz
77% Montmorillonite
8% Illite
5% Other (Zeolite, etc.)

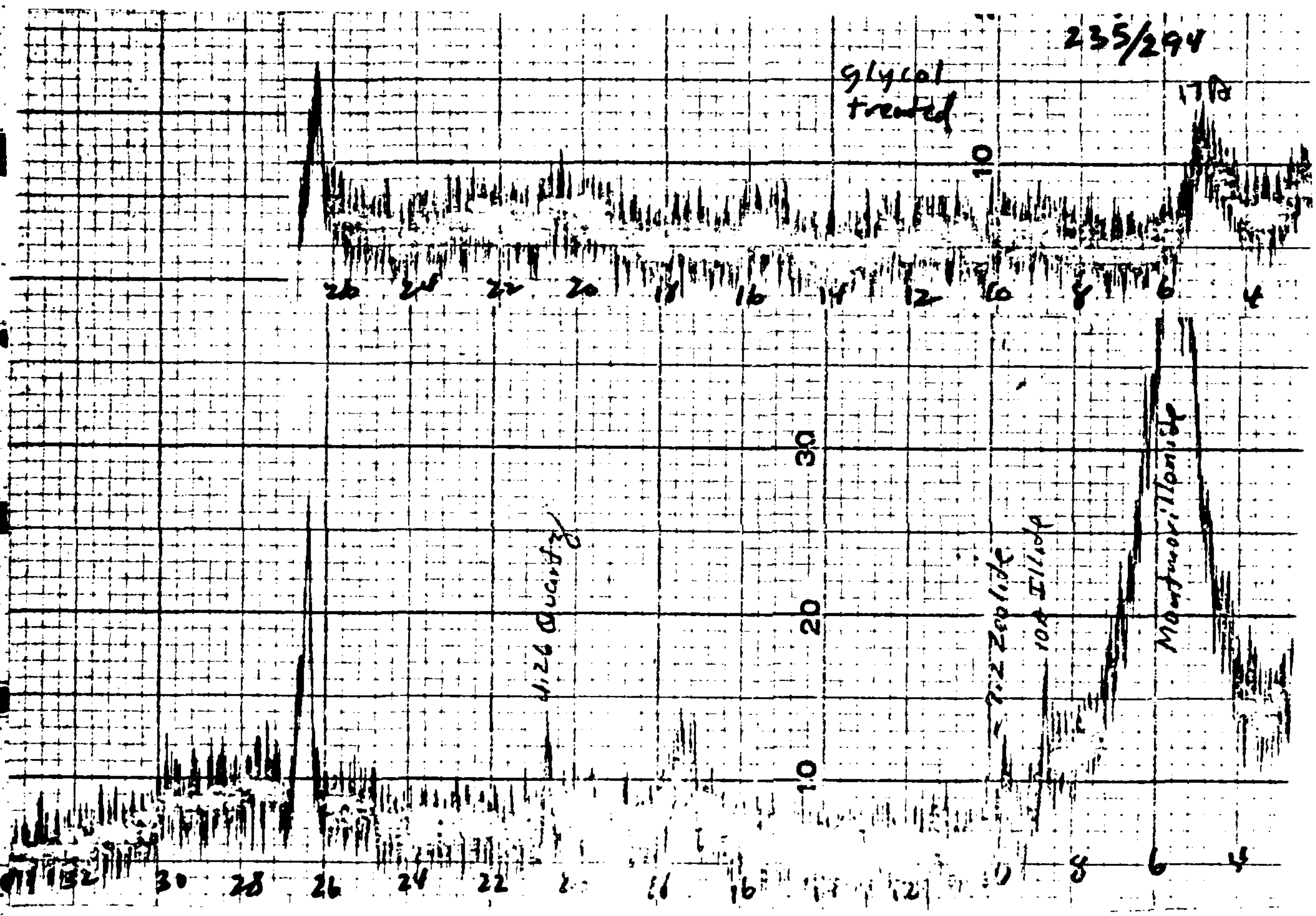
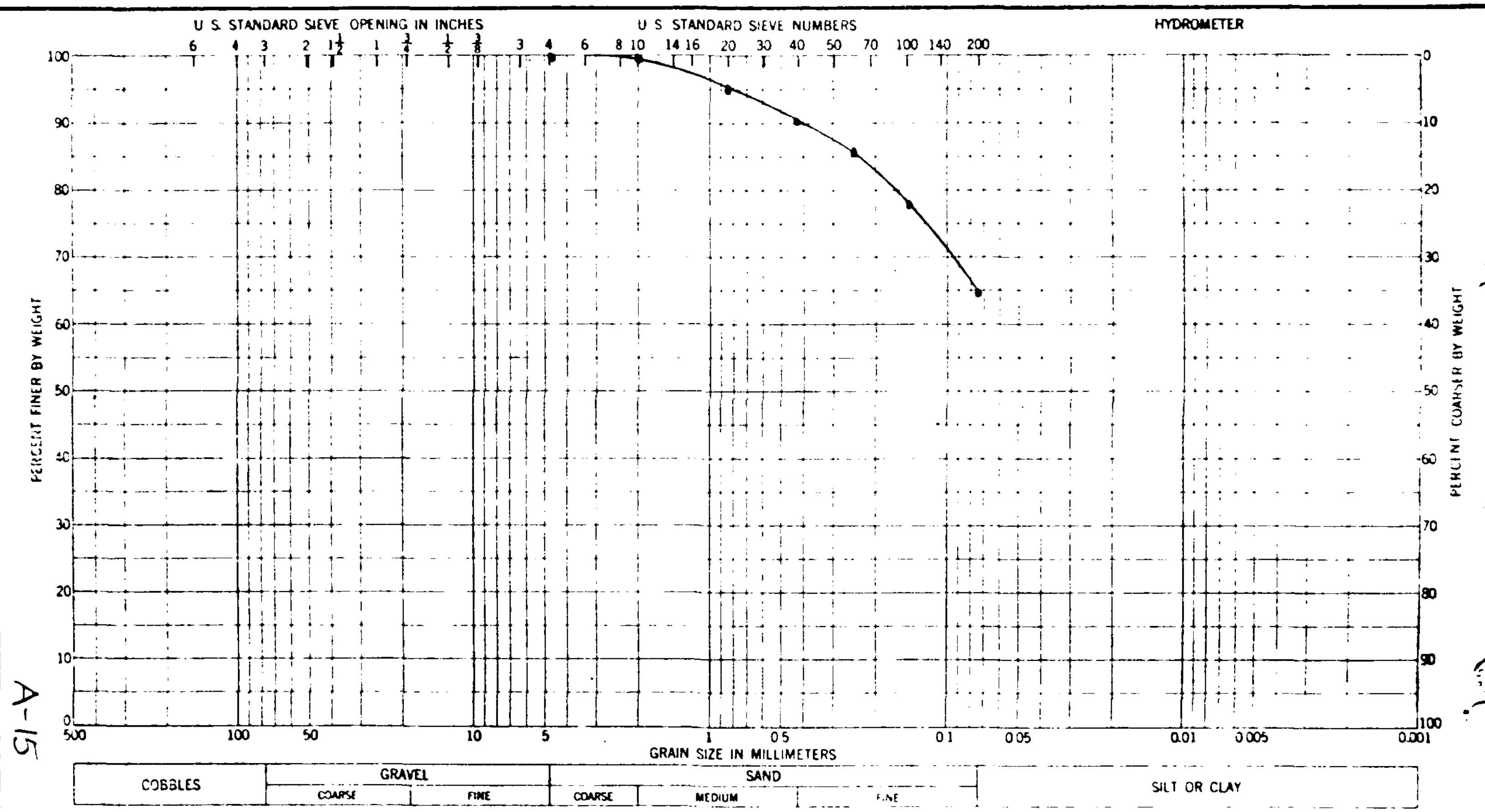


Exhibit A-"2" x-ray diffractogram of sandy clay from 28.0'-29.0' deep of hole 51 of the Cooper River Project.

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY,
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30061

WORK ORDER NO. 9132
REQ. NO. SAS-ENG-CR-9



Sample No	Elevation or Depth	Classification	Nat w %	LL	PL	PI
16	28.8-29.0'	Gray sandy fat clay (CH)	--	131	41	90

Project COOPER RIVER RE-DIVERSION PROJECT
Lab. No. 2357294
Area Powerhouse Site, St. Stephens, S.C.
Boring No 51, Sample No. 16
Date 6 March 1975

GRADATION CURVES

ENG FORM 1 MAY 63 2087

A-15

EXHIBIT a-7 Gradation curve with Atterberg limits

DISPOSITION FORM

For use of this form, see AR 340-15; the proponent agency is The Adjutant General's Office.

REFERENCE OR OFFICE SYMBOL	SUBJECT
SADEN-L	Presence of Montmorillonite in Sample from Cooper River Powerhouse Site

TO Mr. Bill Thompson
SASGF

FROM Director
Division Laboratory

DATE 31 March 1975 CMT 1

1. A few weeks ago we reported to you a soils classification test that appeared somewhat peculiar to me even though we had verified our test results. A copy of the report is attached and you will note the Atterberg limits are very high, yet the samples contained 35 percent sand sizes. To see if a mineralogical analysis would tell us anything, I had Jim Neiheisel make a cursory examination by X-ray diffraction on the material left from the Atterberg limits tests. His results are attached and you will note there appears to be about 75 percent montmorillonite in this material.

2. Since montmorillonite is potentially hazardous, I thought I would bring this to your attention. It may warrant more detailed investigation to either confirm or deny what our rough analyses have indicated.

ROBERT J. STEPHENSON

1 incl
as

Copy furnished:
Mr. Crisp - SADEN-TF

A-16

DA FORM 1 FEB 62 2496 REPLACES DD FORM 96, EXISTING SUPPLIES OF WHICH WILL BE ISSUED AND USED UNTIL 1 FEB 63 UNLESS SOONER EXHAUSTED.

GPO : 1968 O - 333-400

SELECTED REFERENCES FOR GENERAL STRATIGRAPHIC - TIME UNITS

1. Cooke, C. W., 1936, Geology of the Coastal Plain of South Carolina, U. S. Geol. Survey Bull. 867

2. Cooke, C. W. & McNeil, 1952, Tertiary Stratigraphy of South Carolina, U.S.G.S. Prof. Paper 243-B, pp. 19-29

3. Siple, G. E., 1957, Carolina Geological Society Guidebook for the South Carolina Coastal Plain Field Trip

4. Sanders, A. E., 1974, Geologic Notes, Div. Geol., S. C. Dev. Brd., A Paleontologica Survey of the Cooper Marl and Santee Limestone Near Harleyville, South Carolina

5. Beck, W. Jr., 1973, Geologic Notes, Div. Geol. S. C. Dev. Brd., Correlation of Pleistocene Barrier Islands in Lower Coastal Plain of South Carolina, as Inferred by Heavy Minerals

INTRA-ARMY ORDER FOR REIMBURSABLE SERVICES

For use of this form, see AR 37-108 and AR 37-110; the proponent agency is Office of the Comptroller of the Army.

1. RECEIVING OFFICE CONTROL NUMBER	2. ORDER a. NUMBER	b. DATE
☐ FUNDED ☐ AUTOMATIC	SANCA 75-32	14 May 1975
	3. CHANGE ORDER a. NUMBER	b. DATE
	CH 1	18 Jun 1975

4. ORDERED BY (Command, Installation or Activity and address) (Include ZIP Code)

U.S. Army Engineer District, Charleston
P. O. Box 919
Charleston, S. C. 29402
Attn: SANCA

5. TO BE PERFORMED BY (Command, Installation or Activity and address) (Include ZIP Code)

SAD Laboratories
Corps of Engineers
P. O. Box 51
Marietta, Georgia 30061

6. DESCRIPTION OF SERVICES TO BE PERFORMED

1. In reference to SANCE-F 1st Ind. to SADEL basic letter dated 8 Nov 1974, subject "Estimate of Tests & Services, 1 Jan - 30 June 1975".

2. Project Name: Cooper River Rediversion, St. Stephen, S.C.
On or near 6 June 1975, soil samples from the Intake Canal were delivered to the SAD Lab by a Corps of Engineers' employee. These were the second shipment of samples to be sent to your laboratory from the Intake and Tailrace Canals that will be covered by SANCA 75-32 dated 14 May 1975 (SAD Job Number 9283). Other samples will follow as they are drilled along the remaining Intake Canal and Tailrace Canal alignment. Disturbed samples will be sealed in plastic jars and undisturbed samples in 5 Ø Shelby tubes. Soil samples will be drilled along the entire project (between Lake Moultrie and the Santee River). Mobile and Savannah District drill crews and rigs are performing the subsurface investigations. A plan showing boring locations and field logs of borings are attached.

3. It is requested that the soils be tested as stated on the individual worksheets that will follow the shipment of samples to the lab. The cost estimate for total Canals testing will include the following:

(see page 2)

7a. NAME AND TITLE OF ORDERING OFFICER	b. SIGNATURE	c. DATE
JACK J. LESEMANN Chief, Engineering Division		

ORIGINATING FINANCE AND ACCOUNTING OFFICE APPROVAL

8a. ACCOUNTING CLASSIFICATION	b. AMOUNT
96X3122 CG (08134305J22000)	$4,000

c. CHANGE INCREASE AMOUNT $15,500 DECREASE AMOUNT ______ REVISED AMOUNT $19,500

9. Services to be performed pursuant to this order are properly chargeable to the appropriations or other accounts indicated above until ______ (Day - Month - Year) the expiration date of this order.

10a. TYPED NAME AND TITLE OF APPROVING OFFICER	b. SIGNATURE	c. DATE
MARIETTA H. CADE Chief, Office of Comptroller		

ACCEPTING OFFICER

11. THE ABOVE TERMS AND CONDITIONS ARE SATISFACTORY AND ARE ACCEPTED.

a. TYPED NAME AND TITLE OF ACCEPTING OFFICER	b. SIGNATURE	c. DATE ACCEPTED

(1) 1150 visual classifications & moisture content

(2) 50 gradation analyses

(3) 20 Q test

(4) 20 R test (not moisture)

(5) 20 S test

(6) 20 Unconfined compression test

(7) 25 Atterburg Limits

(8) 8 Standard compaction tests (6" mold)

4. The test results should be returned to the Charleston District F&M Section. Please return all unused portions of the disturbed samples. Ship the unused samples by bus (shipping included in cost estimate).

5. It is requested that Clarence Matthews, F&M Section, SANGE-F (803) 577-4171 (ext 319), be notified should any additional information be required and that results be sent to same. It is further requested that Mr. Matthews be notified a few days in advance of trimming of undisturbed samples as it is intended that he visit SADL to observe the trimming and - strength testing.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

IN REPLY REFER TO

NEDED-F

10 February 1976

SUBJECT: Results of Soil Tests, Cooper River Rediversion Project, St. Stephen, South Carolina

District Engineer
U. S. Army Engineer District, Charleston
ATTN: SANGE-F

1. Reference is made to your Intra-Army Order No. SANCA 76-22, dated 12 December 1975, requesting specific soil tests on undisturbed shelby tube samples and on disturbed jar and bag samples from the subject project.

2. Inclosed are duplicate copies of the following data covering results of tests requested:

Inclosure No.	No. of Sheets	Description
1	14	Visual Classifications and Miscellaneous Soil Test Results on Samples from Borings T-9, T-9A, T-11 thru T-20, T-22 thru T-34A and R-6. (Tailrace Canal)
2	12	Gradation Curves on Samples from from Borings T-11, T-14, T-16, T-17, T-23, T-25, T-27, T-30 and R-6. (Tailrace Canal)
3	4	Compaction Test Reports on samples from Borings T-11, T-17, T-27 and R-6. (Tailrace Canal)
4	17	Gradation Curve, Compaction Test Report, Unconfined Compression Test Report, 3-"Q" and 2-"R" Triaxial Compression Test Reports, Direct Shear Test Report and Consolidation Test Report on Composite Sample CS#1, Borings T-14, T-23, T-25 and T-30 (Tailrace Canal).

NEDED-F 10 February 1976

SUBJECT: Results of Soil Tests, Cooper River Rediversion Project, St. Stephen, South Carolina

Inclosure No.	No. of Sheets	Description
5	17	Gradation Curve, Compaction Test Report, Unconfined Compression Test Report, 3-"Q" and 2-"R" Triaxial Compression Test Reports, Direct Shear Test Report and Consolidation Test Report on Composite Sample CS#2, Borings T-11 and T-17. (Tailrace Canal)
6	14	Unconfined Compression Test Report, 3-"Q" and 2-"R" Triaxial Compression Test Reports, Direct Shear Test Report and Consolidation Test Report on Sample B-1, Boring T-27. (Tailrace Canal)
7	7	Undisturbed Sample Log, Gradation Curve, Unconfined Compression Test Report, "Q" and "R" Triaxial Compression Test Reports and Direct Shear Test Report on Sample S-1, Boring T-18 .(Tailrace Canal)
8	7	Undisturbed Sample Log, Gradation Curve, Unconfined Compression Test Report, "Q" and "R" Triaxial Compression Test Reports and Direct Shear Test Report on Sample S-1, Boring T-26. (Tailrace Canal)
9	7	Undisturbed Sample Log, Gradation Curve, Unconfined Compression Test Report, "Q" and "R" Triaxial Compression Test Reports and Direct Shear Test Report on Sample S-1, Boring T-30. (Tailrace Canal)
10	11	Undisturbed Sample Log, Gradation Curve, Unconfined Compression Test Report, "Q" and "R" Triaxial Compression Test Reports (2 each) and 2 Direct Shear Test Reports on Sample S-1, Boring T-32. (Tailrace Canal)

APPENDIX A

GEOLOGY AND SOILS

PIEZOMETER READINGS

APPENDIX A

GEOLOGY AND SOILS

RIPRAP FILTER DESIGN

NEDED-F 10 February 1976

SUBJECT: Results of Soil Tests, Cooper River Rediversion Project, St. Stephen, South Carolina

Inclosure No.	No. of Sheets	Description
11	1	Visual Classifications on Samples from Borings IT-3A, IT-7, IT-13, IT-21 and IT-27 (Intake Channel)
12	13	Gradation Curve, Compaction Test Report, 3 Unconfined Compression Test Reports, 3-"Q" and 2-"R" Triaxial Compression Test Reports and Direct Shear Test Report on Composite Sample "A", Borings IT-3A and IT-13 (Intake Channel)
13	18	Gradation Curve, Compaction Test Report, 3 Unconfined Compression Test Reports, 3-"Q" and 2-"R" Triaxial Compression Test Reports, Direct Shear Test Report and Consolidation Test Report on Composite Sample "B", Borings IT-7, IT-13, IT-21 and IT-27 (Intake Channel)
14	5	Visual Classifications and Miscellaneous Soil Test Results on Samples from Borings R-1 thru R-6, BA-1 thru BA-3, RR-1 and C-4 (Railroad Relocation)
15	6	Gradation Curves on Jar Samples from Borings R-1, R-3, and BA-1 and on Composite Bag Samples C-1, C-3 and C-4 (Railroad Relocation)
16	4	Compaction Test Reports on Composite Bag Samples C-1 thru C-4 (Railroad Relocation)

3. Copies of the above test data were furnished to your Mr. Clarence S. Matthews as they became available.

SADED-F 10 February 1976

SUBJECT: Results of Soil Tests, Cooper River Rediversion Project, St. Stephen, South Carolina

4. As requested, all jar samples and a representative sample of each composite sample tested will be shipped in the immediate future to St. Stephen, South Carolina, in care of Carl's Exon, U. S. Highway 52.

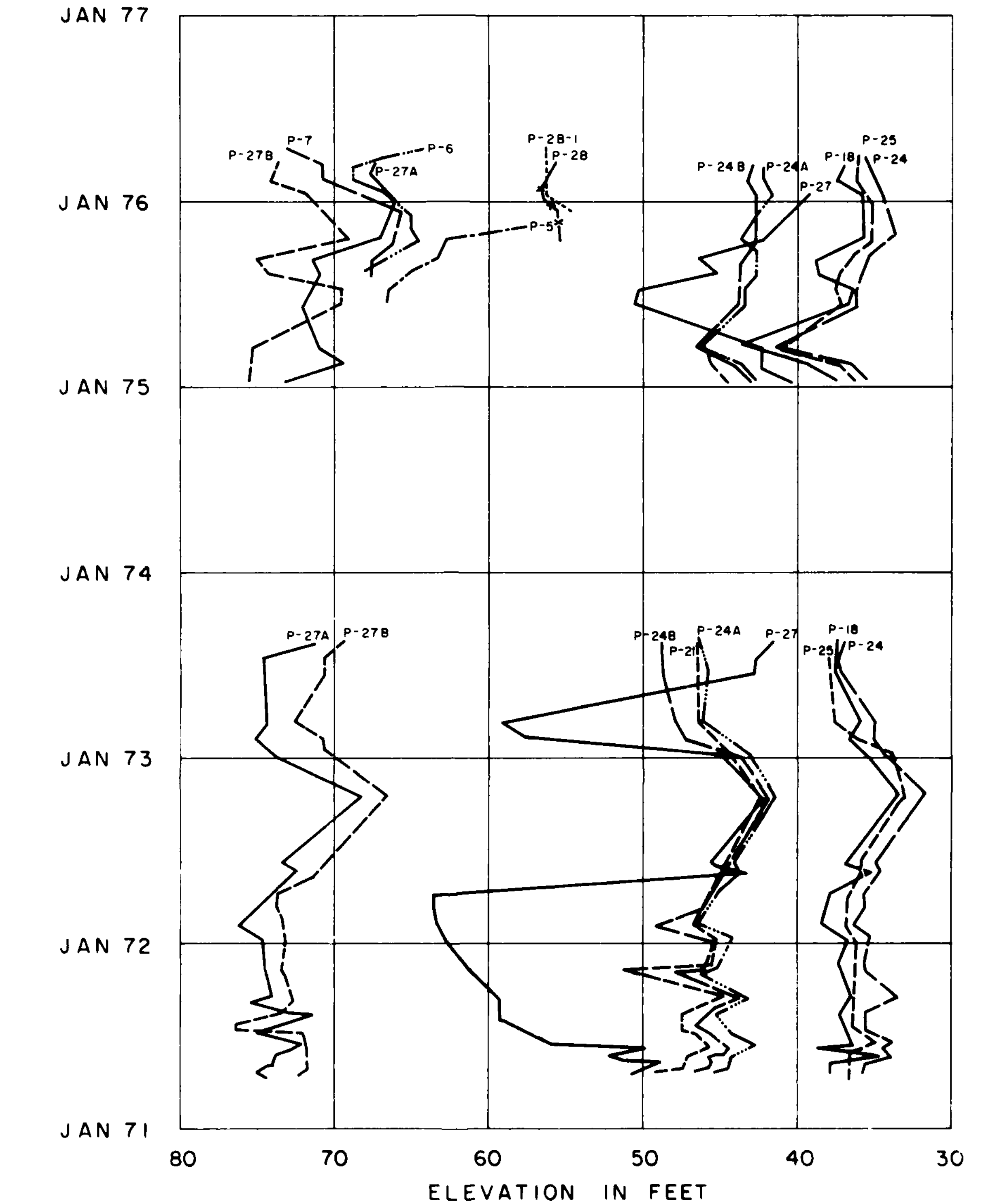
5. Charges for this work will be billed separately on Standard Form 1080.

FOR THE DIVISION ENGINEER:

16 Incls (dupe)
as

JOHN WM. LESLIE
Chief, Engineering Division

CF
DE, SAD
SAD, ATTN: SADEN-TF (Mr. R. Crisp)
w/incls
SADEN-L, ATTN:(Mr. R. Stephenson)
w/incls



INTAKE CANAL

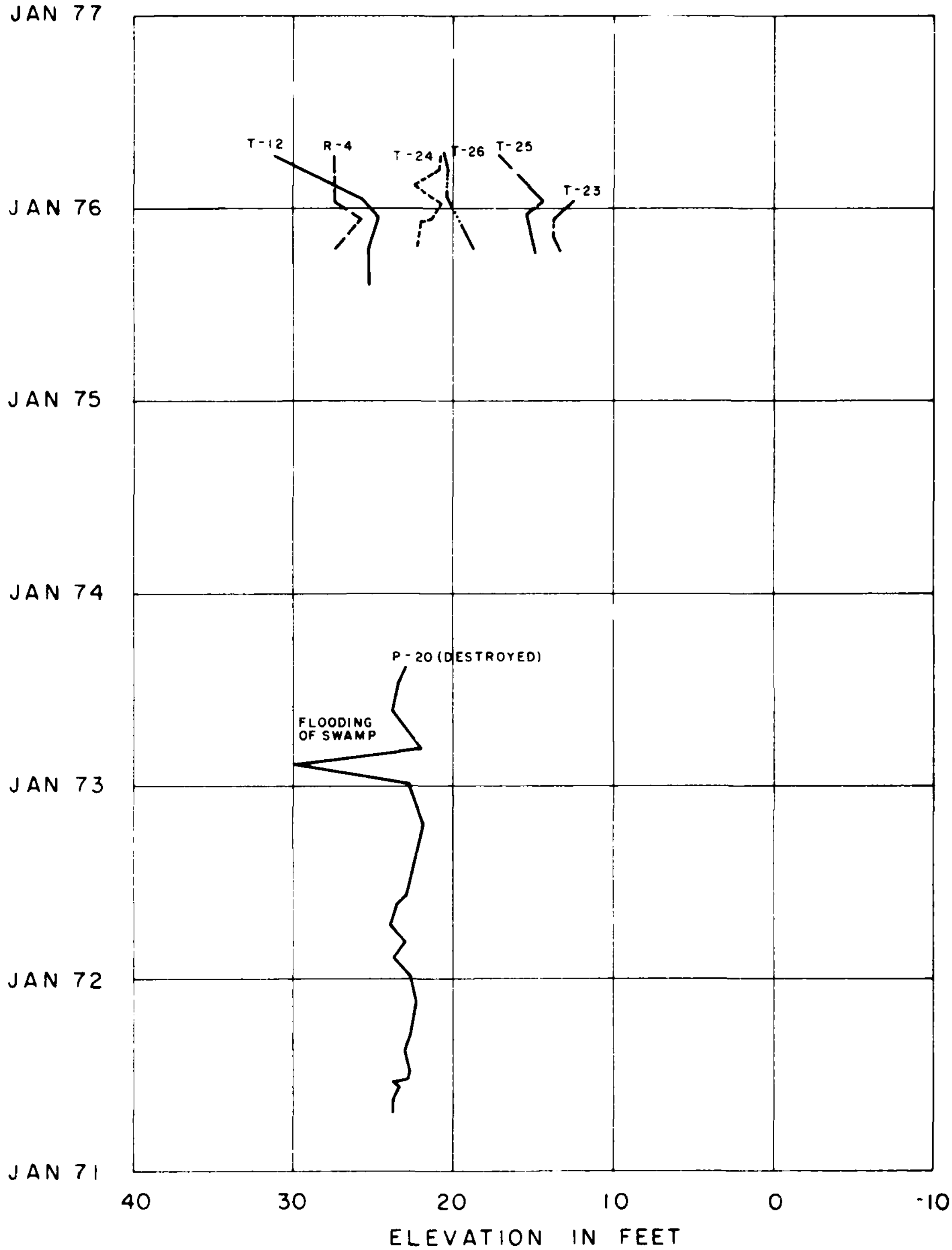
Hole No	LOCATION North	LOCATION East	Ground Elev	Elev Bot Riser
P-5	578,390	2,322,760	75 3	
P-6	576,900	2,316,180	75 5	
P-7	577,000	2,318,200	78 1	
P-28-1	579,870	2,325,135	54 8	
P-18	581,350	2,326,040	50.2	-81 7
P-21	580,440	2,326,420	75 1	-71.9
P-24	581,550	2,326,730	56 8	-57 2
P-24A	581,550	2,326,730	56.8	- 9.9
P-24B	581,550	2,326,730	56 8	+ 37 7
P-25	580,550	2,329,350	39.7	- 66 3
P-27	575,910	2,309,760	77 6	73 4
P-27A	575,910	2,309,760	77 6	52.9
P-27B	573,910	2,309,760	77 6	7 6
P-28	579,870	2,325,130	54 8	

INTAKE CANAL
PIEZOMETER READINGS

COOPER RIVER
REDIVERSION PROJECT

DATE :

PLATE
A-25



TAILRACE CANAL

Hole No	North (Location)	East (Location)	Ground Elev	Elev Bot Riser
CS-20	581,750	2,331,220	23.9	3.9
T-12	581,730	2,329,760	25.1	
T-24	576,040	2,338,090	21.8	
T-26	575,360	2,339,280	20.2	
T-25	575,790	2,339,500	23.9	
T-23	576,430	2,338,280	21.3	
R-4	581,030	2,331,000	26.7	

TAILRACE CANAL PIEZOMETER READINGS

COOPER RIVER
REDIVERSION PROJECT

DATE:

PLATE: A-20

RIPRAP FILTER DESIGN

I Riprap gradation - 12" & 18" Layers
Velocity Range - 0 to 7.6 fps

Percent Lighter by Weight	Limits
100%	84 - 34 lbs.
50%	25 - 17 lbs.
15%	12 - 5 lbs.

II Conversion from weight to size

$$D = \left(\frac{6w}{\Pi \gamma_s}\right)^{1/3} \quad \text{using } \gamma_s = 160\#/cf \text{ (SSD)}$$

Percent Lighter by Weight	Limits of Diameter (ft)
100%	1.0 - 0.74
50%	0.69 - 0.59
15%	0.52 - 0.39

III Diameter in mm = ft(x) 304.8

100%	304.8 - 225.5
50%	210.3 - 179.8
15%	158.5 - 118.8

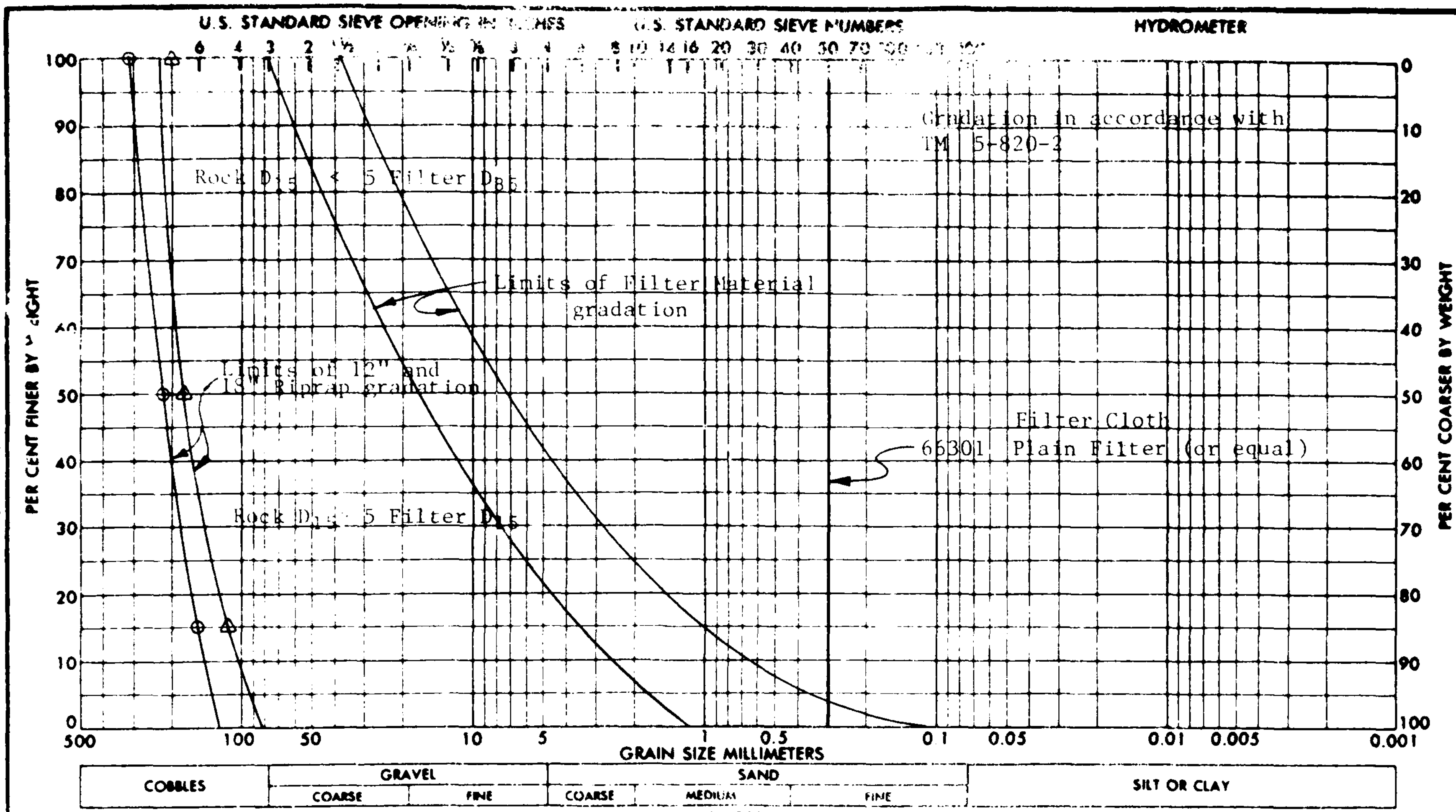
IV Filter criteria -
Gradations in accordance with TM 5-820-2

Rock $D_{15} <$ 5 Filter D_{85}

Rock $D_{15} >$ 5 Filter D_{15}

V Blanket/Filter System

18" System	12" System
18" Riprap	12" Riprap
12" Sand filter	12" Sand filter
.3 mm Filter cloth	.3 mm Filter cloth



Note: 18" Riprap will have the same gradation as the 12" riprap. For the Canal, 18" riprap is required on side slopes between elev. 0.0 and 8.0' msl. 12" riprap is required between elev. 8.0 and 15.0' msl. For drop structures 18" riprap will be required on ditch bottom and 12" on side slopes.

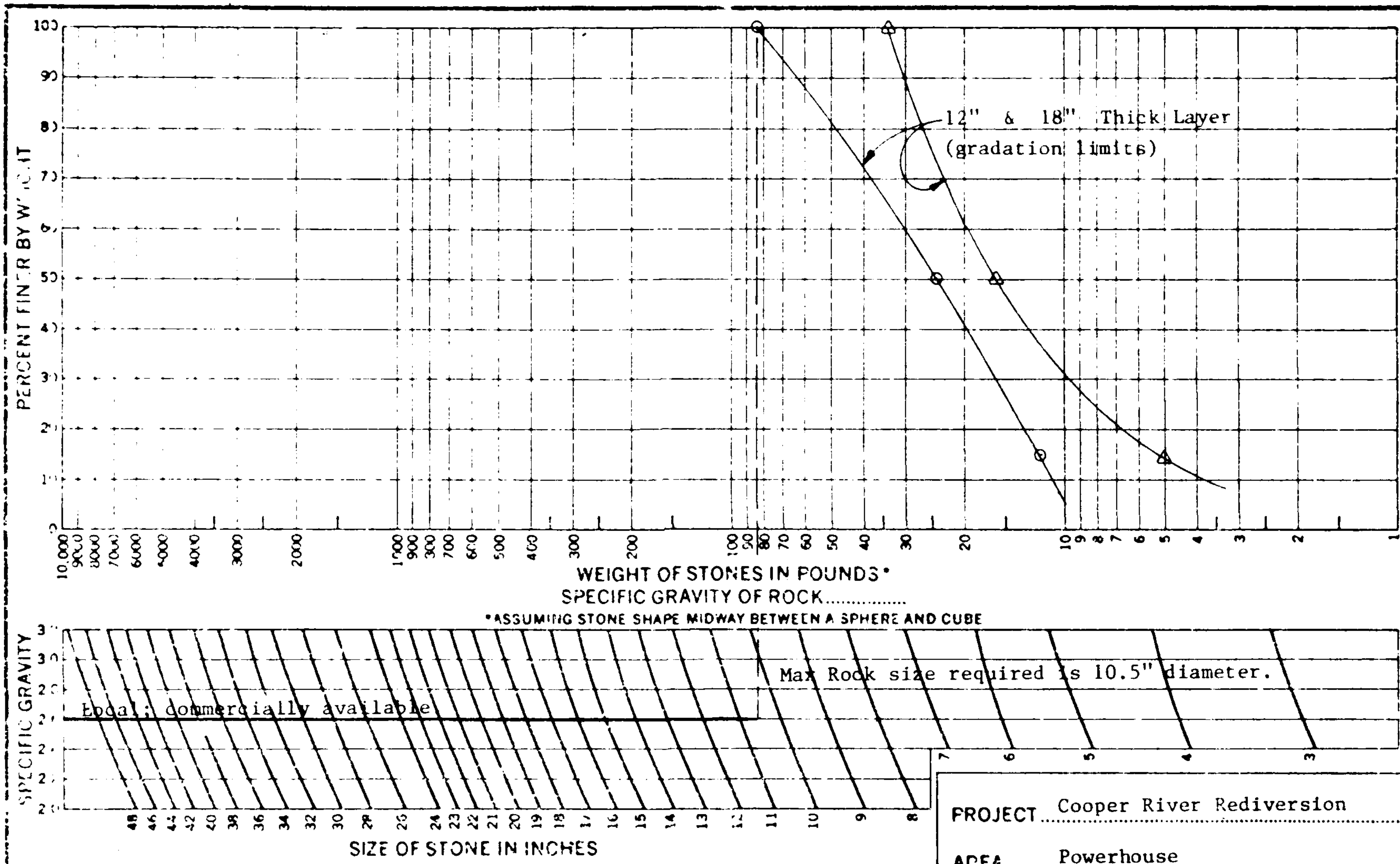
GRADATION CURVES

PROJECT	Cooper River Rediversion Tailrace Canal
AREA	Powerhouse to Sta. 395+62
BORING NO	--
DATE	Apr 76

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE

U S GOVERNMENT PRINTING OFFICE 1963 OF—700-126



PROJECT Cooper River Rediversion

AREA Powerhouse

DATE April 1976

RIPRAP GRADATION CURVES

A-29

ENG FORM 4055

APPENDIX A

GEOLOGY AND SOILS

GEOLOGIST REPORT

ROCK EXCAVATION AT

SCL RAILROAD CROSSING

SASFG SCL Railroad Relocation, Cooper River Rediversion

Chief, Engineering Division
Charleston District
ATTN: SANGE-F

Engineering Division
Savannah District

22 Jan 76
Mr. Titcomb/dwm/263

1. Mr. John Golden of your office requested that the rock core from the subject relocation be inspected by a geologist from this office to determine if shifting of the alinement was feasible. This inspection has been completed and a copy of the geologist's report is attached.

2. The alinement can be shifted a maximum of 200' without encountering material which will require blasting in order to be excavated using a 2 CY capacity drag-line. This evalgation is qualified by the assumptions stated in paragraph 2 of the report. It is also assumed that a 2 CY dragline can excavate to a depth of 30' in a dense soil or clay-shale.

1 Incl
as

JOE G. HIGGS
Chief, Engineering Division

SASFG 14 January 1976

SUBJECT: Trip Report and Memorandum for Files - Cooper River, Realinement of Tailrace Channel

1. During a visit to SAS District Office on 7 January, 1976, Mr. John Golden, Charleston District, advised that consideration was being given to realinement of the tailrace channel which would include the section of the channel passing through the present embankment of the Seaboard Coast Line Railroad. Mr. Golden further advised that within the next few weeks they would be meeting with an AE firm who would design the railroad structure that will bridge the future channel. Prior to the meeting, he would like to determine the feasibility of shifting the channel alinement to the right (southward) of its present alinement by up to approximately 600' at the rail crossing. The present alinement was chosen to keep the rock excavation to a minimum and any realinement to the south would increase the rock excavation.

2. Mr. Golden requested that we visit the site and look at appropriate rock core in an attempt to determine the feasibility of excavating the rock that would be encountered in realinement to the south with a two yard capacity drag line. Further, he assumed the following working criteria: (1) the excavation would not be dewatered, (2) the dragline would operate from the top of the original ground surface, and (3) the rock would not be loosened by blasting or ripping.

3. On 13 January, 1976, accompanied by Charles Deaver, SAS, and Robert Lawson, Charleston District, I visited the site and inspected core from exploratory borings R1 through R4 (see attached cross-section furnished by Charleston District). A soft to moderately hard sandstone forms the bedrock surface to the right (south) of the present alinement. It subcrops at approximate elevation 8 in boring R-1 at the right limit of the present alinement and rises to approximate elevation 13 in boring R-4 approximately 600' south of R-1. This sandstone cap thickens from approximately 2' in boring R-1 to approximately 8' in boring R-4. It also increases in hardness as it thickens toward the south. Southward of boring R-2, the material probably could not be excavated by a 2 cubic yard capacity dragline. (Boring R-2 is approximately 300' south of the centerline of the present alinement). The rocks below the sandstone and above elevation -30 consist of compaction type shales and very weakly cemented siltstone, and would probably normally be considered to be within the capability of a 2 cubic yard dragline.

4. CONCLUSIONS: If excavation of compact, dense material to depths of up to 30' is within the capabilities of a 2 cubic yard dragline, then it appears that at the railroad embankment, the alinement could probably be shifted as much as 200' to the right (southward). Further south than this the hardness

of the sandstone increases as does its thickness, and in all probability would be beyond the capability of 2 cubic yard dragline regardless of depth.

ROBERT G. STANSFIELD
Geology Section

APPENDIX A

GEOLOGY AND SOILS

BORING LOGS

(Bound in Separate Volume)

APPENDIX A

GEOLOGY AND SOILS

LABORATORY TESTING

DATA

(Bound in Separate Volume)

COOPER RIVER REDIVERSION PROJECT

DESIGN MEMORANDUM 9

INTAKE AND TAILRACE CANALS

APPENDIX B

HYDROLOGY AND HYDRAULIC DESIGN

U. S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

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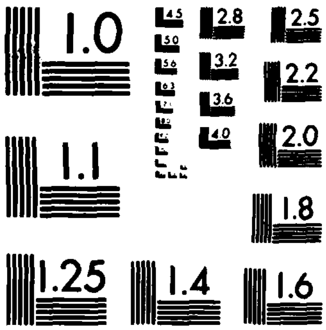
1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
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2.8
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PERTINENT HYDROLOGIC AND HYDRAULIC DATA

SANTEE-COOPER AND COOPER RIVER REDIVERSION PROJECTS

SANTEE COOPER PROJECT

(Completed December 2, 1942)

IMPOUNDMENTS

	Lakes	
	Moultrie	Marion
Watershed	Cooper R.	Santee R.
Dam	Pinopolis	Wilson
Drainage Area (mi^2)		
Pre-Project (approx.)	300	14,700
Post-Project	15,000	14,700
Elevations (feet-msl)		
Maximum Water Surface	76.8 (1)	76.8
Power Pool		
Maximum	75.2	75.7
Minimum	60.0	60.0
Tailwater		
Normal	7.2	27.0
Minimum	-1.5	26.0
Reservoir Capacity (acre-feet)		
At maximum pool (76.8 feet)	1,210,000	1,460,000
At maximum power pool	1,110,000	1,450,000
At minimum power pool	450,000	350,000
Maximum available above 60.0 feet	760,000	1,110,000
Useable for power generation	660,000	1,110,000
Reservoir Area - Maximum (acres)	60,400	110,600
Maximum Water Depth (feet)		35

DIVERSION CANAL

Length - miles	7.5
Bottom width - (feet)	200

DAMS

	Pinopolis	Wilson
Completion Date	7/1/1940	3/23/1942
Type	Earth filled	Earth filled
Top of Dam (feet-msl)	88.0 85.0 (North Dike & Sec. 2, East Dike)	90.0 (north) 88.0 (south)

PERTINENT DATA-SANTEE COOPER PROJECT (cont'd)

	Pinopolis	Wilson
Dimensions		
Length - miles	1.84	7.8
Maximum height (feet)	75.0	48.0
Spillway		
Type	None	Concrete buttressed weir
Length (feet)		3,400
Capacity (cfs)		800,000
Crest elevation (feet-msl)		63.0
Gates		
Type		Tainter
Number		62
Size (feet)		14 X 50
Top, closed position (feet-msl)		76.8
POWER PLANT		
Completion Date	6/28/1942	1950
Release Capacity (cfs)	28,000	500
Average Annual Release (cfs)	15,600	500
Generating Capacity (kw)	132,615	1920
Number of Units	5 (2)	1
Heads (feet)		
Gross static (3)	76.7	49.7
Net effective (4)	67.5	46.7
Minimum net	52.3	31.0
NAVIGATION LOCK		
Interior size (feet)	60 X 180	None
Lift (feet)	75	

SIGNIFICANT DATES	
Santee River Closure	July 1941
Beginning of Impoundment	12/12/1941
Lake Marion reached maximum elevation	9/15/1942

NOTES:

(1) Elevation possible only with prolonged shut down.
(2) Provision is made for future addition of one unit.
(3) Maximum power pool minus minimum tailwater.
(4) Maximum power pool minus normal tailwater and losses.

PERTINENT DATA (cont'd)

COOPER RIVER REDIVERSION PROJECT

CANALS

Basic Design Criteria		
Discharge (cfs)	24,500	
Net Head at Powerhouse (feet)	49.0	
Lake Moultrie elevation (feet-msl)	74.0	
Head Losses (feet)		
Through Powerhouse	0.5	
Total allowable for all bridges.	0.15	
Entrance Channel (1)		
Length (feet)	13,534	
Station 115+34 - 135+34		
Invert elevation (feet-msl)	54	
Bottom width (feet)	375	
Station 0+00 - 89+34		
Invert elevation (feet-msl)	64	
Bottom width (feet)	1500	
Transition length - 89+34 to 115+34 (feet)	2600	
Maximum channel velocity (feet/sec)	3.0	
Side slopes - V on H	1 on 3	

Selected Design

	Intake Canal	Tailrace Canal
Length (miles)	4.30	5.14
Elevations (feet-msl)		
Invert	50.0	0.0
Berm	78.0	26.0
Levee - North side	86.0	45-35
- South side	86.0	35.0
Dimensions (feet)		
Canal bottom width	285	285
Berm width	30 to 130	90
Side slopes - V on H	1 on 3	1 on 3

PERTINENT DATA-COOPER RIVER REDIVERSION PROJECT (cont'd)

Performance

	Intake Canal	Tailrace Canal
Water surface elevation w/24,500 cfs (feet-msl)		
Entrance	74.0	-
Powerhouse	72.6	23.1
Exit	-	21.5
Velocities (feet/sec)		
Maximum expected	3.2	7.6
Steady state - maximum	3.1	3.3

ST. STEPHEN POWERHOUSE

Design Capacity (cfs)	24,500
Number of Units	3
Rated Head (feet)	49.0
Generating Capacity (kw)	84,000
Average Annual Release (cfs)(2)	12,600

HURRICANE SURGE

Design Storm	SPH
Lake Moultrie	
Starting elevation (feet-msl)	75.0
Wind set-up (feet-msl)	82.0
Wave set-up (feet)	0.5
Run-up and freeboard	3.5
Selected Design-Intake Canal Levee (feet-msl)	86.0

SANTEE RIVER FLOOD DATA

Maximum of Record	
Date	July 1916
Peak discharge (cfs)	374,000
Peak stage @ SCL RR. (feet/msl)	47.2
Maximum Spill @ Wilson Dam	
Date	Sept 1945
Discharge (cfs)	155,000
Standard Project Flood	
Peak inflow - Lake Marion (cfs)	631,300
Peak spill @ Wilson Dam (cfs)	578,000
Peak elevation - Lake Marion (feet-msl)	77.32
Peak elevation - St. Stephens Powerhouse (feet-msl)	53.9

PERTINENT DATA-COOPER RIVER REDIVERSION PROJECT (cont'd)

INTERIOR DRAINAGE

Area I	
Drainage Ditch	
Total length (feet)	21,520
Bottom widths (feet)	
2420 feet at:	5
19100 feet at:	10
Average flow depth - 25-yr. flood (feet)	7.0
Side slopes - V on H	1 on 2
Culverts	
Box culverts-concrete (size-feet)	
C-1, S. C. Hwy. 35	3-5 X 4.5
Circular CMP's (size-inches)	
C-2, S. C. Hwy. 204	2-66
C-3, S. C. Hwy. 35 (3)	2-60
Area II	
Drainage Ditch	
Total length (feet)	18,000
Bottom widths (feet)	
4630 feet at:	5
8480 feet at:	15
4890 feet at:	20
Average flow depth - 25-yr. flood (feet)	4.0
Side slopes - V on H	1 on 2 1 on 3
Culverts	
Box culverts-concrete (size-feet)	
C-5, U. S. Hwy. 52 (3)	2-6 X 6
C-6, Powerhouse Access Rd.	2-6 X 7
Circular CMP's (size-inches)	
C-4, S. C. Hwy. 45 (3)	1-48
Area III	
Drainage Ditch	
Total length (feet)	17,880
Bottom widths (feet)	
17165 feet at:	15
715 feet at:	20
Average flow depth - 25-yr. flood (feet)	6.0
Side slopes - V on H	1 on 2 1 on 3

PERTINENT DATA-COOPER RIVER REDIVERSION PROJECT (cont'd)

Drop Structures

	D-1	D-2	D-3
Type	Baffled Chute	Baffled Chute	Baffled Chute
Location-Drainage Area	II	II	III
Design Discharge - 50-yr. (cfs)	795	910	980
Width (feet)	15	20	20
Drop (feet)	16	18	16
Elevations (feet-msl)			
Headwater(4)	65.3	29.7	28.0
Tailwater(4) - No power	47.5	10.8	12.2
- Maximum power		23.0	21.5
Crest	59.0	24.0	22.0
Approach Apron	58.0	23.0	21.0
Outlet Channel Invert	43.0	6.0	6.0

EXTERIOR DRAINAGE - (North of Intake Canal)

Drainage Ditch

Total length (feet)	18,940
Bottom width (feet)	5
Minimum depth of cut (feet)	4
Side slopes V on H	1 on 2

Culverts

Circular concrete pipe (size-inches)

C-7, S. C. Hwy. 45(3)	1-36
C-8, S. C. Hwy. 293	1-36

NOTES:

(1) Data obtained from DM No. 3 "Entrance Channel in Lake Moultrie".
(2) With an average annual release at Pinopolis of 3000 cfs.
(3) Culvert designs are preliminary, since the number and locations of these roads are subject to change.
(4) No flooding in Santee River.

APPENDIX B

HYDROLOGY AND HYDRAULIC DESIGN

1. General. This Hydrology and Hydraulic Design Appendix is intended to serve as the Hydrology and Hydraulics DM for the Cooper River Rediversion Project, since no separate DM on these subjects has been previously submitted and none is scheduled for submission. It was also believed desireable to extract pertinent information and approved design criteria from previous reports and include them in this appendix rather than just reference them. This should aid the user and reviewer of hydrologic and hydraulic data and reduce the amount of time consumed in obtaining and refering to previous reports. Therefore the information contained in this appendix not only discusses the studies conducted for this DM, the procedures used and results obtained, but also this selected pertinent information from previous reports.

2. Prior Reports. The following reports on the Cooper River Rediversion Project are the most pertinent from a Hydrologic and Hydraulic viewpoint:

a. Appendix E - "Hydro-Electric Power Generation Study," Survey Report on Cooper River, S. C. (Shoaling in Charleston Harbor) dated July 1966.

b. DM No. 1 - Vol. I "General Design Memorandum," Cooper River Rediversion Project, dated 18 January 1972.

c. DM No. 3 - "Entrance Channel in Lake Moultrie," Cooper River Rediversion Project, dated March 1974.

3. Pertinent Data. Pertinent Hydrologic and Hydraulic Design Data for the existing Santee-Cooper Project and for the Rediversion Project has been prepared and is shown on pages V-X.

HYDROLOGY

4. Basin Description. The Santee River Basin extends northwest from the coast of South Carolina between Georgetown and Charleston, across the North Carolina state line into western North Carolina. The greatest length of the basin is about 275 miles and the greatest width is about 115 miles. The total drainage area at the mouth is about 15,700 square miles, while the drainage area to Lake Marion is about 14,700 square miles. The Santee River is formed by the confluence of the Congaree and Wateree Rivers, about 145 miles above its mouth at the Atlantic Ocean. The Congaree River, flowing from the northwest, is formed by the confluence of the Saluda (Drainage Area - 2510 mi^2) and Broad Rivers (Drainage Area - 5240 mi^2) at Columbia, South Carolina. The Wateree River (Drainage Area - 5580 mi^2), flowing from the north, is known as the Catawba River in its upper reaches above Big Wateree Creek. A drainage area map of the Santee River Basin is shown on Plate B-1.

5. Adjacent to the Santee River System is the Cooper River, a Coastal Plain stream which historically drained approximately 720 square miles. It comprises a tidal estuary extending 32 miles northward from its mouth at Charleston, to the junction of its east and west branches. These branches have their sources about 20 miles further northwest in a flat, swampy region. At the present time, the flow of the Santee River is directed into Cooper River by the Santee-Cooper project.

6. Topography. The Santee River Basin lies in three well defined physiographic provinces: the Coastal Plain which comprises about 17 percent of the basin drainage area, the Piedmont which comprises about 73 percent, and the Blue Ridge Mountains which comprise the remaining 10 percent of the basin drainage area. The Coastal Plain Province is comprised of coastlands, red hills, and sand hills. The coastlands are flat and featureless, with elevations between sea level and 50 feet msl(1). The red hills are an irregular and interrupted line of high hills joined by a band of sand hills extending inland to the "fall line". The "fall line", which separates the Coastal Plain Province and the Piedmont Plateau, passes through the basin in the vicinity of Columbia and Camden. The Piedmont Plateau extends from the "fall line" at an elevation of about 400 feet to the foothills of the Blue Ridge Mountains at an elevation of about 1,200 feet, and is made up entirely of rolling hills. As its name implies, the Blue Ridge Mountain Region consists of rugged foothills and mountainous terrain. Elevations along the upper watershed boundary vary generally between 3,000 and 5,000 feet. Mount Mitchell, just outside of the basin, with an elevation of 6,684 feet is the highest peak in the southeast.

7. Stream characteristics. Streams in the moutainous areas of the basin have steep slopes of from about 100 to 200 feet per mile. Streams in this region flow on or close to bedrock, have high rates of runoff, and little runoff retention capability. Flood plain deposits are generally narrow and shallow. Streams in the Piedmont section are much flatter than those in the mountains; their slopes ranging between 4 and 11 feet per mile. Flood plains of the rivers gradually widen as they progress through the Piedmont area and become wide swampy areas as they reach the Coastal Plain. Slopes of the rivers in the Coastal Plain Province average about 0.6 foot per mile.

8. Climate. The Santee River Basin has a temperate climate, with warm summers and usually mild winters. Severe cold weather seldom occurs except in the extreme upper watershed, and subfreezing temperatures are usually of short duration. The mean annual temperature is about 62 degrees.

9. Precipitation. Precipitation over the basin occurs chiefly as rainfall. The amount varies with the season and distance from the mountains and the coast. Precipitation is well distributed throughout the year, but is generally highest in July, August, and September. The average annual rainfall is about 49 inches. The maximum and minimum of record are 102.21 and 20.73 inches, respectively.

(1) All elevations in this appendix are referenced to mean sea level datum (msl).

10. Storms of record. Of the several types of storms that occur in the basin, hurricanes and tropical storms are generally the most severe and cause the heaviest, most widespread precipitation. Late afternoon thunderstorms, usually of short duration but with high intensities, may produce large amounts of highly localized precipitation. The more significant storms of record which have occurred over the basin are discussed below.

a. July 1916. The hurricane produced storm of 14-16 July 1916 was accompanied by intense rainfall through North Carolina and South Carolina. Total precipitation in the Santee River Basin varied from 5 to over 15 inches. The storm had two rainfall centers, one at Kingstree, S. C., with 16.8 inches occuring on 14 and 15 July and the other at Altapass, N. C., with 23.7 inches occurring on 15 and 16 July.

b. August 1928. The storm of 13-17 August 1928 was the result of a tropical disturbance that passed over the upper portion of the Santee River Basin. The storm rainfall center occurred at Ceasars Head, S. C., where 13.5 inches fell. Rainfall over the basin varied from 5 to 13 inches. Two-day precipitation totals were 12.33 inches at Linville Falls, N. C.; 11.5 inches at Tryon, N. C.; 8.61 inches at Cherokee Falls, S. C.; and 8 10 inches at Spartanburg, S. C.

c. September-October 1929. This storm was caused by a tropical disturbance that produced heavy rainfall throughout the Piedmont Provinces of the southeastern states. A storm center at Saluda, S. C., recorded 10.98 inches of rainfall. Two-day precipitation totals were 9.91 inches at Newberry, S. C.; 9.86 inches at Spartanburg, S. C.; 9.70 inches at Camden, S. C.; 9.02 inches at Greenwood, S. C.; and 8.75 inches at Mount Holly, N. C.

d. August 1940. The hurricane produced storm of 11-17 August 1940 is without parallel in the South Atlantic states for the great depth of rainfall over a large area. An area of 120,000 square miles experienced rainfall in excess of 4 inches. The heaviest rainfall occurred in the western South Carolina and North Carolina mountains; however, the most intense rainfall occurred at Beaufort, S. C., where 7.2 inches fell in six hours. The storm had four centers of rainfall ranging in magnitude from 12.6 inches at Beaufort, S. C., to 19.61 inches at Swansboro, N. C. Rainfall over the Santee River Basin varied considerably. Rainfall from 6 to 16 inches occurred over most of the upper basin.

11. Runoff and streamflow data. Runoff from the Santee River basin averages about 16 inches a year. This is equivalent to about one-third of the basin average annual rainfall. Annual runoff averages throughout the basin varies from 12.5 to 30.2 inches. The maximum recorded was 46.63 inches on Linville River at Branch, N. C. The minimum of record is 5.23 inches. This occurred on the Saluda River near Columbia, S. C. Runoff varies seasonally; it is highest in the winter and early spring and lowest in the summer

12. Floods of record. No single flood has produced record stages in all portions of the Santee River Basin; however, the August 1940 flood was the most widespread and generally the most severe, especially in the upper basin and in the Catawba-Wateree Basin. The July 1916 flood was the most severe in the lower basin. The August 1928, October 1929, and August 1940 floods were the most notable in the Broad River Basin. The largest floods in the Saluda River Basin were those of August 1908, October 1929, and October 1949.

13. Santee-Cooper Project - Pertinent Hydrologic and Hydraulic Data.

a. General. The Santee-Cooper project was completed in 1942 by the South Carolina Public Service Authority and consists generally of two storage lakes connected by a diversion canal and having a spillway located on the upper lake and a hydroelectric generation plant on the lower lake. The upper lake, Lake Marion, is formed by dikes and a dam (Wilson Dam) on the Santee River. The dam has an integral spillway with a small hydroelectric generation plant both of which discharge into the Santee River. The lower lake, Lake Moultrie, is formed by dikes and a dam in the headwater area of the Cooper River and is supplied by the diversion canal from Lake Marion. Located on the lower lake at Pinopolis is the Pinopolis hydroelectric generating plant.

b. Reservoir System. Lake Marion has a usable storage capacity of 1,110.000 acre-feet above the minimum pool elevation of 60 feet. Maximum pool elevation is 76.8 feet. Lake Moultrie has a usable storage capacity of 760,000 acre-feet above elevation 60. Maximum pool elevation is also 76.8 feet; however, operation of Lake Moultrie at this maximum elevation is not possible due to the hydraulic losses sustained in the diversion canal between the two lakes. Including diversion canal losses, the normal maximum level of Lake Moultrie is between elevations 75.0 and 75.7 feet. The unusable storage capacity for Lake Marion is 350,00 acre-feet and for Lake Moultrie, 450,00 acre-feet. Storage capacity-elevation curves for these lakes are shown on Plate B-2. Head loss curves for the existing diversion canal between Lakes Marion and Moultrie are shown on Plate B-3.

c. Spillway. The spillway located in Wilson Dam on Lake Marion is 3,400 feet long, has 62 - 14x50 foot tainter gates and has a designed maximum discharge capacity of 800,000 cfs. It is a butressed weir constructed of reinforced concrete.

d. Hydroelectric plants. There are two hydroelectric generating plants contained in the Santee-Cooper project. A small hydroplant, rated at 1,920 KW, is located at the spillway on Lake Marion. Average discharge through this plant is 500 cfs. This flow rate is the minimum allowed in the Santee River by the FPC license. The main hydroplant is located on Lake Moultrie at Pinopolis. The Pinopolis plant contains five generating units and has the necessary basic embedded items for a sixth unit. Four of the five units are rated at 30,600 KW each and the fifth is rated at 10,315 KW. Peaking capability is approximately 27,500 cfs. Discharge through the plant, which averages approximately 15,500 cfs, is conveyed to the Cooper River via a dug tailrace canal.

14. Rediversion Project - Hydrologic Considerations. The rediversion project will limit the flow through the Pinopolis hydroelectric plant to an average daily flow of 3000 cfs. The remaining 12,500 cfs will be diverted from Lake Moultrie to the Santee River through a new hydroelectric plant located near St. Stephens. This new plant (St. Stephen) will be sized to pass a continuous maximum flow of 24,500 cfs. A plant of this size will make it possible to utilize for power production about 93% of the available water supply. The power plant will have three turbines rated at 39,000 horsepower at 49 feet of net head and three generators rated at 88,000 KW.

15. Pre and Post Project Flow Conditions. The existing features of the Santee-Cooper project will not be altered by the rediversion project. The average flows through the reservoir system of 500 cfs through the small hydroplant at Wilson Dam and 15,500 cfs through and out Lake Moultrie will not be effected by rediversion; neither will the water supply or inflow into the reservoir system or flows through the Marion-Moultrie diversion canal. The rediversion project will only alter the point at which some of the Lake Moultrie outflow occurs. Therefore, the only flow regimens altered by the rediversion project will be those in the Cooper River and those in the Santee River generally below where the tailrace canal enters the river. This report addresses only the change in flow regimen of the Santee River.

16. Pertinent Gaging Stations. The gaging stations which have a particular significance to this study and are located in the lower portion of the Santee River Basin are shown in Table B-1. Listed in the table for each are the type of gage, period of record, drainage area and recorded maximum and minimum flow rates. The location of the gages are shown on Plate B-1.

17. Update of Survey Report Reservoir Operation Study. The reservoir operation study for the St. Stephen plan contained in Appendix E of the July 1966 Survey Report on Cooper River (See reference paragraph 2a) covered the 56 year period from 1908 through 1963. The computer program used to perform this operation study, for the current studies, was modified to run on a GE-400 series computer. Using this modified program and flow data for the years 1964 through 1975, the reservoir operation study for the St. Stephen plan contained in the Survey Report was extended. The computer print-outs for this period are contained in Exhibit B-1.

18. The reservoir operation study update, used the same procedures as those for the earlier study. The same rating curves, storage curves, hydraulic criteria and rule curve was used. The only change was the method used in obtaining water supply values. For the Survey Report,

TABLE B-1

PERTINENT GAGING STATION DATA

Station number	Station name	Purpose: Stream gaging	Purpose: Lake elev.	Period of record	Drainage area (sq. mi.)	Period of record: Maximum: Date	Maximum: Peak: cfs	Maximum: Peak: Ft. (msl)	Minimum: Date	Minimum: Peak: cfs	Minimum: Peak: Ft. (msl)
02148000	Wateree River near Camden	X		1905-10, 1916 1929-75 (1)	5,070	18 Jul 16	400,000	-	28 Oct 73	155	-
02169500	Congaree River at Columbia	X		1892-75	7,850	27 Aug 08	364,000	-	19 Jan 42	588	-
02169800	Santee River near Fort Motte	X (2)		1952-75	14,100	18,19 Mar 75	-	88.51	-	-	-
02170000	Santee River at Ferguson	X		1908-41	14,600	21 Jul 16	374,000	-	-	-	-
02170500	Lakes Marion-Moultrie diversion canal near Pineville	X		1943-75	-	10 Mar 52	40,200	-	24 Sep 56	61	-
02171000	Lake Marion near Pineville		X	1942-75	14,700	28 Feb 64	-	77.35 (3)	17 Oct 51	-	61.36
02171500	Santee River near Pineville	X		1942-75	14,700	23 Sep 45	155,000	-	23 Feb 47	9	-
02171650	Santee River below St. Stephen	X		1967-75	14,900	21,22 Mar 75	98,900	-	2 Aug 74	491	-
02171680	Wedboo Creek near Jamestown	X		1966-72 1973-75	17.4	26 Aug 71	928	-	(4)	0	-
02171700	Santee River near Jamestown	X (2)		1973-75	-	23 Mar 75	-	21.28	21 Nov 74	-	0.61
02171730	Santee River near Honey Hill	X (2)		1973-75	-	24 Mar 75	-	13.38	26 Dec 74	-	-0.90
02172000	Lake Moultrie near Pinopolis		X	1942-75	-	14 Oct 59	-	76.21	21 Dec 51	-	58.52

Notes:

(1) Stage data only 1886, 1892-1929 except for 1905-10 & 1916
(2) Stage only
(3) Distorted due to high westerly winds
(4) Numerous times during period of record

water supply values were obtained by adding the following items: discharge at Pinopolis, change in storage of Lakes Marion and Moultrie and Santee River flow at Pineville. Since completion of the Survey Report, the USGS has computed daily inflows to Lake Marion from 1942 to date in connection with a study to develop an operational, mathematical model of the lower Santee River system. Flows generated by them were compared with those for the same period used in the Survey Report. The comparison showed that for one month or group of months the USGS flows would be somewhat higher, then it would change to where the Survey Report values would be higher. This pattern continued throughout the comparison period. On a cumulative basis, the USGS values were higher, but only by about 2%. Since the flow values were reasonably comparable, the USGS flow values were used for the 1964 to 1975 period. The water supply values supplied by USGS and used in the update are shown in Table B-2.

19. Design Requirements and Study Concepts - Santee River. The hydrologic data needed for project design purposes consists mainly of identifying the frequency, magnitude and duration of flooding in the Santee River at various project locations. How to best accomplish this, is the subject matter of the following two paragraphs.

20. The USGS has a stream gaging station (02171500) located just below Wilson Dam that has been in continuous operation since completion of the Dam and to date has accumulated 34 years of record. One might suggest that with 34 years of record, sufficient information has been gathered to adequately define the regulated flow regimen of the Santee River. This approach was used in the preliminary GDM studies; however, deriving a discharge frequency curve using data from this gage becomes speculative because of the annual extremes of recorded values due to years with no spills. If this gage is used, the record has to be divided into spill and non-spill years. Also, operational controls for the reservoir system were not consistent throughout the gages period of record. Actual operations prior to 1956 were restricted to pool levels lower than now permitted. Generation during the early years of operation was load limited whereas operations since about 1961 appear to have been limited only by water supply and machine limitations. In addition, Santee-Cooper attempts to evacuate the storage system to the maximum extent possible preceding a flood. Thus, storage effects on each flood of record are different. For these reasons and because more sophisticated computer techniques are now available, the GDM approach was abandoned and this gage was used only for comparative purposes.

21. The study approach adopted for this study was to develop the natural discharge frequency curve using the Hydrologic Engineering Center's (HEC) Regional Frequency Computer Program and discharge records from 5 gages having records ranging from 34 to 84 years. Regulated discharge frequency curves were derived utilizing spill data from the period of record reservoir operation study (1908-1975). This study, using mean monthly flow data, identified spill periods. Using these spill periods, a mean

TABLE B-2

WATER SUPPLY TABLE
INFLOW TO RESERVOIR SYSTEM IN THOUSAND ACRE-FEET

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1964	1518	1644	2132	2750	1305	756	925	1123	1456	3345	1225	1438
1965	1870	1465	2116	1870	1082	1407	1187	1106	748	745	662	509
1966	783	1431	2145	617	609	631	526	508	581	628	727	727
1967	1087	1099	823	400	468	559	801	1719	1076	658	678	1330
1968	2351	940	1122	707	688	1169	1101	751	362	474	691	640
1969	1071	1371	1583	2149	918	922	746	794	813	728	764	932
1970	1012	912	1125	957	729	447	457	1040	722	284	925	609
1971	1241	1753	2074	1181	1400	862	783	1023	708	1353	1271	1650
1972	1946	1641	1003	1030	1316	1294	1009	917	558	580	723	1853
1973	1428	2383	2216	2850	1213	2227	1069	914	1028	554	481	900
1974	1860	1902	1013	1609	1037	743	876	1000	777	559	581	999
1975	2068	1780	3228	1662	1862	1589	1409	937	1143	1225	1283	974

daily reservoir routing was performed by computer for each spill period. These spill hydrographs provided the source data for the regulated frequency curves at selected project locations.

22. Santee River

a. Natural Discharge Frequency. The abandoned Santee River gage (02170000) which was located at Ferguson, South Carolina and HEC's Regional Frequency Computer Program was used to develop the natural discharge frequency curve for the Santee River at Wilson Dam. The Ferguson gage, which was located about 6 miles (14 river channel miles) above Wilson Dam, was inundated by the construction of Lake Marion and consequently, discontinued as a USGS gaging station. Since no significant inflow points occur in this reach, the Ferguson gage can be used without adjustment to define the natural frequency curve. The Ferguson gage collected runoff data for a 34-year period from 1908 to 1941. Using HEC's Regional Frequency Computer Program, No. 723-x6-L2350, a correlation analysis of the annual peak discharges was performed using the Ferguson gage and four upstream gages measuring runoff from the Wateree, Broad, Saluda and Congaree Rivers. These four gages measure 88% of the runoff to Wilson Dam and all are currently operating. They have accumulated record lengths ranging from 49 to 84 years. Regulation effects on the peak discharges of the Saluda and Congaree gages was removed and only the estimated natural peak discharges were used. These values were derived previously for a Flood Plain Information Report. For this report, only the additional years of record were estimated. The results of this correlation analysis are shown in Table B-3. The natural discharge frequency curve derived along with a plot of the recorded and reconstituted flows for the Ferguson gage is shown on Plate B-5.

b. Regulated Frequency

(1) General. The rationale and general procedures used to derive the regulated discharge frequency curves has been discussed previously. The following paragraphs, (1) through (6), discuss these procedures in detail, the study parameters and criteria adopted and results obtained. Study parameter items covered will be: the storage-rule curve, computer program, selection of routing periods, and starting conditions for each routing period.

(2) Rule Curve. Because the rule curve determines the rate of system discharge for any given storage level, it can have an effect on the frequency and magnitudes of spills at Wilson Dam. The rule curve used for these studies is the same as the one used in the hydropower studies contained in Appendix E to the Survey Report (See reference, paragraph 2a). Although this rule curve is not followed by Santee-Cooper, it was adopted for use in these studies for the same reasons it was derived and adopted for use in the Survey Report studies. Also, its use in these studies, provides continuity to project studies and

Table B-3

Correlation Analysis

Lower Santee River Main Stem Gages

Item	Stream Gages(1)				
	02148000	02161500	02169000	02169500	02170000
Drainage Area - Mi^2	5070	4850	2510	7850	14,600
Statistics for Recorded Data					
Mean	4.643	4.788	4.430(2)	4.943(2)	4.918
Standard Deviation	0.340	0.207	0.249(2)	0.242(2)	0.316
Skew	0.724	0.666	-0.489(2)	0.513(2)	0.588
Years of Record	52.0	49.0	49.0	84.0	34.0
Statistics for Recorded and Reconstituted Data					
Mean	4.684	4.825	4.463(2)	4.943(2)	4.851
Standard Deviation	0.355	0.220	0.246(2)	0.242(2)	0.274
Skew	0.594	0.614	-0.356(2)	0.513(2)	0.619
Equivalent Years	72.0	80.7	77.6	84.0	81.8

Note: (1) Gage Names and Locations are:

02148000 - Wateree River near Camden, SC
02161500 - Broad River at Richtex, SC
02169000 - Saluda River near Columbia, SC
02169500 - Congaree River at Columbia, SC
02170000 - Santee River at Ferguson, SC

(2) Estimated unregulated values used - Recorded values adjusted to remove regulation effects.

places the frequency and hydropower studies on the same hydrologic basis.

The rule curve used in the Survey Report hydropower studies was developed to achieve reliable operations for power production during the period of record. The curve provides for an assured refill to maximum levels each spring and limits the average daily discharges to 6,500 cfs during the critical hydrologic periods for power production. The rule curve determines total system discharge based on reservoir storage and the month of the year. The minimum discharge of 6,500 cfs provides 3,000 cfs release from Pinopolis, 500 cfs from the spillway plant and 3,000 cfs from the St. Stephen Plant. At higher levels of reservoir storage, power discharges are increased up to a maximum of about 28,000 cfs which includes 3,000 cfs from the Pinopolis Plant, 500 cfs from the spillway plant and 24,500 cfs (maximum discharge) from the St. Stephen Plant. Releases from the spillway are not determined by the rule curve but are made only as necessary to prevent Lake Marion from exceeding elevation 76.8. The storage discharge rule curve used is shown on Plate B-4.

(3) Computer Program. For the Survey Report, an electronic computer program was developed to explore operation of the Santee-Cooper reservoir system on a monthly basis both with and without the St. Stephen project. Contained in the computer program were system storage-elevation data, power plant characteristics, canal loss parameters, rule curve formula and reservoir operations criteria. Given reservoir inflow, the computer program would derive for each month, power plant releases, system storage, reservoir elevations, spillway releases and power output in terms of average power, monthly energy and end-of-month peaking capability for each individual plant and for the entire system. Monthly values, while adequate for these power studies, are not adequate for frequency studies. In order to correct this deficiency, the computer program was modified to accept and compute daily values. Since only spill data was desired, only the reservoir routing portion of the program was revised. This includes all system inflows, outflows and storages. The power portion would have required additional programming effort, which for these studies did not seem justified. The daily program uses the same methods, procedures and operations criteria as the monthly program. Also, the same storage-elevation data, diversion canal loss parameters and rule curve is used.

(4) Period of Record Routing. All spill periods, as defined by the period of record (1908-1975) monthly routing, was routed on a daily basis using the modified computer program described in the preceding paragraph. Starting conditions, reservoir elevation and flow in the diversion canal, were obtained from the monthly routing for the month immediately preceding the spill month. Selected pertinent information for each spill for the period of record is shown in Table B-4. Information tabulated for each spill are the month and year in which they occurred, peaks of the inflow, spill and routed hydrographs and flow-duration data for the spill hydrograph. Due to the spillway capacity at Wilson Dam, except for the SPF, the maximum elevation at Lake Marion never exceeded the top of power

pool, 76.8 ft. At this elevation, with all gates fully opened, spillway discharge is 544,000 cfs. Because of hydraulic losses in the Marion-Moultrie diversion canal, peak Lake Moultrie elevations would vary. With the reservoir system stabilized, and Lake Marion at 76.8 ft., Lake Moultrie elevation is 75.35 ft. The maximum elevation attained by Lake Moultrie was 75.8, but this was not during a spill period.

(5) Santee River Flood Routing. To determine regulated flow conditions in the Santee River at pertinent project locations, spill hydrographs obtained from the period of record routing were routed down the Santee River to Jamestown using the unsteady gradually varied flow computer program described in the hydraulic design section of this appendix (See paragraph 27). Not all the spill hydrographs were routed since many of them were so near the same peak and shape that to do so would not have been cost effective. Those routed and the peaks obtained at Lake Mattassee are shown on Table B-4.

(6) Discharge Frequency Curves. The regulated discharge frequency curves derived and adopted for project design criteria are shown on Plate B-5. One curve, curve 2, represents conditions in a reach of the Santee River between the St. Stephens Powerhouse and where the Tailrace Canal joins the Santee River. The remaining curve, curve 3, represents the reach immediately below the tailrace-Santee River confluence, where powerhouse discharges becomes a factor. This curve was derived by adding the annual peak discharge rate from the powerhouse, 24,500 cfs, to curve 2. Curve 2 was obtained from a relationship developed between the natural inflow peaks at Lake Marion (curve 1) to the regulated peaks at the powerhouse and Lake Mattassee. This relationship was derived by plotting, for the same flood, the inflow peaks vs the regulated peaks for all the period of record floods routed and deriving a curve of best fit for these plotted points. Using these curves, one for each location, the regulated peak can be determined for any given inflow peak. Only one regulated curve is shown on Plate B-5 because the differences in the curves for the powerhouse and Lake Mattassee are negligible.

c. Standard Project Flood. In response to GDM comments and in order to determine the impact of standard project flood stages on design of the powerhouse and tailrace riverside levee, the standard project flood was determined and routed through the Santee Cooper reservoir system and down the Santee River to Jamestown. The Standard Project Flood inflow hydrograph to Lake Marion was developed by the Savannah District for the Charleston District. The report prepared by them outlining their study is presented in Exhibit B-2. As shown there and on Plate B-6, the standard project flood peak inflow rate is 631,260 cfs. Using the natural discharge frequency curve shown on Plate B-5, the frequency of the SPF would be about 300 years. This is certainly within the frequency band expected for SPF's. This tends to varify both the

Table B-4

PERTINENT SPILL DATA FOR PERIOD OF RECORD FLOODS
(K=1000)

Spill Period Year	Month	Lake Marion Peak Inflow Inst. (Kcfs)	Lake Marion Peak Inflow Daily (Kdsf)	Wilson Dam Peak Spillway Discharge (Kdsf)	Routed Peak(1) - Tailrace-Santee R. Conf. Above Confluence Discharge (Kdsf)	Above Confluence Stage (Ft msl)	Below Confluence Stage (Ft msl)	Spill Durations-Kcfs, Number of Days Flow Greater than: 0	20	40	60	80	100	120	140	160	200	250	300
1908	Jan		83.0	43.2				5	2	1									
	Feb		68.0	35.8				11	4										
	Mar		77.0	28.7				6	2										
	Aug		344.0	304.6	263.2	38.6	39.5	9	8	7	7	7	6	6	6	5	3	2	1
1909	May		74.0	39.8				5	1										
	Jun		101.0	72.2	53.3	26.3	28.4	19	5	3	2								
1910	Mar		50.0	7.3				2											
1912	Feb	215.0	209.0	181.0	146.5	33.0	34.4	51	14	4	4	4	4	3	2	2			
	Apr		50.0	20.7				10	1										
	May		41.0	12.8				13											
	Jun		41.0	5.1				2											
1913	Mar	101.0	98.0	60.2				19	3	2	1								
1915	Jan		71.0	43.0	33.2	24.0	26.7	31	11	1									
	Feb		50.0	22.3				9	3										
	Mar		47.0	11.7				5											
1916	Jul	374.0	368.0	336.2	311.0	40.5	41.4	26	15	13	12	11	10	9	[illegible]	6	5	3	2
1917	Mar	74.0	74.0	41.31				7	3	1									
	Apr		53.0	24.69				17	3										
1918	Dec		68.0	29.11				5	2										
1919	Apr		47.0	18.29				9											
	Feb		50.0	18.45				6											
	Mar		53.0	24.89				20	2										
	Jul	146.0	146.0	112.34	94.1	29.7	31.5	9	7	5	4	4	3						
1920	Apr	53.0	53.0	21.58				19	1										
1921	Feb	158.0	149.0	120.76	101.0	30.2	31.8	22	8	6	4	3	2	1					
1922	Feb	106.0	101.0	51.96	32.3	23.7	26.6	6	3	1									
	Mar		56.0	26.96				22	5										
	Apr		59.0	29.20				13	2										
	Jun		38.0	5.02				4											
1923	Mar	89.0	89.0	42.92	27.9	23.0	26.1	8	3	1									
1924	Apr	50.0	44.0	3.70				6											
	Oct		92.0	49.47				6	3	1									
1925	Jan	146.0	143.0	113.88	103.4	30.4	32.0	23	12	7	5	4	2						
1928	Aug	251.0	248.0	214.32	193.4	35.6	36.7	16	10	8	7	6	6	5	4	3	1		
	Sep		125.0	96.82				28	12	9	7	4							
1929	Mar	160.0	155.0	126.09	113.3	31.2	32.6	38	14	10	8	6	4	1					
	May		47.0	16.90				11											
	Oct	263.0	260.0	225.81	198.4	35.8	36.9	15	9	7	7	6	5	5	4	3	2		
1932	Dec		56.0	27.12	23.1	22.4	25.7	28	6										
1933	Feb		44.0	15.0				10											
1936	Jan		74.0	42.16				17	6	1									
	Feb		53.0	24.50				20	7										
	Apr	245.0	242.0	213.59	201.3	36.0	37.0	28	18	12	10	3	6	5	5	4	2		
	Oct		53.0	20.85				13	1										
1937	Jan	59.0	59.0	23.03	17.9	20.5	25.1	54	12										
	Apr		44.0	12.71				6											
	May		50.0	20.0				10	1										
1939	Mar	86.0	83.0	52.26	43.9	25.4	27.6	20	10	3									
1941	Jul		48.0	17.16	15.1	19.9	24.8	9											
1944	Mar		130.63	70.95	56.5	26.7	28.7	37	8	4	3								
1945	Sep		214.46	174.37	134.5	32.2	33.8	6	5	4	3	3	3	2	2	2			

Table B-4 Continued

Spill Period		Lake Marion Peak Inflow		Wilson Dam Peak Spillway Discharge	Routed Peak[1] - Tailrace-Santee R. Cont Above Confluence		Below Confluence	Spill Durations-Kcfs Number of Days Flow Greater than:											
Year	Month	Inst. (Kcfs)	Daily (Kdsf)	(Kdsf)	Discharge (Kdsf)	Stage (Ft msl)	Stage (Ft msl)	0	20	40	60	80	100	120	140	160	200	250	300
1946	Jan		81.00	46.50	28.5	23.1	26.2	15	3	1									
	Feb		62.18	18.68				4											
1948	Feb		96.02	29.18				3	2										
	Mar		65.80	15.62				4											
	Apr		83.55	53.43				15	3	2									
	Dec		167.94	128.50	72.2	27.8	29.9	6	4	2	2	1	1	1					
1949	Jan		66.54	24.44				3	1										
	May		60.54	30.82				7	4										
	Sep		119.04	81.27	52.1	26.2	28.4	10	4	3	1	1							
1952	Mar		142.16	78.54	46.6	24.7	27.8	23	11	4	3								
1954	Jan		109.14	38.76	13.2	18.4	24.6	2	1										
1956	Apr		53.15	23.86				6	2										
1957	Apr		43.44	9.73				4											
	Nov		78.15	37.99				9	3										
1958	Apr		84.51	56.16	41.9	25.1	27.4	23	6	3									
1959	Oct		93.11	60.66	37.8	24.4	27.1	29	8	2	1								
1960	Feb		98.67	70.41	59.6	27.1	28.9	53	24	8	3								
	Apr		95.51	67.05				17	10	6	2								
1961	Feb		108.65	61.48	37.4	24.5	27.0	10	4	2	1								
	Apr		61.04	30.28				13	7										
1962	Feb		47.87	15.66				4											
	Mar		72.55	41.12				36	10	2									
1963	Mar		101.68	64.82	47.1	25.6	27.9	10	4	2	1								
1964	Mar		79.88	42.10				12	5	1									
	Apr		159.78	129.47	100.3	30.8	32.4	11	7	4	4	3	2	1					
	May		44.11	13.28				5											
	Sep		67.06	38.22				5	3										
	Oct		155.19	123.97	103.0	30.4	32.0	19	8	7	4	3	2	1					
								6											
1965	Mar		42.6	19.5															
	Mar		63.84	34.07	29.3	23.4	26.3	28	6										
	Jun		77.97	47.15				6	4	2									
1966	Mar		91.81	58.49	45.8	25.5	27.7	9	5	3									
1967	Aug		155.92	116.75	75.1	28.2	30.1	7	4	4	2	2	1						
1968	Jan		77.66	40.78	24.5	22.5	25.8	9	2	1									
	Jun		56.63	21.74				4	2										
1969	Apr		112.94	81.52	64.6	27.5	29.3	11	6	5	3	1							
1971	Mar		101.55	65.87	41.3	24.9	27.4	8	3	2	1								
	May		65.32	31.71				6	2										
1972	Jan		78.14	24.72				13	1										
	Feb		46.9	5.9				5											
1973	Feb		101.22	67.48				16	5	3	2								
	Mar		126.87	98.76	74.6	28.2	30.0	32	14	7	3	2							
1975	Feb		47.5	11.32				3											
	Feb		50.0	12.78				6											
	Mar		157.0	124.60	106.3			28	12	7	5	4	2	1					
	May		48.1	16.94				4											
	May		46.9	17.14				6											
	Jun		48.3	17.91				5											
Standard Project		631.26	620.0	578.0	552.6[2]	48.2[2]		10	9	9	6	6	6	5	5	5	4	4	3

Spill Durations (Average and extremes)[3]

	0	20	40	60	80	100	120	140	160	200	250	300
Max	74	24	13	12	11	10	9	7	6	5	3	2
Min	2	0	0	0	0	0	0	0	0	0	0	0
Avg	13.9	4.5	1.9	1.2	0.8	0.6	0.4	0.3	0.3	0.1		

Notes: (1) Routed using unsteady flow comput. program
(2) 3 hr. peak discharge
(3) Does not include SPF

SPF study and the natural discharge frequency curve. Starting lake elevations for the SPF routing through the reservoir system was 76.8 and 75.7 for Lakes Marion and Moultrie respectively. These elevations were considered the practical maximums. The routing could have been started with Lake Moultrie full (76.8) and no discharge through the diversion canal, but since the only way for Moultrie to reach this elevation is for a continued shutdown to occur, it was believed this condition would have been unreasonable. The results of the system and river routings are shown on Plate B-6. As shown there, the spill hydrograph had a peak of 578,000 cfs, the routed hydrograph near the powerhouse a peak of 556,000 cfs, (53.9 stage ft.), the peak lake elevation at Marion was 77.32 ft. and the corresponding peak at Lake Moultrie was 75.55 ft. Critical design elevations at the powerhouse, the floor and deck levels, are set at 57.0 ft. msl. As can be seen, this elevation is well above the SPF level but other design considerations took precident.

d. Stage Frequency. Stage frequency curves were not derived; however, the stage for any given frequency flood can be determined at two project locations. These are near the Powerhouse and at Lake Mattassee. A stage-discharge rating curve for each of these locations are shown on Plate B-7. They were derived using stage-discharge data obtained from the Santee River flood routings using the unsteady flow computer program. By using these ratings curves' and the discharge frequency curves on Plate B-5, the stage for a given flood frequency can be derived.

e. Stage and Discharge for Selected Flood Frequencies. A summary of discharges and stages for selected frequencies are shown in Table B-5. This information was obtained from the discharge frequency and discharge rating curves shown on Plates B-5 and B-7, respectively.

Table B-5

Design Discharges and Stages - Santee River

Flood Frequency (Years)	Inflow to Lake Marion (1,000's cfs)	Powerhouse(1) Discharge (1,000 cfs)	Powerhouse(1) Stage (Ft. msl)	Lake Mattassee(2) Discharge (1,000 cfs)	Lake Mattassee(2) Stage (Ft. msl)
10	157.0	108.0	36.5	130.0	31.3
25	250.0	188.0	41.2	205.0	35.1
50	330.0	265.0	44.6	282.0	38.8
100	425.0	365.0	48.2	380.0	42.5
SPF	631.0	555.8	53.9	576.9	48.3

(1) Mile 59.5 - See Plate B-6

(2) Mile 51.7 - See Plate B-6, below Tailrace Canal and Santee River confluence

f. Flow Duration. An analysis of the spill data obtained from the period of record routing indicates that spills will occur about 5.5 percent of the time. A flow-duration curve of these spills was prepared and is presented on Plate B-8. Additional flow duration data for spills at Wilson Dam is presented in Table B-4. Flow-duration curves nearer the project limits (Powerhouse or Lake Mattassee) were not derived because all the spills from the period of record were not routed down the Santee River. However, the flow duration data at Wilson Dam will be quite similar to the flow duration near the rediversion project. The flow characteristics near the project will have peaks somewhat lower and flow durations somewhat longer than those immediately below the dam.

g. Recorded State-Discharge. Recorded stage discharge hydrographs in the Santee River below the rediversion project have been prepared and are included to aid designers and other interested persons in determining pre-project conditions. Hydrographs included are those for the USGS gages at Lake Mattassee and Jamestown. Stage-discharge data for the years 1972 through 1975 for the Lake Mattassee gage (02171650-Santee River below St. Stephen) are presented on Plate B-9. Stage data for the Jamestown gage (02171700) are shown on Plate B-10 for the years 1974 and 1975. These gages have been in operation since 1967 and 1973, (see Table B-1). As can be seen, except for spill periods, the normal flow and stage at Lake Mattassee are about 500 cfs and 7.0 feet, respectively. Normal stages at Jamestown are influenced by tidal forces. As shown on Plate B-10, the normal range between the daily lows and highs varies from about one to two feet while the values themselves generally, fall between 1.0 and 3.5 feet.

23. Lake Moultrie-Minimum Elevation for Powerhouse Design Head. The minimum design level for Lake Moultrie was selected and approved in DM 3 (see reference 2c) where the background and rationale for its selection are fully discussed. A summary of this rationale and supporting arguments are presented in the following paragraph.

24. The Lake Moultrie entrance channel and the intake and tailrace canals are designed to deliver 24,500 cfs to St. Stephens' hydroelectric plant with a head of 49 feet at a Lake Moultrie elevation of 74.0 feet. This design elevation was taken as the second lowest pool elevation during the peak load month, August, for the period of record (1908-1972). Pool elevation frequency analyses indicated a pool elevation less than 74.0 feet has a two percent chance of occurrence in any given year or a recurrence interval of once in 50 years. A pool-elevation frequency curve for the month of August is shown on Plate B-11. The minimum lake elevation during the August period of record (73.0 feet) was not used due to the prohibitive cost of providing a channel which would meet design requirements. In the event the lake level falls below 74.0 feet in August it would be possible to use the existing Pinopolis plant to supplement St. Stephens' generation during these periods. For example, at elevation 73.0 it is possible to generate 80,800 kw at St. Stephens. The additional 3,200 kw of capacity

needed to meet dependable capacity for the month of August could be generated at Pinopolis with a slight increase in discharge. Lake levels less than 74.0 feet will be infrequent short-term occurrences, hence the supplemental use of the Pinopolis plant during these periods is considered more practical than a canal system designed for a lower lake level.

25. Interior Drainage.

a. Drainage Plan. Because the intake and tailrace canals will cut off local runoff from many areas south of the canal, a drainage plan to provide an outlet for this runoff is required. The recommended plan to provide drainage for these areas is shown on Plate B-12. Basically, the plan will divert runoff from a point about halfway between highways 35 and 45 to Halfway Swamp (Area I), provide a ditch generally located at the toe of the excavated material to collect runoff intercepted by the intake canal and discharge it into the tailrace canal just above the SCL Railroad (Area II) and provide a collector ditch to intercept runoff to the tail-race canal below the SCL Railroad and discharge into into Mattassee Lake (Area III). The plan provides for three drop structures to make the required transitions from one design level to another. Various plans considered and the reasons for changing the plan contained in the GDM is discussed in Appendix C, "Alternate Studies".

b. Derivation of Design Discharges. Because of comments concerning the use of the rational formula to derive design discharges for the GDM, for these studies a regional frequency analysis was used. For 25 stream gages located in the Coastal and lower Piedmont sections of the Charleston District and having drainage areas under about 200 square miles, statistical parameters (mean, standard deviation, and skew) were derived using the Hydrologic Engineering Center's Computer Program No. 723-X6-L2350, Regional Frequency Computations. The gages used in this study, their drainage areas and period of record are listed in Table B-6. Listed also in the table are the statistical parameters generated by the computer program. Using the data contained in this table, the standard deviations vs square root of the drainage area and the mean discharges (Log Q) vs drainage area were plotted. These are shown on Plate B-13. Gages located nearer geographically and whose contributing waterheads have characteristics more similar to the study area, are indicated both in the table and on the plate. More weight was given these stations, in selecting a curve for design purposes. The curve selected and one used in computing design discharges is shown as a dashed line on Plate B-13.

Table B-6

Statistical Data for Recorded and Reconstituted Flows

U.S.G.S. Station No.	Drainage Area (mi^2)	$\sqrt{DA}$	Period Of Record (yrs)	Equivalent Record (yrs)	Mean	Standard Deviation	Skew
1089.6*	15.3	3.91	19	22.2	2.652	0.280	-0.607
1096.4*	16	4.0	18	21.7	2.665	0.378	0.117
1100.2*	3.8	1.95	18	25.8	2.229	0.399	-0.150
1270	110	10.49	39	40.8	3.377	0.357	0.527
1273.9	0.9	0.95	19	34.5	2.190	0.481	-1.680
1282.6	15.4	3.92	18	30.8	3.053	0.261	1.184
1294.4	17	4.12	18	27.7	2.939	0.190	0.636
1305	64	8.0	19	38.4	2.678	0.259	1.749
1309	108	10.39	15	38.7	2.936	0.131	-0.763
1309.1	173	13.15	14	40.8	2.960	0.202	0.601
1306	55	7.42	4	38.9	2.821	0.135	0.294
1311.5*	28	5.29	8	36.4	2.586	0.398	-0.548
1322.3*	6.2	2.49	21	26.6	2.017	0.289	0.582
1335.9	4.7	2.17	19	34.5	1.840	0.288	-1.550
1339.6*	40.0	6.32	19	40.4	2.585	0.337	-2.152
1343.8*	16	4.0	21	24.6	2.336	0.198	-0.399
1353	70	8.37	6	33.4	2.861	0.250	-0.474
1483*	38.1	6.17	8	28.5	2.414	0.269	0.628
1695.5	136.0	11.66	15	27.4	2.938	0.121	0.542
1696.3*	10	3.16	8	33.1	2.214	0.406	0.817
1725	198	14.07	30	34.9	3.183	0.208	0.053
1742.5*	23.4	4.84	4	33.0	2.510	0.401	0.448
1765.0	203	14.25	24	30.0	3.226	0.250	0.124
1716.8*	17.4	4.17	8	28.0	2.362	0.398	-0.277
1973	87	9.33	8	33.0	2.525	0.096	0.843

*Stations that are located in Coastal Plain and whose contributing watersheds can be more closely compared to those of the study area.

c. Design Discharges. Using the drainage area determined for each sub-area shown on Plate B-12, values for the mean (M) and standard deviation (S) obtained from Plates B-13 and k values using a skew of zero, design discharges were computed using the following formula:

$$\text{Log } Q = M + kS$$

The drainage area of each sub-area, the values selected for the mean and standard deviation and the computed discharges for the 10, 25, and 50-year frequencies are shown in Table B-7. The design discharges for each channel segment of the interior drainage plan for the drop structures and for the culverts are discussed in the Hydraulic Design portion of this appendix.

TABLE B-7

HYDROLOGIC DATA FOR INTERIOR DRAINAGE PLAN

	RECOMMENDED PLAN								EXISTING DRAINAGE							
Location	Sub-Area No.	Drainage Area Square Miles Sub-Area	Drainage Area Square Miles Cum Area	Statistical Paramenters Mean	Statistical Paramenters Std Dev	Design Discharges 10yr	Design Discharges 25yr	Design Discharges 50yr	Sub-Area No.	Drainage Area Square Miles Sub-Area	Drainage Area Square Miles Cum Area	Statistical Paramenters Mean	Statistical Paramenters Std Dev	Design Discharges 10yr	Design Discharges 25yr	Design Discharges 50yr
AREA I																
St. Hwy 35, Culvert C-3	1	0.39	0.39	1.82	0.422	230	365	490	1*							
St. Hwy 204, Culvert C-2	2	0.62	1.01	1.96	0.398	295	455	600	2*							
	3	0.63	1.64	2.04	0.383	340	515	670	3	0.63	0.63	1.89	0.409	260	405	535
	4	0.19							4	0.19						
St. Hwy 35, Culvert C-1	5	0.34	2.17	2.08	0.378	365	555	720	5	0.34	1.16	1.98	0.392	305	465	610
	6	0.32							6	0.32						
End of Improvement	7	0.12	2.61	2.11	0.371	385	575	750	7	0.12	1.60	2.03	0.385	335	505	665
	8	0.29	2.90	2.13	0.370	400	600	780	8	0.29	1.89	2.06	0.381	355	535	695
	9	1.40	4.30	2.19	0.360	450	660	850	9	1.40	3.29	2.15	0.367	420	620	805
	10	1.29							10	1.29						
U.S. Hwy. 52	11	0.42	6.01	2.24	0.351	490	715	920	11	0.42	5.00	2.23	0.358	490	715	920
AREA II																
State Hwy 45, Culvert C-4	1	0.24	0.24													
	2	2.19	2.43	2.10	0.372	375	565	735								
U.S. Hwy 52, Culvert C-5	3	0.18	2.61	2.11	0.371	385	580	755								
Access Road, Culvert C-6	4	0.52	3.13	2.14	0.369	410	615	795								
Drop Structure D-1	-	-	3.15			410	615	795								
	5	1.02	4.17	2.19	0.360	455	675	870								
Drop Structure D-2	6	0.36	4.53	2.21	0.359	475	705	910								
AREA III																
Head of Collector Ditch, SCLRR	1	1.74	1.74	2.05	0.382	345	520	680								
	2	0.34														
	3	0.61	2.69	2.11	0.371	385	575	750								
	4	0.40	3.09	2.14	0.368	408	610	790								
	5	0.27														
	6	1.12	4.48	2.20	0.360	455	670	870								
	7	1.05	5.53	2.24	0.354	495	730	930								
	8	1.20														
Drop Structure D-3	-	-	6.73	2.27	0.350	525	770	980								

*Sub areas 1 and 2, under existing conditions drain to Santee River via Crawl Creek. Under recommended plan, those sub areas will drain to Cooper River via Halfway Swamp.

26. Intake and Tailrace Canal Design.

a. Basic Criteria for Design of Canals. The intake and tailrace canals are to be designed to convey to the powerhouse a flow of 24,500 cfs and to provide a net head at the powerhouse of 49 feet when Lake Moultrie is at an elevation of 74.0 feet. This criteria was set in paragraph 28 of approved DM 3 (See reference 2c).

b. Water Surface Profile Computations. Steady state water surface profiles were computed using the Hydrologic Engineering Center's HEC-2 computer program "Water Surface Profiles" and a backwater program developed in the Charleston District for use on a Monroe 1880 programmable desk calculator. The backwater program developed for the Monroe 1880 uses the standard backwater computational procedures, but was programmed to accept only trapezoidal sections thus reducing memory size. Since all the intake and tailrace canal designs considered have a trapezoidal section and no out of channel flow, this program can be used without an appreciable loss in accuracy and with much more efficiency. Water surface elevations computed using the 1880 were compared with those obtained from HEC-2 and found to be compatiable. The method of computation employed is similar to method 1 given in EM 1110-2-1409.

c. Optimum Studies for Canal Dimensions. Because there are numerous combinations of canal widths and depths that will satisfy basic design criteria, the curves shown on Plate B-14 were developed to facilitate selection of the optimum combination. Neglecting bridge losses, and using side slopes of 1 vertical on 3 horizontal any combination of tailrace and intake bottom widths shown on these curves will satisfy the basic design criteria. As indicated on the Plate, the curves are derived for invert elevations of 0.0 ft. and 3.5 ft. for the tailrace and 50 ft. and 54 ft. for the intake. Lower invert elevations for both intake and tailrace canals were investigated but were abandoned because of excessive rock excavation. Rock profiles are shown on Plates 19 and 20[2]. Also, raising the invert elevations above elevation 3.5 ft. for the tailrace and above elevation 54 ft. for the intake, resulted in excessive excavation. Canal bottom widths were found to be very sensitive to small changes in canal invert elevations. As can be seen from the

(2) Plates without an appendix prefix, i.e. B-12, C-1, etc. follow the main report section and before the appendices.

Plate, a tailrace canal invert of 0.0 ft. has a bottom width almost 100 ft. less than one with an invert elevation of 3.5 ft. This is due primarily to the very flat hydraulic gradient required. The total allowable head loss is only 3 ft. in approximately 10 miles.

The optimized dimensions for the intake and tailrace canals are: a bottom width of 285 ft. with 1 on 3 side slopes, an intake invert elevation of 500 ft. and a tailrace invert elevation of 0.0 ft. The optimization studies conducted and the rationale for selecting the recommended design are discussed in Appendix C, Alternate Studies.

d. Alternate Alignment Studies. Alternate alignments considered for both the intake and tailrace canals are also discussed in Appendix C. Plate C-1, Appendix C, shows a plan view of these alignments. In order to compare alignments, each was designed to meet the same basic design criteria specified in paragraph a, above. Several invert elevations were evaluated for each alignment. From purely hydraulic considerations, the principle difference between plans were the canal lengths. The canal bottom widths would vary depending upon the total channel length. Plans having longer channels would require slightly larger bottom widths. However, because of the extremely flat hydraulic gradients involved, the total spread in bottom widths was only about 20 feet. All plans considered used 1 vertical on 3 horizontal side slopes.

e. Bridge Pier Losses. Initial studies which dealt with optimum canal dimensions and various alignments did not include the effects of bridge pier losses. Because of the small allowable canal loss, 3 feet in about 10 miles, it was necessary to estimate these losses and to make allowances for them by increasing canal bottom widths. The method used in evaluting bridge pier losses is presented in Chapter 11, page 13-15 of "Hydraulic of Bridge Waterways" HDC No. 1, U. S. Department of Transportation, dated 1973. There will be one railroad bridge crossing the tailrace canal and possibly four highway bridges crossing the intake canal. At this time, none of these bridges have been designed. Thus, it was necessary to assume various pier shapes and configurations and evaluate their losses. Figure 7 of HDC No. 1 shows the pier shapes used in this analysis. During the course of evaluating the pier shapes, it became apparent that pier losses would be quite small, less than .05 feet, and that it would not be a severe constraint on the bridge designer to limit bridge pier losses to .03 feet per bridge. This would allow for a total bridge pier loss, if all possible bridges are built, of .15 feet. The intake and tailrace canals' bottom widths were increased sufficiently to compensate for this amount of loss.

f. Recommended Canal Dimensions and Alignment. The recommended alignment is shown on Plate 1. The recommended canal dimensions and a summary of pertinent hydraulic criteria used are shown in Table B-8.

TABLE B-8

CANAL DIMENSIONS AND HYDRAULIC DESIGN CRITERIA
INTAKE AND TAILRACE CANALS

Basic Criteria

Discharge - cfs	24,500
Net Head at Powerhouse - Ft.	49
Design Elevation - Lake Moultrie - Ft	74

Allowable Head Losses

Powerhouse - Ft	0.5
Bridges - Ft	0.15

Backwater Criteria

Manning "N" Value	0.025
Expansion Coefficient	0.3
Contraction Coefficient	0.1
Tailwater Elevation - Lake Mattassee - Ft	21.5

Canal Dimensions

Bottom Width - Ft	285
Side Slopes Von H	1 on 3

Invert Elevations

Intake Canal - Ft	50.0
Tailrace Canal - Ft	0.0

g. Computed Water Surface Profiles. Computed water surface profiles for the recommended intake and tailrace canals are shown on Plate B-15. Water surface elevations shown are for one, two and three units operating with a Lake Moultrie elevation of 74.0 feet and with three units operating and a Lake Moultrie elevation of 75.2 feet. For backwater computations, starting water surface elevations at Lake Mattassee were derived using the stage-discharge relationship shown on Plate B-7. Water surface elevations at each end and the head loss occurring for the intake and tailrace canals are shown in Table B-9.

TABLE B-9

SUMMARY OF COMPUTED WATER SURFACE ELEVATIONS

	No. of Units Operating		
Location	One (8,200 cfs)	Two (16,400 cfs)	Three (24,500 cfs)
Lake Mattassee - Elev. Ft. msl	16.1	19.8	21.5
Lake Moultrie Elevation - 74.0 Ft.			
Upstream of Powerhouse - Elev. Ft. msl	73.6	73.2	72.6
Drop (Moultrie to Powerhouse) - Ft.	0.4	0.8	1.4
Downstream of Powerhouse - Elev. Ft. msl	16.7	20.8	23.1
Drop (Powerhouse to Lake Mattassee) - Ft.	0.6	1.0	1.6
Lake Moultrie Elevation - 75.2 Ft.			
Upstream of Powerhouse - Elev. Ft. msl			74.1
Drop (Moultrie to Powerhouse) - Ft.			1.1
Downstream of Powerhouse - Elev. Ft. msl			23.1
Drop (Powerhouse to Lake Mattassee) - Ft.			1.6

27. Computer Program - Gradually Varied Unsteady Flow Profiles. The Hydrologic Engineering Center's computer program 723-G2-L2450 entitled "Gradually Varied Unsteady Flow Profiles" was used to simulate Santee River flows and to conduct surge studies in the Intake and Tailrace Canals. A brief description of this program is given in the following paragraph. A more detailed explanation of computational methods and techniques used in the program can be found in HEC's users manual.

28. The Unsteady Flow Computer Program employs an implicit finite difference solution to the one-dimensional equations of unsteady flow. The equations of continuity and momentum are solved using a numerical intergration scheme. The computation scheme requires that an odd number of evenly spaced nodes be specified. At each node or section the average section number, elevation, area, hydraulic radius to the two-thirds power and top width are required for each vertical line in the computation net. HEC has developed an auxiliary program that will compute these geometric properties for randomly spaced cross sections. The unsteady flow model is assumed to be one-dimensional in the sense that the flow characteristics such as depth and velocity are considered to vary only in the longitudinal direction and with time. The channel geometry is three-dimensional. The unsteady flow program requires that the upstream and downstream boundaries be specified. Boundary conditions may be a discharge hydrograph, stage hydrograph, or rating curve. Lateral inflow to interior points may also be specified as a discharge hydrograph. The computations simulate the response of the interior portions of the study reach to changes in depth or discharge at the end boundaries. The program output gives the discharge, water surface elevation and velocities at interior points for specified time intervals. This information can be printed out at all nodal points or for only those that the user selects. Computational stability depends upon nodal spacing, ΔX, and the computational interval Δt. Criteria for stability is presented in the program documentation once the ΔX has been selected a suitable Δt may be determined.

29. Santee River Model. In order to derive stage-discharge-frequency data for the Santee River within project limits, it was necessary to develop a river model that would simulate flows. Because of the extensive flood plain of the lower Santee River, 5 to 8 miles in width, it was felt that storage effects would play an important part in modifying peak flows and that the unsteady flow model would best simulate this effect. A description of the model, its calibration and use are discussed in the following paragraphs.

a. Model Description. The reach of the Santee River which was modeled is shown on Plate B-16. The study reach covers approximately 50 river miles from Jamestown, South Carolina to Wilson Dam. Hydrographic data coded into the auxiliary program, "Geometric Elements," included 34 cross sections. River cross sections were chosen so that they would be as nearly representative of the river reach as possible. Because of the extensive flood plain of the Santee River and the surveying costs required to obtain cross sections covering the flood plain from bank to bank it was necessary to supplement the hydrographic survey with topographic information from USGS quadrangle maps. The Bonneau, South Carolina quadrangle, having a contour interval of 20 feet and a scale of 1:62,500 and the Jamestown, South Carolina quadrangle, having a contour interval of 20 feet and a scale of 1:24,000 were used for this purpose. To check if there was agreement between the hydrographic survey and the topographic data obtained from the quadrangle maps, selected river cross sections were extended into the flood plain so that a portion of the Santee flood plain was surveyed. It was found that the quadrangle topographic data compared favorably with the surveyed data.

To reduce computation time and computer cost, a ΔX of one mile was used in this study. Using the stability criteria given in the program documentation and after several trial runs, a computational interval of 15 seconds was selected. The end boundaries used in the model were: Upstream boundary - the discharge hydrographs at Wilson Dam; downstream boundary - a stage-discharge rating curve at Jamestown. The Jamestown rating curve is shown on Plate B-17. After model calibration was achieved, a third boundary defining tailrace canal inflows to the Santee River, was incorporated into the model.

b. Model Calibration. Since a large number of runs were anticipated it was decided to calibrate the model for the base flow condition of 500 cfs and use this as the initial condition for all runs. The discharge of 500 cfs was selected because it is the minimum required release from Wilson Dam. It quickly became apparent that calibration for this small flow was not feasible because depth in the model became so shallow that instabilities resulted. The base flow was then increased to 1,000 cfs and the model was calibrated. Calibration was achieved for the 1,000 cfs flow by varying Manning's "n" value. The Unsteady Flow computer program allows the user to vary "n" value at each cross section and with depth of flow. By being able to vary "n" with flow depth, calibration is greatly facilitated. Computed water surface elevations for the 1,000 cfs steady state discharge were compared to the USGS rating curve for the Pineville and St. Stephen's gages. Manning's "n" was adjusted until suitable agreement was obtained.

Initial calibration runs for flood flow conditions on the Santee River were made for the March - April 1973 flood. This flood was chosen because there is ample high water mark and hydrograph information available for this flood. Results of the calibrated high water profile computed for the March - April flood is shown on Plate B-18. The high water data was obtained from crest stage indicators and recording gages at Pineville, St. Stephens and Jamestown. Additional high water data for this flood is shown on Plate B-19. This Plate shows the computed and recorded hydrographs at U.S. Highway 52, Lake Mattassee and Jamestown. To insure that suitable calibration had been achieved, several other floods were routed. The high water mark information available for each of these floods, along with the computed water surface profiles are also shown on Plate B-18. The upstream boundary for all the simulated flood routings was the recorded discharge hydrograph at the Pineville gage (02171500). As stated previously, Manning's "n" was varied at locations along the river as well as with depth of flow. The range of "n" values that were used varied from .021 in the channel to .17 in the overbanks.

30. Intake and Tailrace Canal Model. The unsteady flow computer program was used to investigate transient conditions in the intake and tailrace canals. In order to study surging in the intake and tailrace canals, the hydraulic elements of the recommended plan were coded into to model. Canal geometry, Manning's "n" value and channel lengths were the same as used in the HEC-2 computer program. The only difference is that the unsteady flow program requires a fix grid having an odd number of evenly spaced nodes. The nodal spacing, or ΔX's, used for both the intake and tailrace canals was 330 feet. The time interval, Δt, was set equal to 1 second. This small time interval was necessary because some tests had rapidly varying end boundary conditions.

31. Surge Studies - Intake and Tailrace Canals. Surge studies were conducted in order to investigate transient conditions in the intake and tailrace canals, to investigate the need for forebay storage, and the effects of rapid openings and closures at the powerhouse.

a. Intake Canal. For the intake canal studies, the upstream boundary was set for a constant Lake Moultrie elevation of 74.0 feet. The downstream boundary located at the powerhouse was driven by a discharge hydrograph simulating powerhouse operations. A rapid opening or rapid closure of the powerhouse gates are two conditions which will produce surges in the intake canal. The peak surge should occur in the intake canal when the discharge at the powerhouse is brought from zero discharge to a peak discharge of 24,500 cfs in a very short time interval or when the system is operating at full capacity, 24,500 cfs, and the discharge is reduced very rapidly. Both of these conditions are shown on Plate B-20. Shown on the Plate are water surface profiles at selected time intervals for each of these conditions (rapid opening and rapid closure). As can be seen from the profiles on the lower half of the Plate, the peak surge wave results from a rapid closure. The surge height for this condition is approximately 2.3 feet above the steady state condition. This wave would not create a hazard to small boats should any be in the intake canal at the time because of its long wave length, over 2,000 feet, and consequently its flatness. The surge wave celerity is approximately 16 feet/second traveling up the intake canal towards Lake Moultrie. The upper half of Plate B-20 shows the water surface profiles for the rapid opening condition. The maximum drawdown for this condition occurs approximately 20 minutes after the gates are opened. The magnitude of the drawdown is approximately 2.3 feet below the initial, no discharge, water surface elevation of 74.0 feet. This is 1.1 feet below the steady state water surface elevation predicted by the model. It should be noted that the magnitude of these surges are considered to be the maximum values that could occur because of the physical limitations of opening or closing all gates in a 5-second period. Also, the inertial forces of the turbines were not considered in this rapid opening analysis.

The maximum velocity to occur in the intake canal was 3.2 feet/second and occurred under the rapid opening condition. It was located just upstream of the powerhouse. Results from the rapid openings analysis indicates that additional forebay storage for the powerhouse is not required and that canal surging and drawdown will not be a problem in the intake canal.

b. Tailrace Canal. The discharge hydrograph at the powerhouse used as the downstream boundary in the intake canl studies was also used in the tailrace canal studies, except for the tailrace studies, it was the upstream boundary. For the downstream boundary, the Lake Mattassee stage-discharge rating curve shown on Plate B-7 was used. The upper half of Plate B-21 shows water surface profiles at selected time intervals caused by the same rapid opening condition (5 seconds) used in the intake canal studies. For this test, initial water level in the tailrace canal at Lake Mattassee was elevation 7.0 feet. This corresponds to a no-flow condition in the tailrace canal and a flow of 500 cfs in the Santee River. A more realistic 5-minute opening was also ran. The water surface profile for this condition is shown on the lower half of Plate B-21. From a surge standpoint in the tailrace canal, a rapid closure is not significant and therefore was not computed.

The rate of gate opening has a pronounced effect on maximum velocities in the tailrace canal immediately downstream from the powerhouse. To ascertain the range of values that might be expected, powerhouse discharges were increased from no flow to the design flow of 24,500 cfs in five second and five minute periods. Velocity profiles of the maximum velocities produced by these two openings are shown on Plate B-22. The dashed portion of the curve is for the 5-second opening while the solid curve is for the 5-minute opening. The two curves join at a velocity of about 4.8 feet per second and at a distance of about two miles below the powerhouse. Both tests were run with an initial water level in Lake Mattassee of 7.0 feet. With this low tailwater, maximum channel velocities in the tailrace canal can be assured for both rates of opening. The dashed curve shown on Plate B-22 should represent the maximum upper limit of channel velocities that could be experienced. The solid curve, because of the more reasonable five minute opening, was used for design of any necessary slope protection. As shown on Plate B-22, gate opening rates effect maximum velocities for approximately 1.5 miles downstream of the powerhouse. However, the maximum difference in velocities occurs in the first 800 feet. For distances greater than 800 feet, the differences are generally less than 0.7 feet/second. Maximum velocities generally occurred as the rising water passed through a flow depth of 11 feet. Velocities then decreased as water depths increased. This trend continued until steady state flow conditions were achieved. The velocity for this condition averages about 3.2 feet per second for the tailrace canal.

c. Study Results. For all tests that were conducted on the intake and tailrace canals, no apparent computational instabilities were encountered. Because of the rather extreme boundary conditions being tested, the test results were shown to staff members of the Hydrologic Engineering Center who are familiar with the use and limitations of the unsteady flow computer program. It was agreed that, considering the current state of the art concerning solutions to rapidly varied unsteady flow problems, that the methods used and the results obtained certainly looked reasonable and were probably adequate for design purposes.

32. Combined Tailrace and Santee River Model. Once calibration of the Santee River Model was achieved, the effects of the tailrace canal were incorporated into the model. This was done by treating tailrace canal flows as a local inflow. The local inflow was placed in the model between river miles 51 and 52 which in the prototype would be the approximate location of the tailrace canal. A limitation in the current version of the unsteady flow model is, that there is no way to describe the physics of a river junction. Local inflow has to be treated as a unit discharge over a given reach. However, considering intended use of study results this limitation for these studies can be disregarded. This model can provide a great deal of information regarding future effects of the St. Stephens hydropower plant on the Santee River hydraulic regime. The results of this study are discussed in the following paragraphs.

33. Water surface profiles in the Santee River for the combined models are shown on Plate B-23. The initial condition for all profiles shown, is the 1,000 cfs steady state base flow profile which is also shown on the Plate. The profiles shown are for time intervals of 6, 12, and 18 hours. They represent the instantaneous water surface elevations that would exist at these specific times, if the powerhouse releases were brought from no flow to 24,500 cfs in a 5-minute period and held at this rate indefinitely. Steady state profiles in the Santee River with tailrace canal flows of 24,500 cfs and 12,600 cfs are also shown on Plate B-23.

34. Stage hydrographs for the Santee River at Lake Mattassee and Jamestown for what is considered a typical week were computed and are shown on Plate B-24. The procedure used in developing these hydrographs was first to route powerhouse release shown on the Plate down the tailrace canal to Lake Mattassee. This was accomplished using the tailrace canal unsteady flow model. The computed flows at Lake Mattassee were then introduced as local inflow into the Santee River model. Powerhouse releases chosen, represent a 60 percent load factor for the Monday through Friday period. Powerhouse operation would be 15 hours per day at 24,500 cgs and 9 hours per day at zero flow. Saturday and Sunday releases were set at 16,300 cfs for 5 hours each day and zero flow for the remaining 19 hours. With this weekly schedule, the average daily flow is 12,500 cfs. This rate of flow was selected to be comparable with the long time average inflow into the lake system of 15,500 cfs per day. The remaining

3,000 cfs per day will be passed through the existing hydropower facilities at Pinopolis. Using this release schedule it can be seen that during the week, daily water level fluctuations at Lake Mattassee will be approximately 3 feet. The water level fluctuation at Jamestown, will be about 1.3 feet.

35. Tailwater Rating Curves - St. Stephen Plant. From backwater analysis, powerhouse tailwater rating curves were developed and are shown on Plate B-25. The curves give powerhouse tailwater elevations for various potentail flood flows in the Santee River above the confluence of the tailrace canal.

36. Riverside Levee. Project formulation as set forth in this DM calls for a tailrace canal riverside levee. The purpose of the levee is to increase power production at the St. Stephens Hydroplant and to prevent Santee River flood flows from encroaching laterally into the tailrace canal which could erode channel sideslopes and deposit sediment in the canal. Because of the cost of closing off the SCL Railroad, a levee higher than its embankment (elevation 46 feet) is not economical. Therefore, a levee that would experience overtopping during project life had to be designed. After investigating several alternatives, a sloping levee that would be progressively overtopped from the downstream end was selected. With this type design, the differential level across the levee just prior to overtopping would be minimal and flood flows across the levee should not cause a great deal of damage. The recommended levee has an elevation at the SCL Railroad of 45 feet and an elevation at its lower end of 35 feet. It would commence overtopping at the lower end at about the 30-year flood level. At the SCL Railroad, overtopping would occur at about the 50-year flood event. Above the SCL Railroad the levee would have a crest elevation of 45 feet which would also provide about 50-year protection to the tailrace canal. For flood flows greater than the 50-year discharge, water elevations in the tailrace would be approximately equal to those in the river.

37. Riprap Design. Riprap design was accomplished in accordance with EM 1110-2-1601 and ETL 1110-2-120. Riprap will be placed on side slopes where velocities are in excess of 6.0 ft/sec or where the presence of bends in the channel necessitates slope protection. The channel bottom was estimated to have an equivalent spherical diameter (effective roughness range) of .5 to .8 foot. The available sources near the project should produce riprap material having a specific weight of 160 pounds per cubic foot (SSD). Riprap gradation will be as described in referenced EM and ETL. Inclosure 1 of referenced ETL was used for riprap design. Design data for riprapped reaches in the tailrace canal, are shown in Appendix A, Geology and Soils. A typical riprapped channel section is shown for Station 370+00 on Plate 14. The recommended extent of riprap placement is shown on Plate 9.

38. Lake Moultrie Hurricane Surge Study.

a. General. In order to determine the height of protective levees along the intake canal, it was necessary to determine the maximum water level that could be expected in Lake Moultrie. Hydrologic studies indicate that if Lakes Marion and Moultrie were subjected to the Standard Project Flood, Wilson Dam has the capacity of holding the stage in Lake Moultrie, below elevation 75.6 msl. Because of the large spillway capacity at Wilson Dam, headwater or fluvial flooding would probably not induce the maximum water levels that could reasonably be expected to occur. However, wind induced superelevations in the lake level are possible and according to U.S.G.S. data published in "Water Resources Data for South Carolina" the maximum recorded lake level, 76.21 ft, was "affected by high wind." This elevation occurred in October 1959. Because of this, a wind set-up study was conducted for Lake Moultrie. The following paragraphs discuss selection of the design storm and other pertinent procedures used in the analysis.

b. Design Storm. The study area's close proximity to the Atlantic Ocean, approximately 30 miles, makes it succeptable to hurricanes and tropical storms. In order to determine the maximum water level that could reasonably be expected to occur, the Zone 2 Standard Project Hurricane (SPH) was used. Parameters for certain snythetic storms and methods for derivation of others have been furnished by the National Weather Service. The methods used in deriving the Zone 2 SPH are discussed in the National HURRICANE RESEARCH PROJECT Report No. 33 and MEMORANDUM HUR 7-120, entitled "Revised Standard Project Hurricane Criteria for the Atlantic and Gulf Coasts of the United States."

The SPH used for the study area was based on the revised SPH wind field patterns given in HUR 7-120. The National Weather Service's revised wind field data was derived from a study of 60 hurricanes that occured in the region over a period of 82 years. Other characteristics of the SPH were not changed. The SPH track critical to the entrance canal in Lake Moultrie is shown on Plate B-26 and the prior to landfall isovel pattern are shown on Plate B-27. Over land wind velocities were reduced in accordance with criteria given in HUR 7-120. Characteristics of the SPH and design parameters for the storm are shown in Table B-10.

Table B-10
Standard Project Hurricane
Characteristics and Design Parameters

Item	Value Used
Asymptotic Pressure - P_n (inches)	29.87
Central Pressure Index - P_o (inches)	27.48
Radius to Maximum Winds - R (Nautical miles)	30
Forward Speed - T (knots)	11
Maximum Over Water Windspeed - V_x (mph)	100

c. Wind Direction. Wind direction was determined in accordance with criteria given in NHRP No. 33. To facilitate wind direction calculations, an archimedial spiral was constructed so that its angles of incurvature were the same as specified in NHRP No. 33. The spiral was constructed to the same scale as the SPH isovel pattern. By using the same scale base map, the isovel pattern and spiral can be overlayed and the wind speed and direction determined at any point on the map.

d. Wind Set-up. The effect of strong winds blowing over shallow inclosed bodies of water, such as Lake Moultrie, is to drive large quantities of water ahead of the winds. It was necessary for the purpose of determining the height of protective levees along the intake canal to determine the maximum wind set-up in Lake Moultrie. The computation of wind set-up was based on the segmental integration method and was calculated by use of the step method formulas developed by Bretschneider. Bretschneider's formulas have been modified by others to facilitate calculations. The ones used in this study were modified by the New Orleans District and appear in Appendix A of "Interior Survey Report on Hurricane Study of the Lake Pontchatrain, Louisiana and Vicinity" dated November 1962. In the event that this report is not readily available, the equations used for set-up and set-down are as follows:

$$\text{Set-up} = d_t \left(\sqrt{\frac{0.00266\ U^2FN}{d_t^{\,2}} + 1} \quad -1 \right)$$

$$\text{Set-down} = d_t \left(1 - \sqrt{1 - \frac{0.00266\ U^2FN}{d_t^{\,2}}} \right)$$

Parameter definitions:

1. Set-up or set-down in feet measured above or below the initial lake level.
2. d_t = average depth of fetch in feet below initial lake level.
3. U = component of wind velocity in mph over fetch.
4. F = fetch length in miles.
5. N = planform factor, assumed equal to unity for this study.

The procedure used in performing the calculations was to construct on a base map various ranges across Lake Moultrie. The leaward end of the range was terminated at the point where the entrance canal joins the intake canal. Plate 28 shows the ranges used in the Lake Moultrie calculations. These ranges were divided into incremental one mile lengths and the average depth below the normal lake level determined for each of the one mile segments. The SPH isovel pattern and wind spiral were used to determine the wind speed and direction for each of the segments along the ranges. The design track was fabricated so that wind speeds and directions would be critical to the rediversion project; but, in order to determine the maximum set-up, each range had to be evaluated so that the critical combination of depths, wind speed and direction could be found. Set-up computations were performed for several hours before and after proximity time in order to find this critical combination. The method requires that the user guess a nodal point about which the lake water level is assumed to pivot. Set-up is computed on the leaward side of the nodal point and set-down is computed on the windward side. The computations are done in a stepwise fashion for each incremental distance along the range. After the incremental set-up and set-down calculations have been performed, a volume check is made. If the volumes of set-up and set-down balance within 1% accuracy then the correct nodal point was selected. If the volume check does not balance to the desired accuracy, a new nodal point must be estimated and the incremental set-ups and set-downs recalculated until a suitable volume check is obtained. Because of the tedious nature of these calculations, a computer program developed in the New Orleans District for a GE 400 series computer was used in determining the maximum wind set-up. The maximum elevation caused by wind set-up for the Zone 2 SPH was 82.0 ft.

e. Wave Set-up. Estimates of wave set-up were made using criteria and procedures recommended in Volume 1 of the U. S. Army Coastal Engineering Research Center's "Shore Protection Manual." Wave set-up is defined as that superelevation of the mean water level caused by wave action. Thus in order to determine the maximum still water level at the intake canal, it was necessary to add the wave set-ups to the water level caused by wind set-up. In order to compute the wave set-up it is first necessary to calculate the SPH deep water wave characteristics. Wave characteristics were calculated for the winds and depths associated with the maximum wind set-up. Figures 3-23 and 3-24 of the Shore Protection Manual were used to calculate the shallow water waves. Table C-1 of Volume 3 of the Shore Protection Manual was used to obtain the shoaling coefficient H/H_O'. The net wave set-up computed for the critical hour of the SPH was found to be about 0.5 ft. This incremental rise in the still water level (SWL) was added to the maximum computed SPH wind set-up to obtain the total still water level of 82.5 ft msl. Table B-11 gives the pertiment values of parameters used in computing the wave set-up.

Table B-11
SPH Deep Water Wave Characteristics
Lake Moultrie Vicinity of Intake Canal

Item	Value Used
SPH Zone	2
Windspeed - u (mph)	70.5
Fetch Length - L (miles)	5
Windtide Level - SWL (ft msl)	82
Average Depth of Fetch - d (feet)	16
Wave Height - H (feet)	4.7
Period (time) - T (seconds)	3.6
Deep Water	
Wave Length - Lo (feet)	66
Wave Height - Ho' (feet)	5.06
Breaking Depth - d_b (feet)	4.64
Breaking Wave Height - H_b (feet)	3.62

f. Wave Run-up. Wave run-up calculations were performed in order to determine the crest elevations of the intake canal levees. The still water level was assumed to be the maximum still water level produced by the SPH. The deep water significant wave, given in Table B-11, was tested for run-up using the composite slope method presented by Saville. Run-up was computed using Figure 7-10 of the Shore Protection Manual, Volume 2. Figure 7-10 gives relative run-up, $R/_{Ho'}$, for a $ds/_{Ho'}$ value approximately equal to 0.8. Run-up produced by the significant wave was very small, less than 1 foot, because of the extensive shallow water depths fronting the levee. This causes the wave to break before it reaches the 1 on 3 sloping portion of the levee. Plate 13 shows typical cross sections of the intake canal levee near Lake Moultrie. Other smaller waves in the wave spectrum were also tested for run-up. Maximum run-up was produced by a wave breaking at the toe of the 1 on 3 levee slope. The elevation of the levee toe is 78.0 feet. This wave would run-up approximately 4 feet or to elevation 86.5 (total SWL = 82.5 feet). Run-up calculations assume that the structure is subjected to direct wave attack. In the case of the intake canal levees, the land area fronting the structure provides a high degree of natural protection because of the presence of trees and underbrush. It is expected that a large amount of the incident wave energy would be dissipated by the trees and underbrush and that actual wave run-up would be much less than the calculated values. With this natural barrier, overtopping of the levee is highly unlikely and an additional increment for freeboard is not considered warranted; therefore the crest elevation of the intake canal levees was set at elevation 86.0 ft. msl. The design crest elevation of the Lake Moultrie levee was 85.0 feet; however, survey data in the vicinity of the project, indicates that current crest elevations vary between 85 and 86 feet.

39. Slope Protection for SPH. The intake canal levees, in the vicinity of the existing Lake Moultrie levee, will have a large degree of natural protection from wave attack because they will be located over a 1,000 feet from the normal shore line. As previously discussed, this land area, between the existing shore line and the Lake Moultrie Dyke, is heavily vegetated with trees and shrubs. This vegetation would absorb most of the incident wave energy during the SPH. Because of this and the small probability of occurrence of the SPH, slope protection is not considered necessary for the intake canal levees at Lake Moultrie.

40. Interior Drainage.

a. General. The proposed interior drainage plan is shown on Plate B-12. Interior for this report is defined as those areas south of the intake and tailrace canals whose normal drainage to the Santee River has been blocked by the rediversion project. The proposed drainage system for this interior area consists of three separate canals or ditches, each of which drains a portion of the three separate drainage areas, three drop structures and culverts at up to six locations. The number of locations where culverts will be required will depend upon the number of highways that will ultimately cross the intake canal. The rational for selecting the proposed plan and other drainage plans considered are discussed in Appendix C, "Alternate Studies." The following paragraphs discuss various features of the selected plan and the hydraulic criteria used in their design.

b. Backwater Computations. Backwater computations were performed to derive water surface profiles for all interior drainage ditches using the same techniques and procedures as discussed in paragraph 26b. Water surface profiles were computed for floods having recurrence intervals of 10, 25, 50 and 100 years. An "n" value of .035 was selected for the design of all drainage ditches. All though the initial "n" value of these ditches will probably be less, the "n" value will most likely increase to the selected value due to vegetation of the channel side slopes. Vegetation is expected as a result of the infrequent occurrence of significant flows. Where existing condition profiles were computed, Manning's "n" values were selected using the Geological Surveys Water Supply Paper No. 1849 entitled "Roughness Characteristics of Natural Channels," from field observations and from past experience with streams of the same type. Minor loss coefficients included contraction, expansion, and culvert losses. The contraction and expansion losses used were .1 and .3, respectively.

c. Culverts. All culverts required for the proposed interior drainage plan were designed in accordance with criteria presented in the U. S. Department of Transportation's, Hydraulic Engineering Circular No. 13 dated August 1972 and entitled "Hydraulic Design of

Improved Inlets for Culverts." All culverts except two were designed to pass the 50-year frequency flood. These two, because of low road elevations, were designed to pass the 10-year frequency flood. For design conditions, all culverts will operate under outlet control. Because a beveled edge inlet was selected in designing the culverts, an entrance loss coefficient, K_e, of 0.2 was used.

d. Drop structures. Drop structures were used in the interior drainage plan where needed to prevent channel erosion and for channel grade control. Where a structure was required, the U. S. Bureau of Reclamation's Type IX Basin baffled chute drop structure was used. The baffled chute drop structure was selected as the type structure to use for the interior drainage plan because of the vertical drop heights, the varying tailwater conditions, and its' performance through a wide range of flows. With this type structure, a baffled chute is used to dissipate the energy contained in the drop. The multiple rows of baffle piers contained on the chute prevent excessive flow acceleration and exit velocities regardless of drop height. Since lower unit discharges result in lower exit velocities, the baffled chute is also effective for flows less than the design condition. The structure also requires no initial tailwater to be effective, yet, it's effectiveness as an energy dissipator is not impared by rising tailwater. In designing the baffled chute drop structures, criteria contained in "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, United States Department of Interior, Bureau of Reclamation, dated March 1974, was used.

The design flow condition for all drop structures was the 50-year frequency flood. Channel improvements in the vicinity of the structures are designed to pass the 25-year frequency flood but are also adequate to pass the 50-year flood as well. Approach sections to each structure will be of reinforced concrete and rectangular in shape. They are sized to have velocities in the approach section approximately equal to the recommended velocity of 5 fps less than the critical velocity computed at the structures crest. To improve inlet conditions, wing walls and a two stage transition from the trapezoidal to a rectangular section is provided. The two stage transition is from 1 on 3 (vertical on horizontal) to 1 and 2 and 1 on 2 to the rectangular section. The approach inlet for each structure will have an invert elevation of 1 foot below the crest elevation and side wall heights to at least the 100-year flood level. The crests will be formed by a 1 foot radius curve and the upstream invert elevation of the first row of baffles on the chute will be within 1 foot vertically of the crest elevation. The baffled chute for all structures will be placed on a 1 vertical on 2 horizontal slope. Baffle piers will be constructed normal to the chute slope and will have heights approximately 0.8 times critical flow depth measured at the crest. Baffle piers and the spaces between them, will be equal and will be sized to have widths that are between 1.0 and 1.5 times the baffle pier height. Widths of partial baffle piers and spaces adjacent to the training walls will not be less than 1/2 or greater than 2/3 the baffle pier height. Distance between rows of baffles will be set at 2.0 times the baffle height with baffle piers and spaces alternating between rows. Chute training wall heights will be set at 3.0 times the baffle pier height. Square wing walls will be placed at the end of the chute at the same elevation as the training walls and will tie back, into the outlet channel side slopes. Riprap protection provided at each structure will be designed in accordance with criteria presented in inclose 1 of ETL 1110-3-120, dated 14 May 1971. All channel transitions will be designed in accordance with tranquil flow criteria given in EM 1110-2-1601.

e. Drainage Design for Area 1.

(1) Drainage plan. The boundaries for Drainage Area 1, an area of about 6 square miles, is shown on Plate B-12. Under existing conditions, drainage for sub-areas 1 and 2 (Area 1), is via a ditch, constructed by the Georgia Pacific Company (located in sub-area 2), across the proposed intake canal alignment to Crawl Creek and thence to the Santee River. Under the proposed plan, runoff from these sub-areas (1 and 2), which have a combined drainage area of about one square mile, will be diverted to Halfway Swamp and thence to the Cooper River. Improvements planned for Halfway Swamp have been designed to receive this additional runoff without increasing flood levels. A plan view of the proposed drainage plan is shown on Plates B-29, B-30 and B-31.

(2) Drainage plan design. Shown also on Plates B-29, B-30 and B-31 are design details of the proposed plan. Shown on these plates are invert profiles of the improved and existing channel, profiles of either the low bank or natural ground line, typical channel cross sections of existing and improved conditions, recommended bottom widths and limits of channel improvement. In addition to the plates, recommended bottom widths at selected locations for the improved channel are also presented in Table B-12. As shown, channel enlargement for Halfway Swamp begins at centerline station 119+00 with clearing and grubbing recommended between stations 108+00 and 119+00. Station 108+00 is a little less than one-half a mile below State Highway 35. Above station 249+00, the proposed drainage ditch will provide an outlet for the 10-year flood with little or only minor out of bank flow. Below this point, due to low relief of adjoining lands, this degree of flood protection was not considered justified. At this point, station 249+00, the improved channel invert was selected so that drainage would be provided for the lowest point in the existing Georgia Pacific drainage ditch.

Bottom widths of the improved channel were selected to insure that flood levels for the recommended drainage paln would always be less than those for existing conditions. Improvements in Halfway Swamp terminate where a definite increase in channel slope occurs and where no increase in flood levels due to the proposed drainage plan can be assured. The proposed drainage ditch will cross three State roads. These are: Highway 35 at station 128+00, 204 at Station 220+85 and 35 at station 294+32. The selected sizes and number of culverts at each of these road crossings are shown in Table B-16.

(3) Drainage plan performance. To test performance of the proposed drainage plan, water surface profiles for the 10, 25, and 50-year floods were computed. All computed profiles confirmed that flood levels for improved conditions were always equal or less than existing conditions. Profiles of the 25-year flood for both existing and improved conditions are shown on Plates B-29, B-30 and B-31. In addition, the 25-year flood design discharges, average channel velocities and computed water surface elevations are presented in Table B-12. Backwater computations for these profiles were started immediately upstream of U. S. Highway 52 which is over two miles below the lower limit of the proposed improvement. Starting elevations were computed by the slope area method, assuming a frictional slope of .0018 for all discharges. This frictional slope is approximately equal to the slope of the channel invert. With the starting point being over two miles below the beginning of improvement, this distance is more than sufficient to assure that channel control has been established and that water surface elevations in the reaches of interest are unaffected by any error associated with starting conditions. Selected values of Manning's "n" for the channel and overbank areas for existing conditions ranged from .05 to .08 and .08 to 1, respectively. No highwater mark information for Halfway Swamp was available to confirm these values. An "n" value of 0.035 was used for the improved channel.

f. Drainage design for Area II.

(1) Drainage plan. Drainage Area II, which is shown on Plate B-12, encompasses an area of about 4.5 square miles. The objectives of the drainage plan for this area are to provide an outlet for those existing drainage channels that drain to the Santee River but will be cut-off by the Rediversion Project and to provide drainage for the excavated material disposal area south of the Intake Canal. Plan views of the drainage ditch provided to meet these objectives are shown on Plates 7, 8 and 9. Basically, the main features of the drainage plan consists of a drainage ditch about 3.4 miles in length, culverts at possibly three road locations and two drop structures, one (D-1) located near the Powerhouse where flows are dropped into an existing Creek and one (D-2) located just upstream of the SCL Railroad bridge where flows are dropped into the

TABLE B-12

PERTINENT HYDRAULIC DESIGN DATA

INTERIOR DRAINAGE DITCH - AREA I

Center Line Station (feet)	Hydraulic Elements Bottom Width (feet)	Hydraulic Elements Side Slopes (hor/ver)	25-Year Recurrence Flood Discharge (cfs)	25-Year Recurrence Flood Average Channel Velocity (ft/sec)	25-Year Recurrence Flood Water Surface Elevation (ft - msl)
0+50	Natural	Natural	715	2.0	51.2
17+20	Natural	Natural	715	1.3	53.7
29+40	Natural	Natural	715	2.1	55.5
49+70	Natural	Natural	660	1.3	59.5
66+10	Natural	Natural	600	1.0	61.3
81+90	Natural	Natural	600	2.5	66.0
94+80	Natural	Natural	600	2.6	69.7
108+00	Natural	Natural	575	1.0	71.7
122+90	10	2	575	1.0	72.0
127+00	10	2	555	1.5	72.2
127+75	18	2	555	4.3	72.3
128+00	SC Hwy 35		3 - 5 X 4.5	Box Culverts	
128+25	18	2	555	3.4	73.2
128+75	10	2	555	1.5	73.2
142+00	10	2	555	1.4	73.3
156+07	10	2	515	1.5	73.5
178+20	10	2	515	2.1	73.8
201+60	10	2	515	1.3	74.1
220+35	10	2	455	1.0	74.4
220+60	10	2	455	1.7	74.4
220+85	C/L Hwy 204		2 - 66"	Ø Culverts	
221+10	10	2	455	1.7	75.4
221+35	10	2	455	1.6	75.4
252+60	10	2	365	1.4	75.7
268+00	10	2	365	2.2	75.8
276+00	10	2	365	2.2	76.1
288+00	10	2	365	2.2	76.6
293+75	10	2	365	2.3	76.8
294+00	10	2	365	2.4	76.8
294+32	C/L SC Hwy 35		2 - 60"	Circular Culverts	
294+65	10	2	365	1.7	78.5
294+90	10	2	365	1.7	78.5
300+00	10	2	350	1.9	78.6
305+50	10	2	350	2.1	78.8
306+00	5	2	175	1.4	78.9
312+00	5	2	150	1.5	79.0
324+00	5	2	125	2.0	79.3
325+60	5	2	100	2.1	79.4

tailrace canal. This two drop structure plan was found to be more economical than the plan utilizing one large drop structure with flows dropped directly into the tailrace canal just below the Powerhouse. This was the plan contained in the GDM. A further discussion of the alternatives considered can be found in Appendix C.

(2) Drainage plan design. Hydraulic design criteria and design details of the proposed drainage plan and appurtenances are shown on Plate B-32 and in Tables B-13, B-15 and B-16. The drainage ditch recommended has bottom widths varying from 5 to 20 feet and is generally designed to pass the 25-year frequency flood in bank; however, there are a few low areas that the ditch traverses that could experience some minor out of bank flow. Canal bottom widths at selected centerline stations are presented in Table B-13 and are shown on Plate B-32. Shown also on Plate B-32, are profiles of the natural ground line and proposed ditch invert. The drop structures contained in the drainage plan were required to maintain channel grades and to prevent erodiable velocities. Hydraulic design criteria developed for these structures are presented in Table B-15. Plan views of the drop structures are shown on Plate 16. Because of problems associated with structural design, 45 degree wing walls were not provided at the entrance of drop structure D-1. Typical structural details for drop structure D-1 are shown on Plate 17. Both drop structures, D-1 and D-2, are designed to pass the 50-year frequency flood. Culverts will be required where roads cross the interior drainage ditch and Intake Canal. The culverts that would be required if the roads remained at their present locations are shown in Table B-16. At this time, the number of roads that will cross the Intake Canal and their locations are not known. Final culvert designs for these roads will be accomplished in the roads relocation DM. Typical cross sections of the drainage ditch above drop structure D-1, are shown on Plates 13 and 14. Typical cross sections for the reach between structures D-1 and D-2 are shown on Plate B-33.

(3) Design tailwater conditions. In selecting the hydraulic design criteria for drop structure D-2, water surface elvations in the tailrace canal are of prime importance. From a critical design standpoint, a low tailrace canal elevation coupled with the 50-year design flood runoff would produce the highest velocities in the lower reaches of the interior drainage ditch and the largest drop in water surface at the control structure. The minimum elevation in the tailrace canal would be when the powerhouse is not operating and the minimum flow of 500 cfs is being released at Wilson Dam. With these conditions (no power and 500 cfs in Santee River) and the peak rates of runoff from interior drainage Area II and III for the 50-year design flood, the elevation of the Santee River at the Lake Mattassee gage is 9.5 feet. This gauge (02171650) is located just below the confluence of the tailrace canal and Santee River. A discharge rating curve for the Lake Mattassee gage is shown on Plate B-7. Other tailwater elevations for various frequency floods in the Santee River and with maximum power generation are presented in Table B-15.

(4) Drainage plan performance. Performance of the proposed drainage plan was tested by computing water surface profiles for various flood frequencies up to and including the 100-year flood. The flood profile for the 25-year flood is shown on Plate B-32. The design discharges, average channel velocity and computed water surface elevations, also for the 25-year flood, at selected centerline stations are shown in Table B-13. The starting elevation for these design computations were derived by computing the water surface profile in the tailrace canal starting with and elevation of 9.5 feet at the Mattassee Lake gage. The elevation occurring at the confluence of the interior drainage ditch and the tailrace canal was used as the starting elevation for backwater computations up the interior drainage ditch. Backwater computations were continued up to the downstream end of each drop structure. It was necessary to restart computations on the upstream side of each structure, since a discharge rating curve could not be computed. Because of the turbulent flow conditions created by the row of baffles near the structures crest, model test data is needed to rate this type structure. At the present time, this type structure has not been model tested sufficiently to determine the emperical relationships needed to compute a discharge rating curve. Therefore, backwater computation above each structure were initiated by assuming critical depth at the crest of the structure. Channel sections just upstream of the crest were spaced very close together in order to more accurately determine the point at which channel control commences. This point usually occurred within about three feet of the crest. In addition to the above tailwater condition, flood profiles with various tailwater conditions were computed to investigate performance through a wide range of conditions. Headwater elevations at each structure for a few of these conditions are presented in Table B-15.

g. Drainage design for Area III.

(1) Drainage plan. The Area III watershed contains an area of about 6.7 square miles. Its boundaries are also shown on Plate B-12. The planned drainage objectives for this area are to intercept existing runoff imminating from a culvert that passes under the SCL Railroad and from existing drainage channels that currently drain to the Santee River via Lake Mattassee, provide drainage for the excavated material disposal areas on the south side of the Tailrace Canal and to convey this runoff to the Santee River below project appurtenances. To accomplish these objectives a drainage ditch of about 3.4 miles and one drop structure is required. Plan views of this drainage ditch and associated appurtenances are shown on Plates 9, 10, 11 and 12.

(2) Drainage design plan. Hydraulic design criteria and plan details are shown on Plate B-32 and Tables B-14 and B-15. The upper end of the drainage ditch starts below an existing 8' x 8' arched

TABLE B-13

PERTINENT HYDRAULIC DESIGN DATA

INTERIOR DRAINAGE DITCH - AREA II

Center Line Station (feet)	Hydraulic Elements Bottom Width (feet)	Hydraulic Elements Side Slopes (hor/ver)	25-Year Recurrence Flood Discharge (cfs)	25-Year Recurrence Flood Average Channel Velocity (ft/sec)	25-Year Recurrence Flood Water Surface Elevation (ft - msl)
0+00	20	3	705	1.1	9.5
2+08	20	3	705	5.1	10.6
2+78	20	3	705	5.0	28.8
DROP STRUCTURE D-2					
3+01	20	3	705	3.4	29.4
6+18	20	3	705	3.0	29.6
8+68	20	3	705	3.0	29.9
8+78	20	2	705	3.5	29.9
12+68	20	2	705	5.8	30.4
26+18	20	2	675	5.8	36.5
34+23	20	2	675	5.9	40.1
44+68	20	2	675	6.1	45.1
47+78	20	2	615	6.1	46.6
48+19	20	3	615	4.7	47.0
DROP STRUCTURE D-1					
49+28	15	3	615	2.4	65.1
51+88	15	3	615	2.8	65.2
55+88	15	3	615	3.4	65.6
56+28	C/L Power House Access Rd.		2-6' X 7' Concrete Box Culverts		
56+55	15	2	615	3.2	66.8
63+39	15	2	610	4.8	67.6
70+64	15	2	600	5.4	69.7
75+64	15	2	595	5.5	71.5
86+64	15	2	590	3.2	73.61
90+64	15	2	590	3.3	74.0
94+89	15	2	585	3.4	74.4
96+14	15	2	580	4.0	74.6
96+76	C/L U.S. 52		2-6' X 6' Concrete Box Culverts		
97+38	15	2	580	3.0	75.8
113+12	15	2	570	2.6	76.8
135+12	15	2	560	2.4	77.8
135+87	5	2	75	.5	77.9
143+12	5	2	75	.5	77.9
146+62	5	2	75	.5	77.9
146+98	C/L S.C. Hwy 45		48" Circular Culvert		
147+34	5	2	75	.4	78.7
150+92	5	2	75	.4	78.7
163+12	5	2	60	.8	78.8
173+12	5	2	50	2.7	79.0
182+12	5	2	50	2.8	81.6

box culvert which passes under the SCL Railroad at tailrace station 418+93. The drainage ditch recommended has bottom widths that vary from 15 to 20 feet and is designed to pass the 25-year frequency flood within the proposed trapezoidal cut. Invert and natural ground line profiles are shown on Plate B-32 as well as other design details. Canal dimensions at selected centerline stations are presented in Table B-14. Typical cross sections of the proposed drainage ditch are shown on Plates 14 and 15. One drop structure (D-3), is required to control velocities and to lower flows about 16 feet. This structure is also designed to pass the 50-year frequency flood. Hydraulic design criteria as well as other design details for drop structure D-3 are given in Table B-15. A plan view of this structure is shown on Plate 16.

(3) Drainage plan performance. To ascertain the performance of the proposed plan, a series of water surface profiles for floods up to the 100-year flood and for varying tailwater conditions were computed in the same manner as described in paragraph 40f (3). Design discharges used, average channel velocities and computed water surface elevations for the 25-year flood at selected centerline stations are presented in Table B-14. A water surface profile for this same flood is shown on Plate B-32.

TABLE B-14

PERTINENT HYDRAULIC DESIGN DATA
INTERIOR DRAINAGE DITCH - AREA III

Center Line Station (feet)	Hydraulic Elements: Bottom Width (feet)	Hydraulic Elements: Side Slopes (hor/ver)	25-Year Recurrence Flood: Discharge (cfs)	25-Year Recurrence Flood: Average Channel Velocity (ft/sec)	25-Year Recurrence Flood: Water Surface Elevation (ft - msl)
0+00	20	3	770	3.5	9.5
2+00	20	3	770	4.8	10.7
5+00	20	3	770	3.9	11.5
DROP STRUCTURE D-3					
6+00	20	0	770	6.3	27.1
6+40	20	2	770	3.4	27.7
7+19	20	2	770	3.4	27.7
7+44	15	3	770	4.1	27.8
10+00	15	3	770	4.0	27.9
20+00	15	3	770	4.0	27.9
32+00	15	3	730	3.8	27.9
56+00	15	3	730	3.7	27.9
57+00	15	3	670	3.4	28.0
81+00	15	3	670	3.4	28.0
100+00	15	3	610	3.0	28.0
116+00	15	3	520	2.6	28.1
165+00	15	3	520	2.6	28.1
178+00	15	3	520	2.6	28.2

Table B-15

PERTINENT HYDRAULIC DESIGN DATA

DROP STRUCTURES - DRAINAGE AREAS II and III

	Structure No.		
	D-1	D-2	D-3
Type Structure	Baffled chute	Baffled chute	Baffled chute
Location (Sta. No.)			
Drainage Area - MI^2	3.15	4.53	6.73
Design Discharge[(1)] - cfs (Q)	795	910	980
Headwater Elevation - ft msl			
No Power[(2)], no flooding[(2)]	65.3	29.7	28.0
Max. Power, no flooding		29.7	28.0
Max. Power, 10-year flood		32.6	31.4
Max. Power, 25-year flood		36.1	35.1
Max. Power, 50-year flood		44.2	38.8
Tailwater Elevation - ft msl			
No Power, no flooding	47.5	10.8	12.2
Max. Power, no flooding		23.0	21.5
Max. Power, 10-year flood		32.6	31.3
Max. Power, 25-year flood		36.1	35.1
Max. Power, 50-year flood		44.2	38.8
Embankment			
Top Elevation - ft	69.0	36.5	35.0
Side Slopes (V on H)	1 on 2	1 on 2	1 on 2
Crest			
Elevation - ft msl	59.0	24.0	22.0
Shape	Circular	Circular	Circular
Width - ft (W)	15	20	20
q - Q/W - cfs/ft	53.0	45.5	49.0
Approach Apron			
Invert El. - ft	58.0	23.0	21.0
Width - ft	15	20	20
Length - ft	18	23	24
Side Wall El. (Min Design El.)-ft msl	69.5	31.0	29.5
Proposed Constructed El. - ft msl	69.5	36.5	35.0

Table B-15 Continued

	Structure No.		
	D-1	D-2	D-3
<u>Chute</u>			
Drop Depth(3) - ft	16	18	16
Width - ft	15	20	20
Length - ft (along slope)	49.2	51.4	47.0
Slope - V on H	1 on 2	1 on 2	1 on 2
Side Wall Height - ft	10.5	10.0	10.0
<u>Baffles</u>			
Height - ft	3.5	3.25	3.33
Width - ft	4.0	4.75	4.75
Lateral Spacing - ft	4.0	4.75	4.75
Row Spacing - ft	7.0	6.50	6.66
Number of Rows	7	8	7
<u>Inlet Channel</u>			
Bottom Width - ft	15.0	20.0	20.0
Invert El. - ft msl	58.0	23.0	21.0
Side Slopes (V on H)	1 on 3	1 on 3	1 on 3
<u>Outlet Channel</u>			
Bottom Width - ft	20	20	20
Invert El. - ft msl	43.0	6.0	6.0
Side Slopes (V on H)	1 on 3	1 on 3	1 on 3
<u>Riprap Protection</u>			
Side Slopes (ETL 1110-2-120, Incl 1)	12 inch	12 inch	12 inch
Channel Bottom (ETL 1110-2-120, Incl 3)	18 inch	18 inch	18 inch

(1) 50-year discharge
(2) Discharge at powerhouse and Santee River flooding
(3) Vertical drop at crest el. to outlet channel invert

41. Exterior drainage.

a. General. A general plan view of the proposed exterior drainage plan is shown on Plate B-12. Exterior for this report is defined as those areas north of the Intake and Tailrace Canals that drain to the Santee River and whose drainage is generally uneffected by the Rediversion Project. Exterior drainage plans recommended for the Intake and Tailrace Canal areas are discussed separately in the following paragraphs.

b. Intake canal. Most of the natural drainage north of the Intake Canal drains away from the project boundary. Exterior drainage plans for this area consists primarily of providing an outlet for runoff from the various excavated material disposal areas. This is accomplished by providing a small collector ditch located at the toe of each area. This ditch will collect runoff from the excavated material disposal area and convey it to an existing natural drain. Plates 6, 7 and 8 show the proposed location of these ditches and the direction of flows. Low areas, where ponding might occur due to poor drainage, will be filled with excavated material from the Intake Canal. The proposed collector ditch will have a five foot bottom width, a minimum depth of cut of four feet and side slopes of 1 vertical on 2 horizontal. Culverts at two raod crossings will be required. These are C-7 at State Road 45 and C-8 at State Road 293. Hydraulic data for these culverts are contained in Table B-16. The design for culvert C-7 is preliminary since the ultimate location of this road is not known at this time.

c. Tailrace canal. The area north of the Tailrace Canal is drained by a series of channels in the Santee River flood plain, the primary channel being Mattassee Run. The riverside levee that parallels the Tailrace Canal cuts off the existing Mattassee Run channel at several locations. Therefore, the exterior drainage plans for this area are to provide a drainage channel along the toe of the riverside levee at these cut off locations in order to provide a continuous channel for Mattassee Run. At these cutoffs, the invert elevation of the proposed ditch will be constructed to match the existing channel inverts. The total length of all these ditches at the cut off locations is about 1,320 feet. In addition, approximately 3,250 feet of channelization will be required at the lower end of the project where the alignment of the tailrace canal and Mattassee Run coincide. This channel will start where the existing Mattassee Run channel is cut off, at Tailrace Canal Station 568+30, and run along the toe of the riverside levee until it confluences with the Tailrce Canal, at Tailrace Canal Station 596+80. The proposed ditch will have a bottom width of 20 feet and side slopes of 1 vertical on 3 horizontal. Between Tailrace Canal Stations 568+30 and 592+80, the ditch will have an invert elevation of 8 feet. The remaining 800 feet (Tailrace Canal Stations 592+80 to 596+80) will slope from elevation 8 to elevation 4. A detailed plan view of these drainage ditches are shown on Plates 11 and 12. Additional survey information covering this area will be necessary before plans and specifications can be prepared.

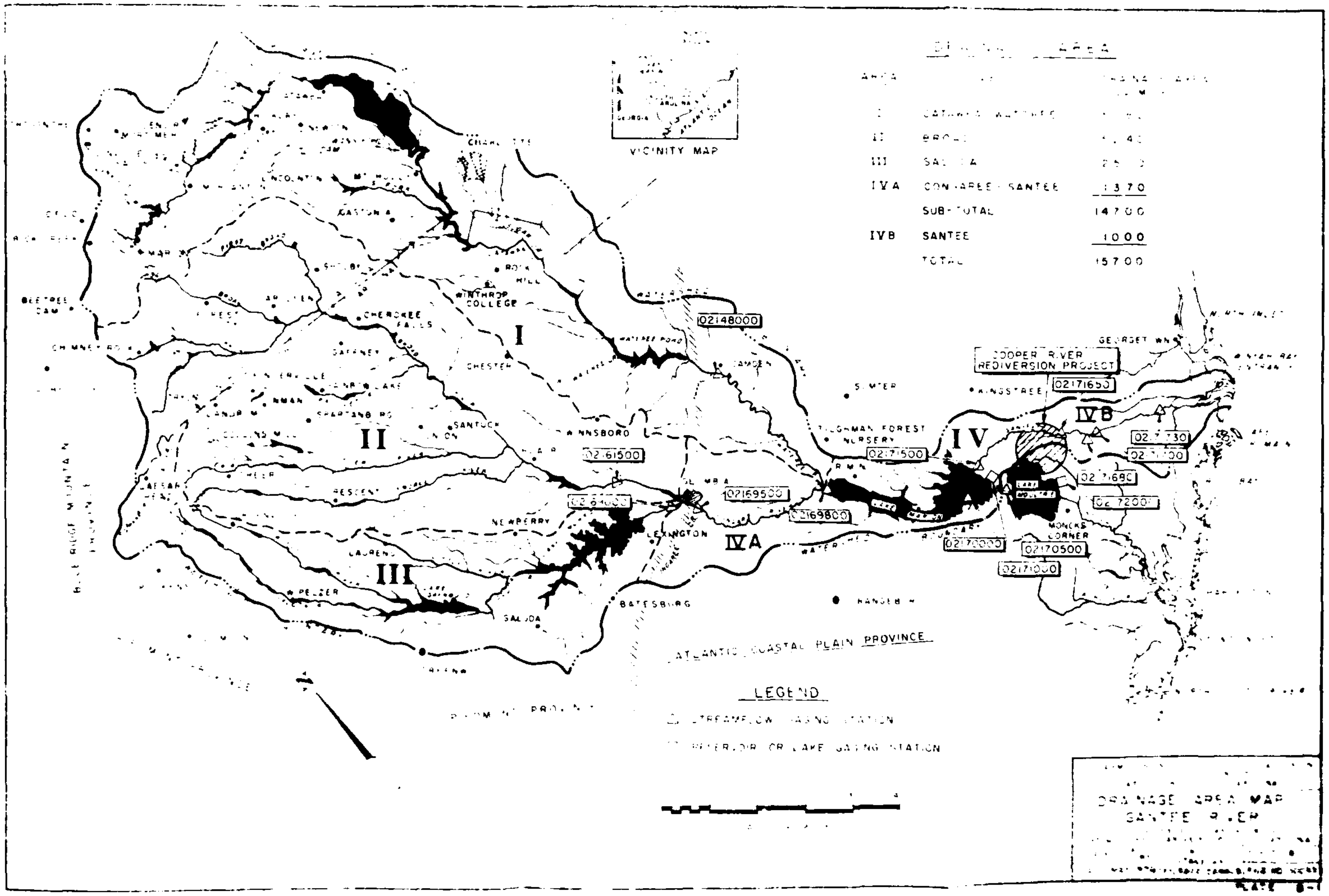
TABLE B-16

PERTINENT HYDRAULIC DESIGN DATA

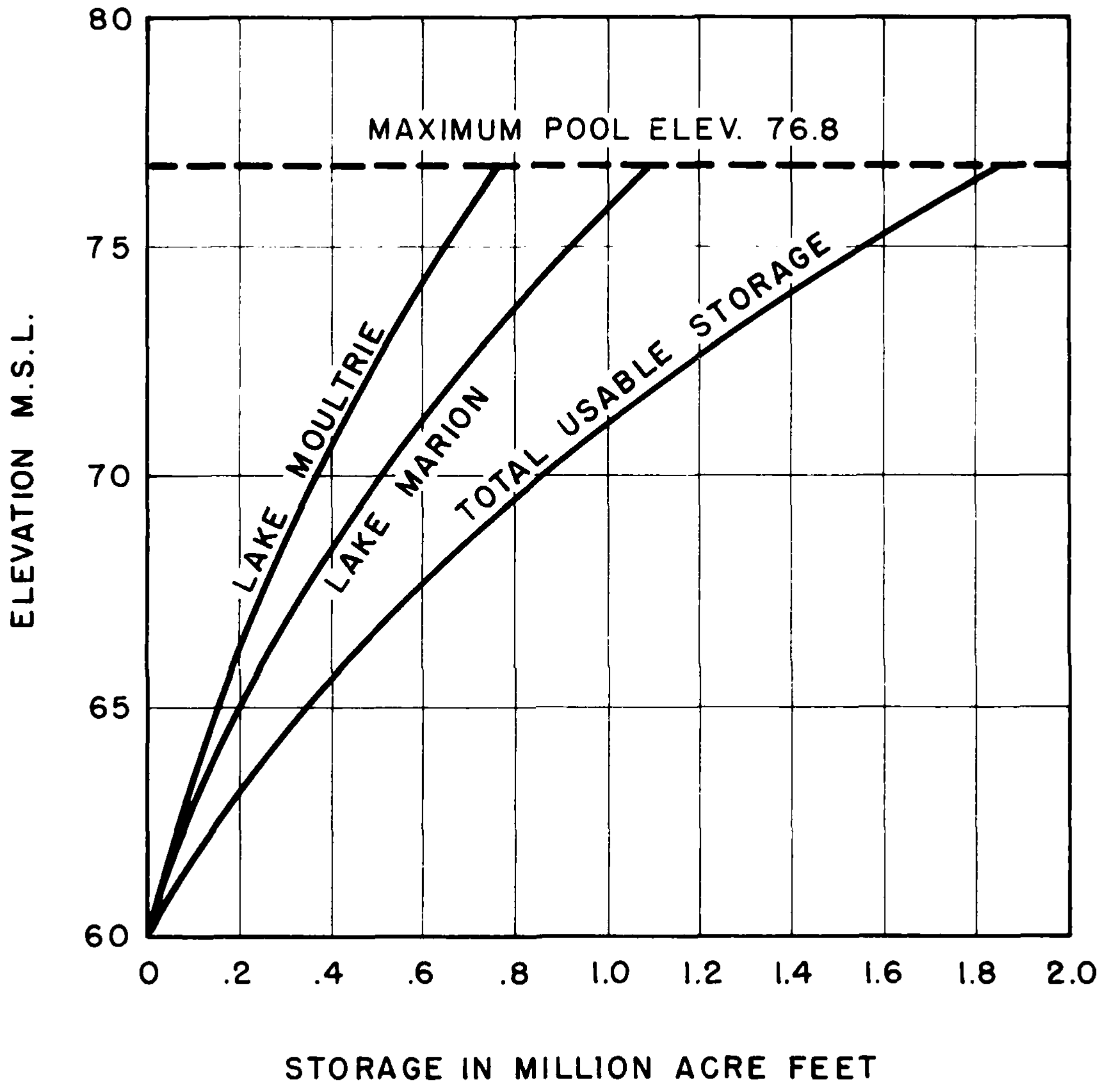
CULVERTS - INTERIOR & EXTERIOR DRAINAGE AREAS

Location	Designation	Invert Elevation	Approx Road El.	Ditch Bottom Width Ft.	Length Feet
	DRAINAGE AREA I				
(C-1) SR 35 - Lower Crossing	3-5.0' x 4.5' Box Culverts	67.0	72.5	10	40
(C-2) SR 204	2-66" Ø Conc. Pipe	67.0	75.0	10	52
(C-3) SR 35 - Upper Crossing*	2-60" Ø Conc. Pipe	70.5	79.0	10	52
	DRAINAGE AREA II				
(C-4) SR 45*	1-48" Ø Conc. Pipe	70.0	79.0	5	72
(C-5) US 52*	2-6.0' x 6.0' Box Culverts	69.0	79.0	15	122
(C-6) P. H. Access Rd.	2-6.0' x 7.0' Box Culverts	60.0	71.2	15	48
	DRAINAGE AREA NORTH OF INTAKE CANAL				
(C-7) SR 45*	1-36" Ø Conc. Pipe	78.0	83.0	5	60
(C-8) SR 293	1-36" Ø Conc. Pipe	66	71.0	5	40

NOTE: *Culvert designs at these locations are preliminary, since the number of roads that will cross the Intake Canal and their locations are not finalized at this time. Final designs will be accomplished in roads relocation DM.


VICINITY MAP
IVA CONGAREE SANTEE 1370
SUB-TOTAL 14700
IVB SANTEE 1000
TOTAL 15700
I
II
III
IV
IVA
IVB
02148000
02161500
02169500
02169800
02171500
02171650
02170000
02170500
02171000
COOPER RIVER REDIVERSION PROJECT
CHARLOTTE
ROCK HILL
WINTHROP COLLEGE
CHEROKEE FALLS
GAFFNEY
CHESTER
WINNSBORO
SANTUC
SPARTANBURG
CAMDEN
SUMTER
KINGSTREE
TILGHMAN FOREST NURSERY
GEORGETOWN
MONCKS CORNER
NEWBERRY
LEXINGTON
BATESBURG
SALUDA
PELZER
LAURENS
BEETREE DAM
BLUE RIDGE MOUNTAIN PROVINCE
ATLANTIC COASTAL PLAIN PROVINCE
LEGEND
DRAINAGE AREA MAP
SANTEE RIVER

THIS IS A REPRODUCTION OF FIGURE 1 CONTAINED IN APPENDIX E (HYDRO-ELECTRIC POWER GENERATION STUDY) OF SURVEY REPORT DATED JULY 1966.

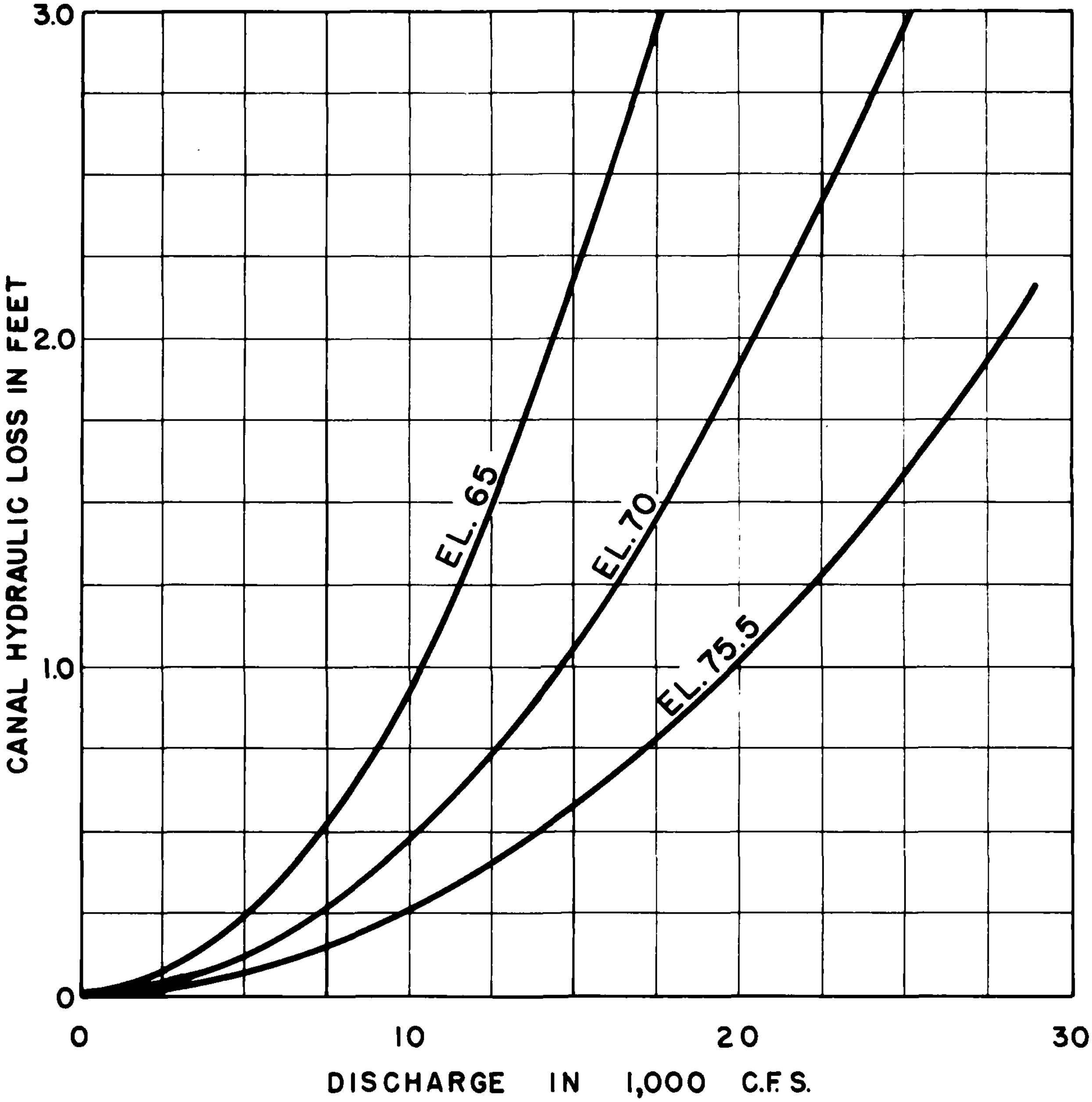


US ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

STORAGE CAPACITY – ELEVATION CURVE
SANTEE – COOPER RESERVIOR SYSTEM
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-2
DATE: MAY 1976		FILE NO.: 10043

THIS IS A REPRODUCTION OF FIGURE 5 CONTAINED IN APPENDIX E (HYDRO-ELECTRIC POWER GENERATION STUDY) OF SURVEY REPORT DATED JULY 1966.



NOTE:
ELEVATIONS ARE AVERAGE BETWEEN MARION & MOULTRIE.

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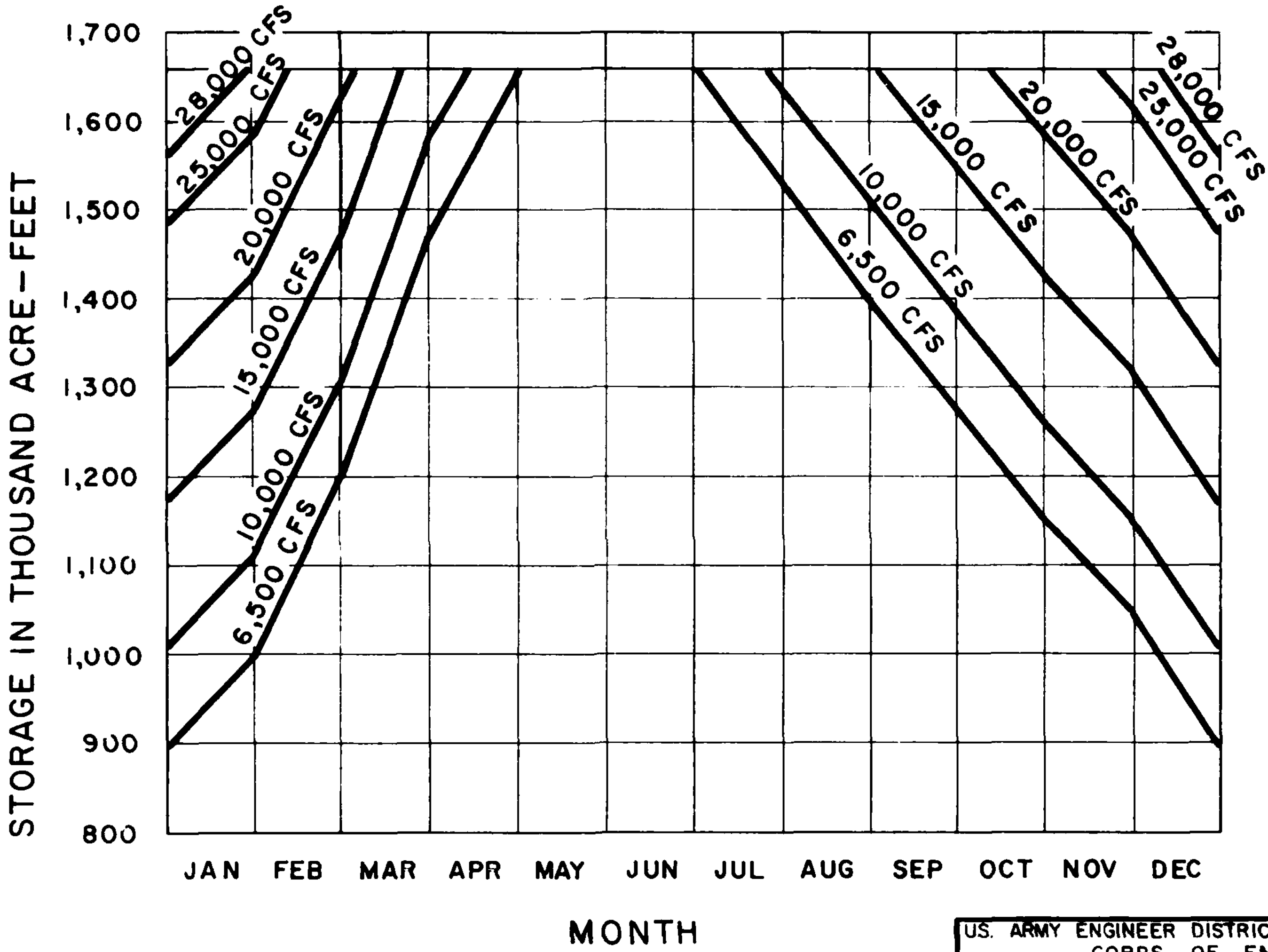
HEAD LOSS CURVES FOR EXISTING MARION-MOULTRIE DIVERSION CANAL

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-3
DATE: MAY 1976		FILE NO.: 10043

THIS IS A REPRODUCTION OF FIGURE 7 CONTAINED IN APPENDIX E (HYDRO-ELECTRIC POWER GENERATION STUDY) OF SURVEY REPORT DATED JULY 1966



NOTES:

1. SPILLS ARE MADE TO PREVENT LAKE MARION FROM EXCEEDING ELEVATION 76.8 FT. M.S.L.
2. DISCHARGE IS TOTAL FOR ST. STEPHEN, PINOPOLIS, AND SPILLWAY PLANTS.

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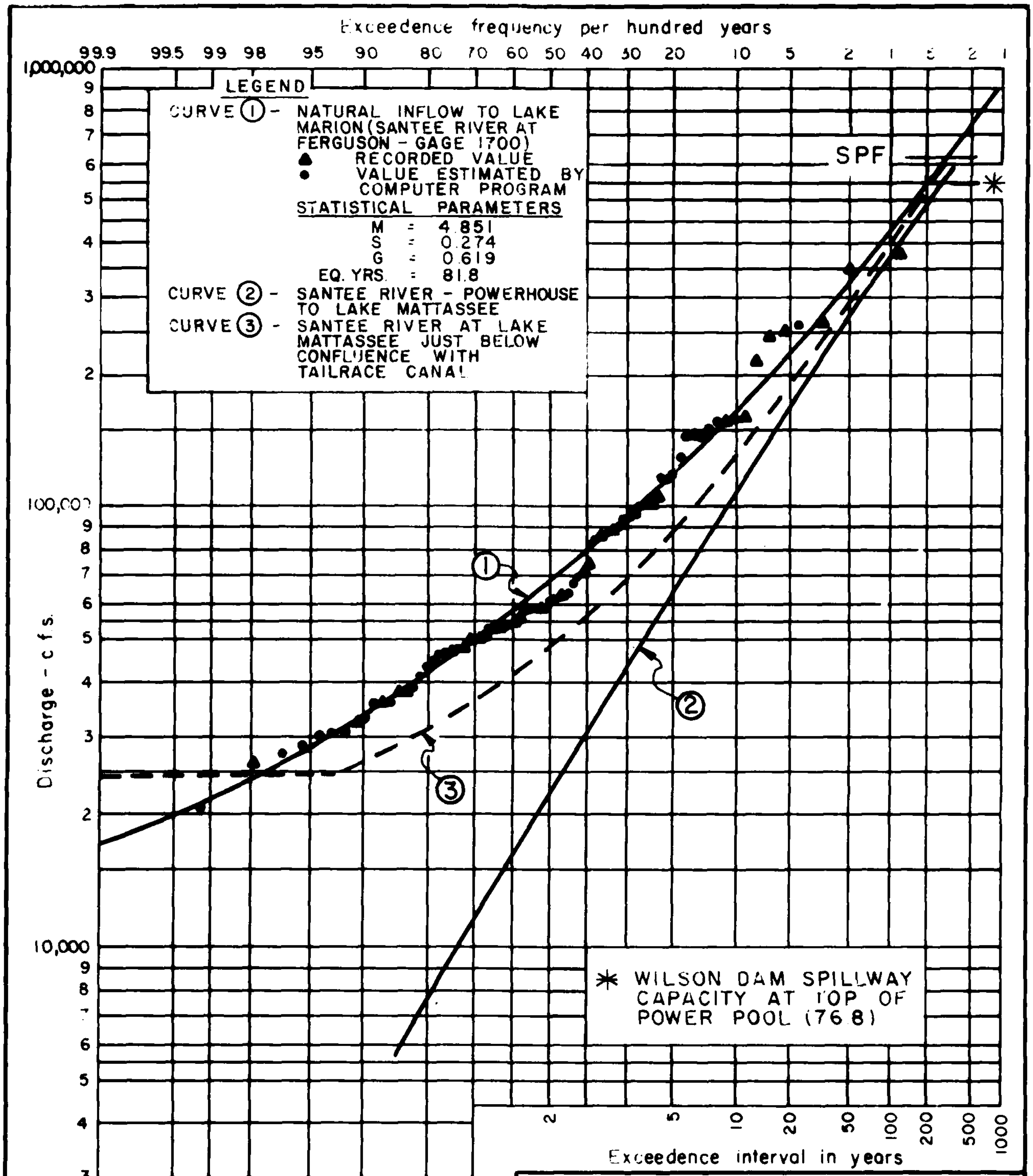
STORAGE DISCHARGE RULE CURVE FOR RESERVOIR OPERATION STUDIES

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-4
DATE: MAY 1976		FILE NO.: 10043

PLATE B-4



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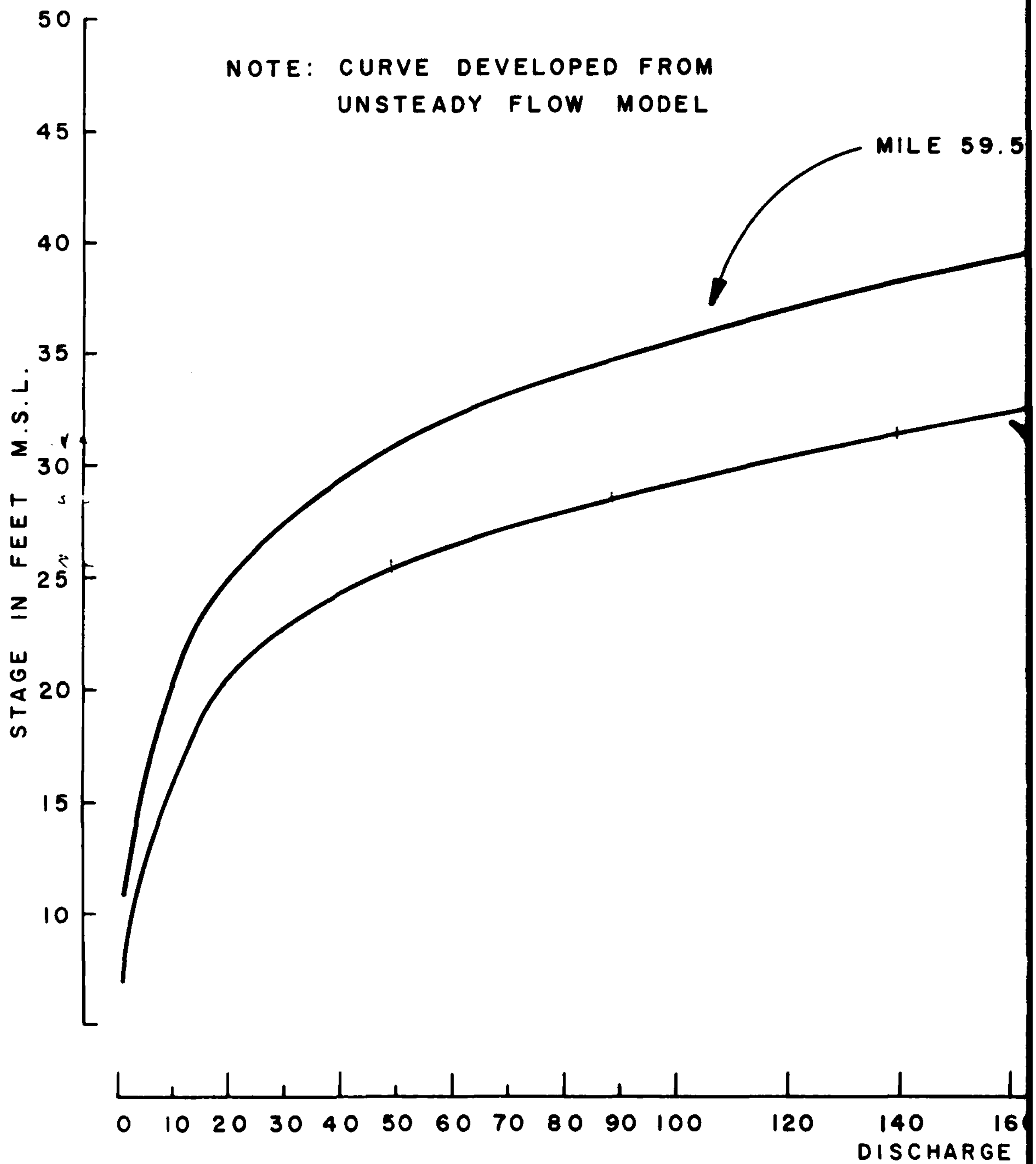
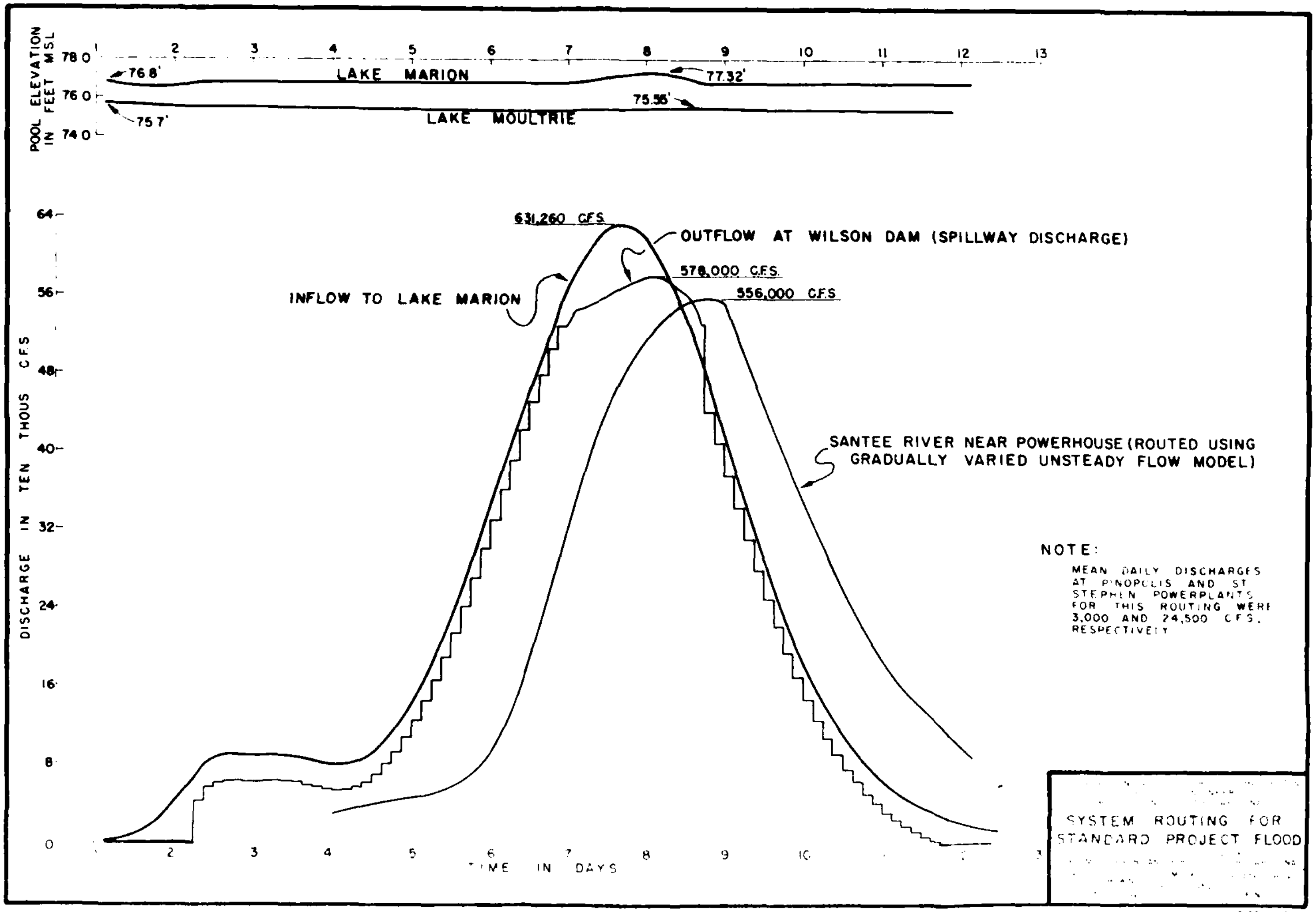
DISCHARGE FREQUENCY CURVES
SANTEE RIVER

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

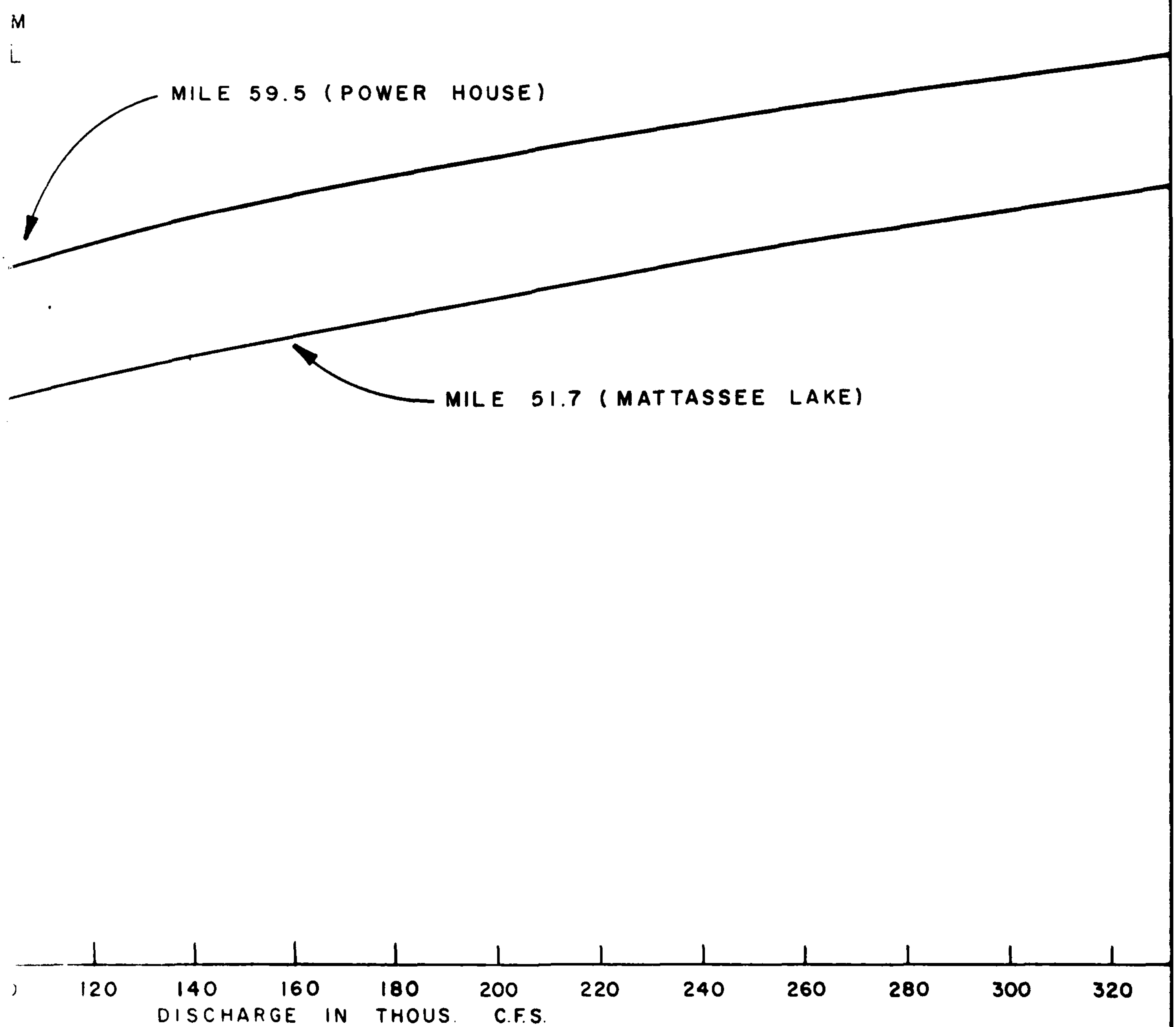
SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-5
DATE: MAY 1976		FILE NO.: 10043



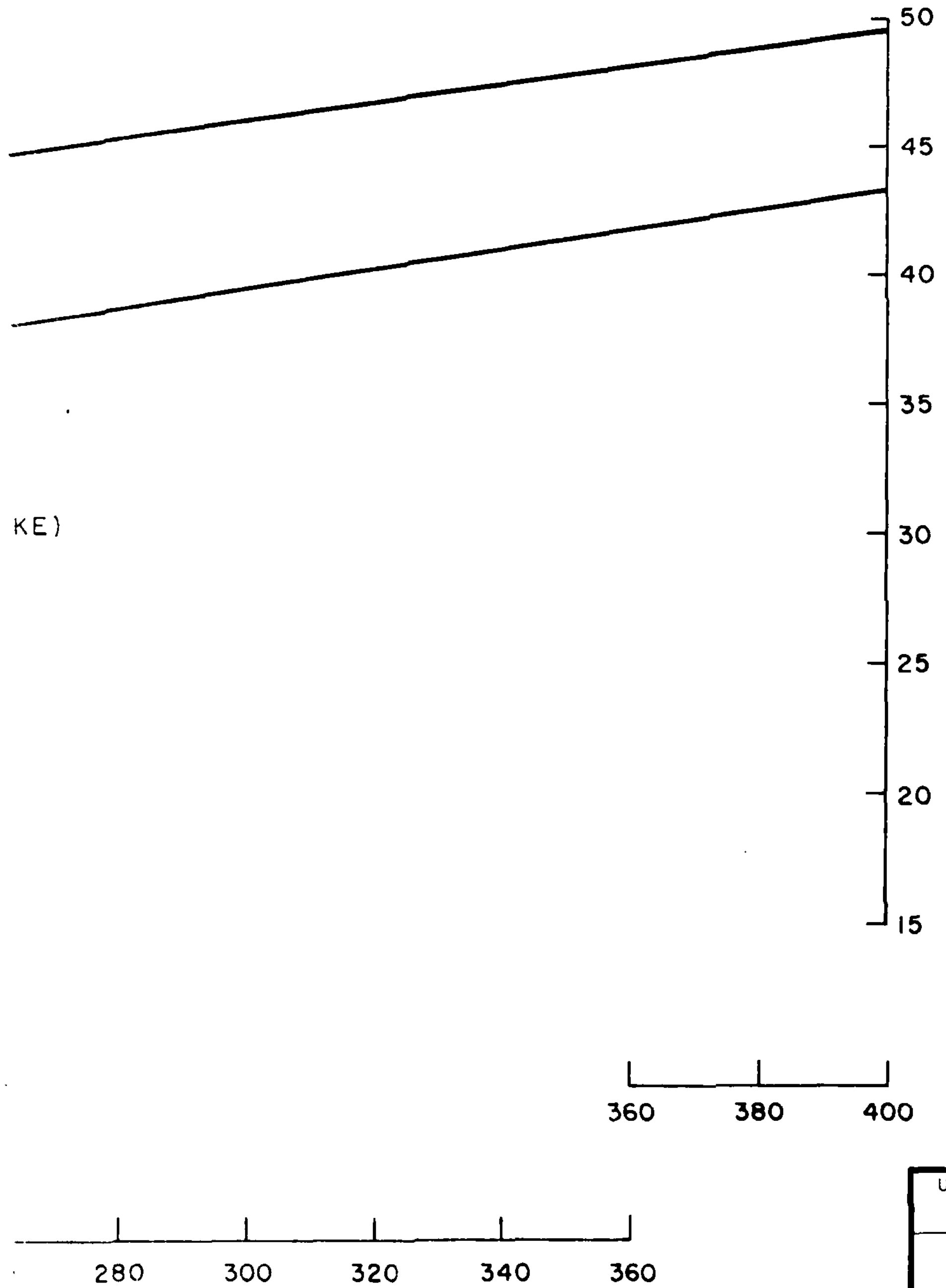
NOTE: CURVE DEVELOPED FROM UNSTEADY FLOW MODEL
MILE 59.5
STAGE IN FEET M.S.L.
50
45
40
35
30
25
20
15
10
0
10
20
30
40
50
60
70
80
90
100
120
140
DISCHARGE



SYSTEM ROUTING FOR STANDARD PROJECT FLOOD


M
L
MILE 59.5 (POWER HOUSE)
MILE 51.7 (MATTASSEE LAKE)
120
140
160
180
200
220
240
260
280
300
320
DISCHARGE IN THOUS. C.F.S.

2



US ARMY ENGINEER DISTRICT, CHARLESTON
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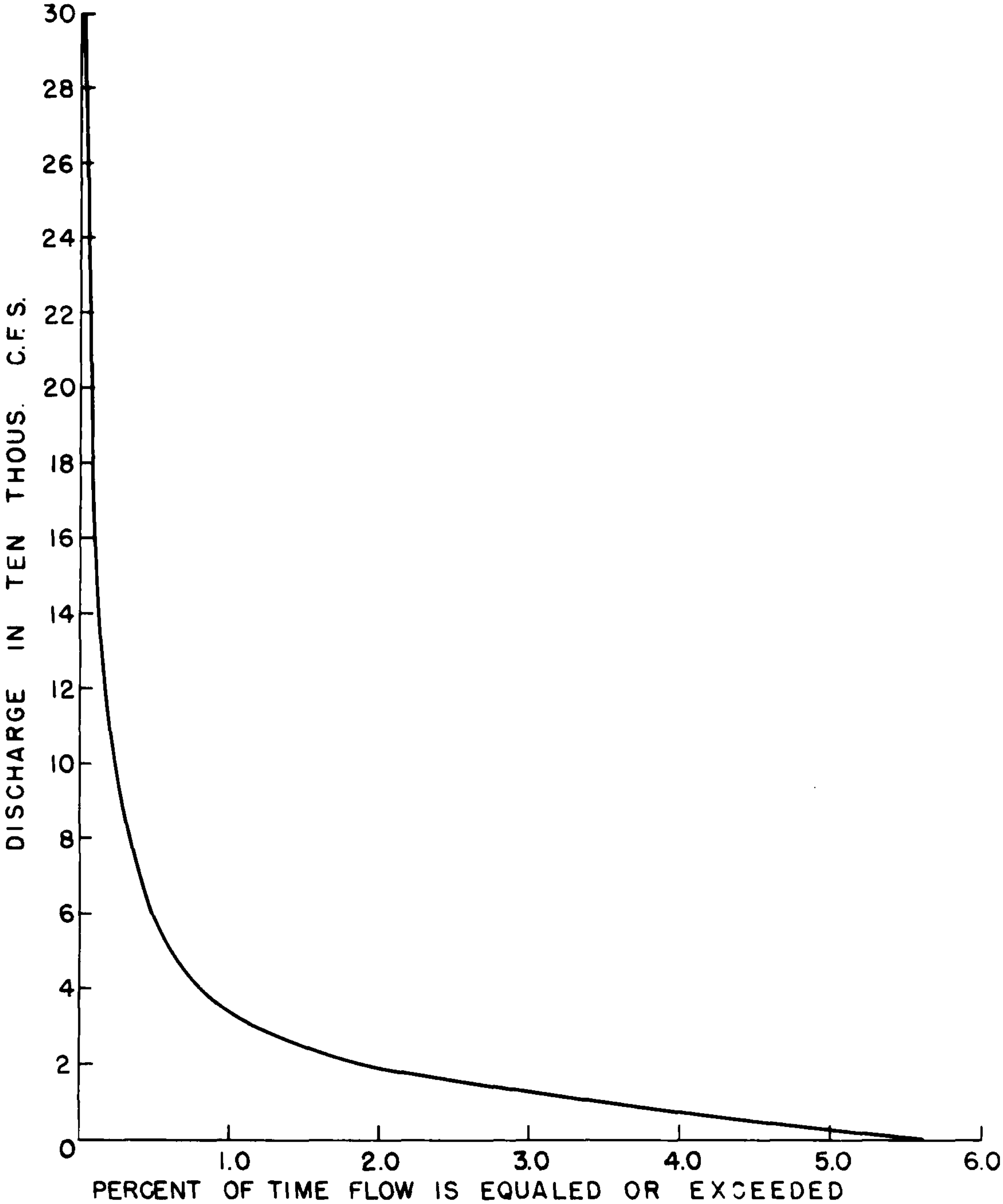
RATING CURVES FOR SANTEE RIVER AT INDICATED MILE

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B-7
DATE MAY 1976		FILE NO 10043

3

PLATE B-7



NOTE:

CURVE BASED ON SYSTEM ROUTING FOR 1908 TO 1975 PERIOD.

U.S. ARMY ENGINEER DISTRICT, CHARLESTON
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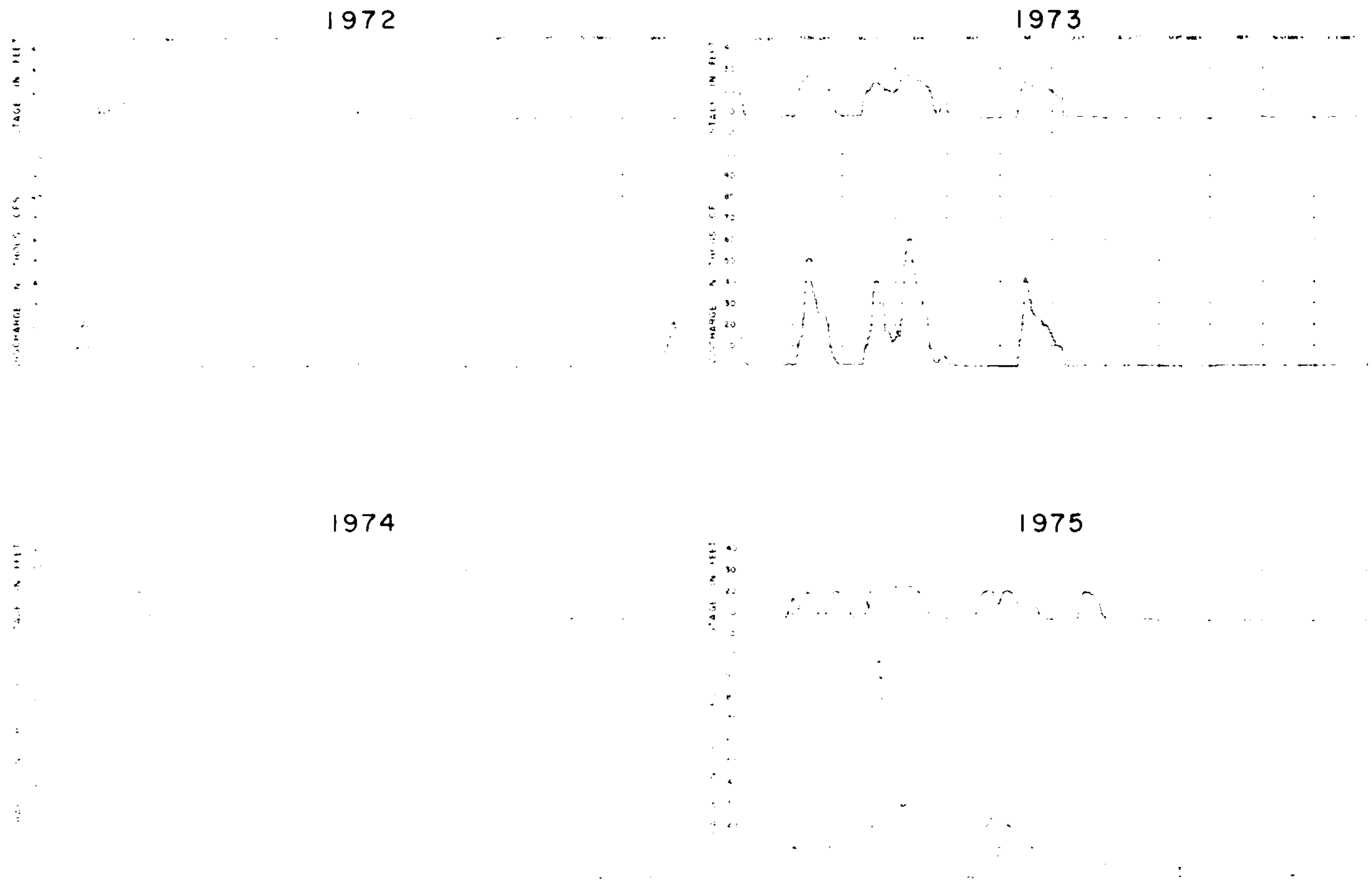
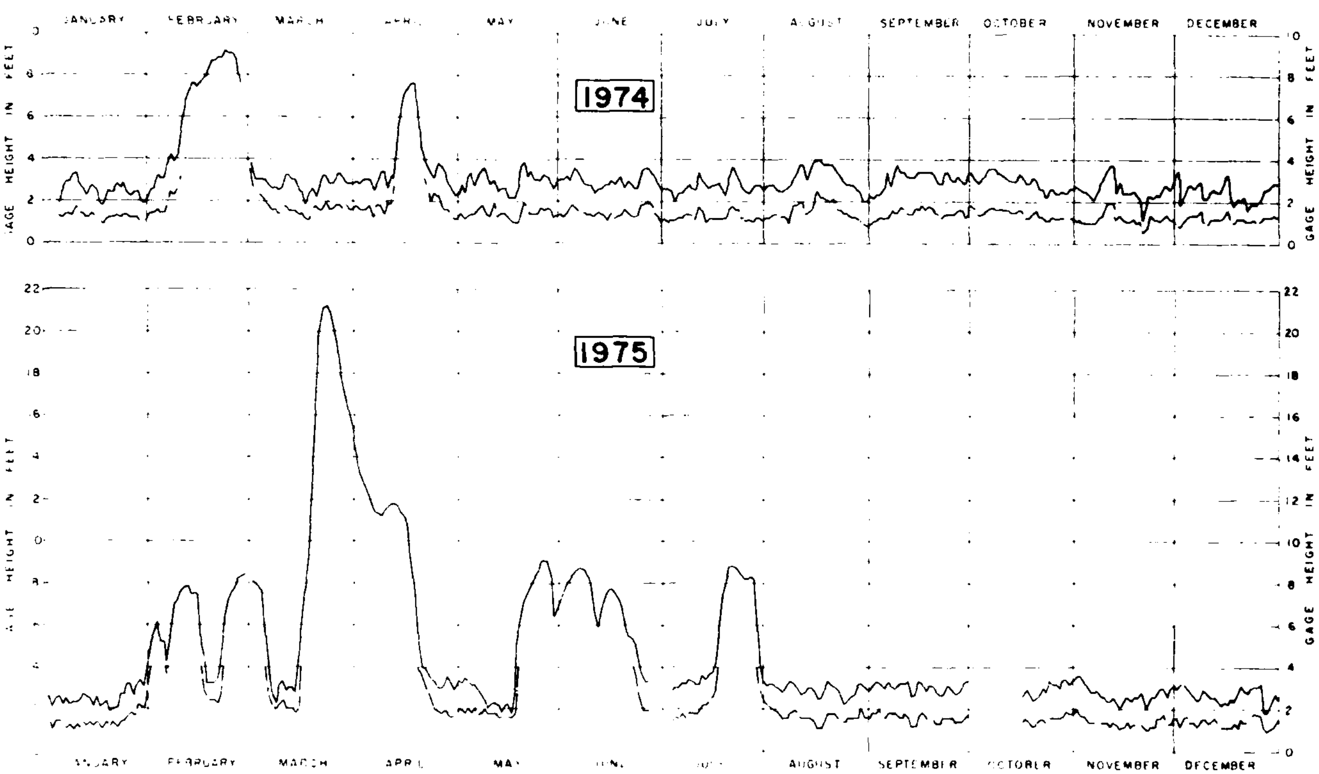
FLOW DURATION CURVE
SPILLS AT WILSON DAM

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-8
DATE: MAY 1976		FILE NO: 10043



STAGE DISCHARGE
SANTEE RIVER BELOW ST STEPHEN
GAGE NO 02171650
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

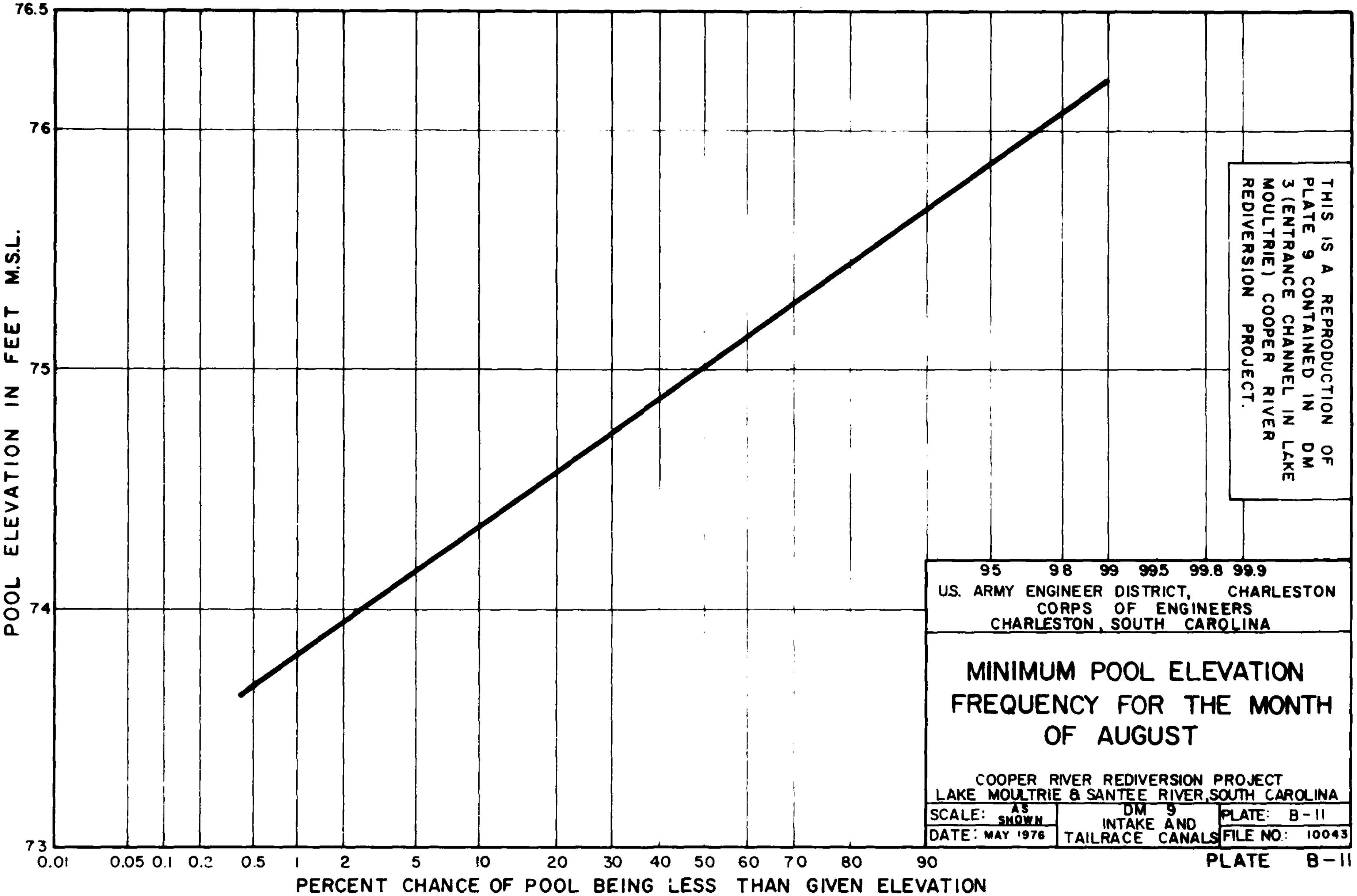


NOTE:
DATUM OF GAGE ... AT
MEAN SEA LEVEL

LEGEND

MAXIMUM GAGE HEIGHT
MINIMUM GAGE HEIGHT

STAGE - DISCHARGE
SANTEE RIVER NEAR JAMESTOWN, SC
GAGE NO 02171700



THIS IS A REPRODUCTION OF PLATE 9 CONTAINED IN DM 3 (ENTRANCE CHANNEL IN LAKE MOULTRIE) COOPER RIVER REDIVERSION PROJECT.

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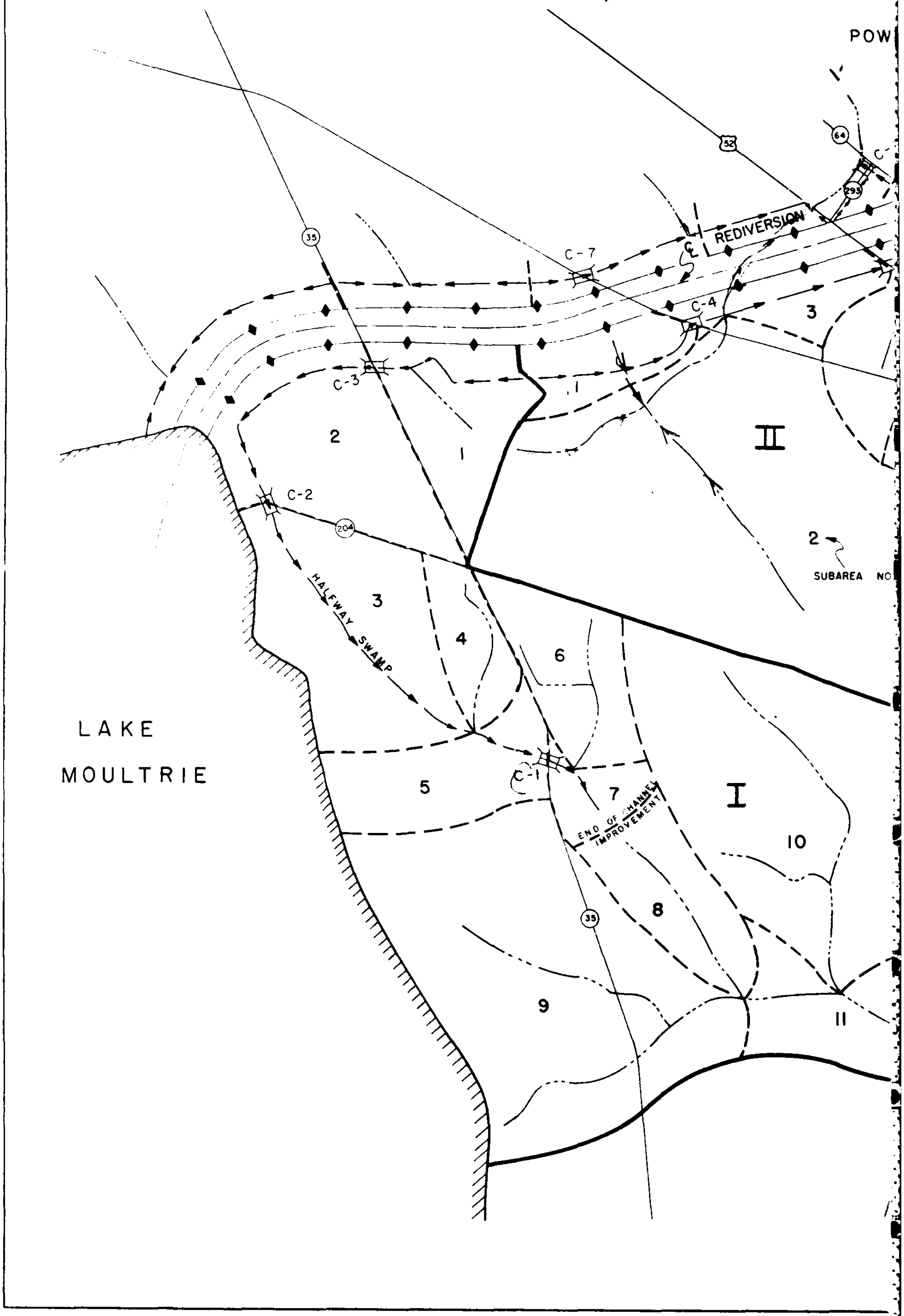
CHARLESTON, SOUTH CAROLINA

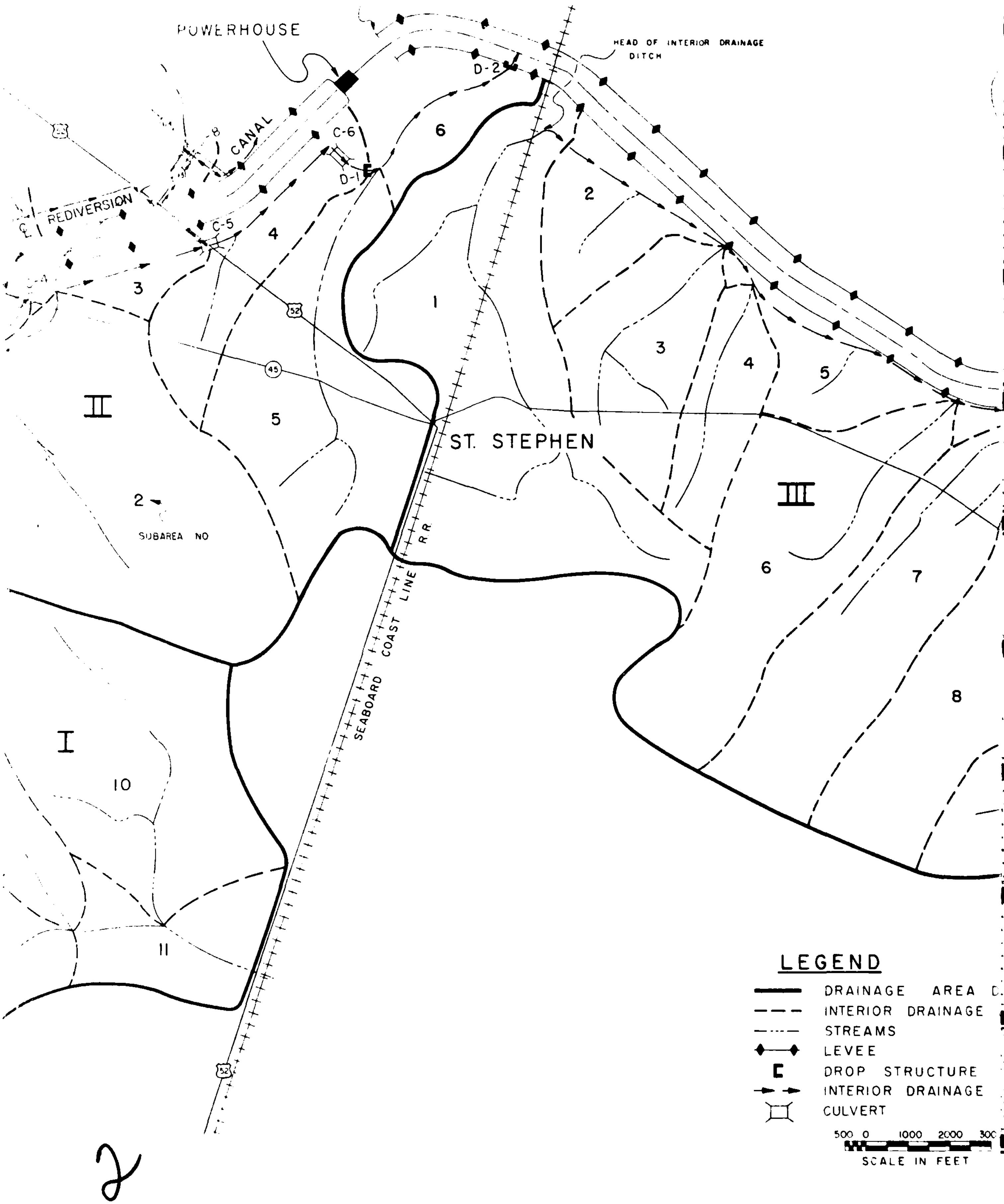
MINIMUM POOL ELEVATION FREQUENCY FOR THE MONTH OF AUGUST

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE: AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-11
DATE: MAY 1976		FILE NO: 10043

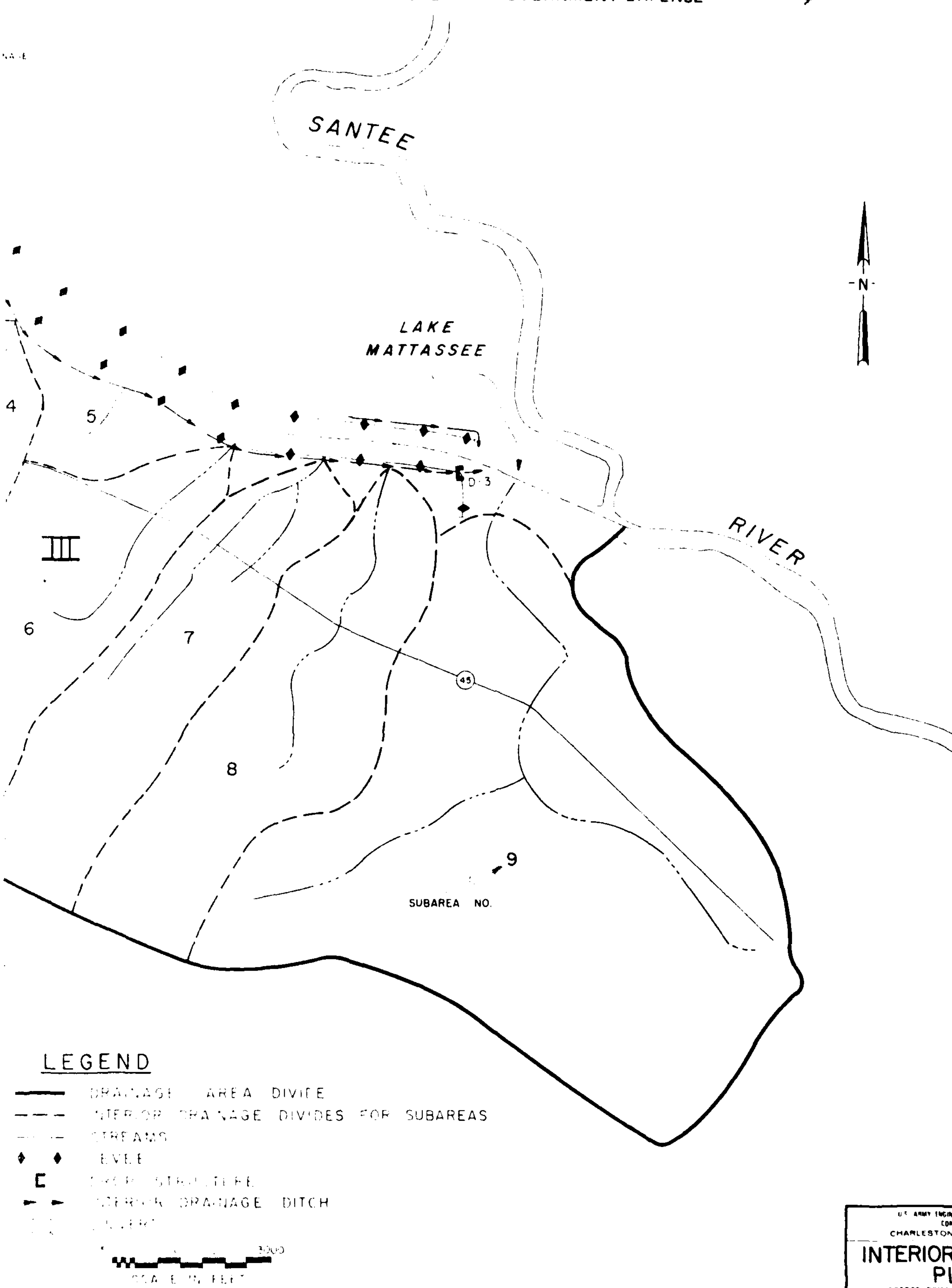
PLATE B-11


LAKE
MOULTRIE
HALFWAY SWAMP
REDIVERSION
END OF CHANNEL IMPROVEMENT
C-1
C-2
C-3
C-4
C-7
35
52
64
204
293
I
II
1
2
3
4
5
6
7
8
9
10
11


POWERHOUSE
HEAD OF INTERIOR DRAINAGE DITCH
D-2
CANAL
C-6
D-1
REDIVERSION
C-5
6
2
4
3
1
II
5
ST. STEPHEN
III
2
SUBAREA NO
3
4
5
6
7
8
R.R.
SEABOARD COAST LINE
I
10
11
45
52
LEGEND
DRAINAGE AREA
INTERIOR DRAINAGE
STREAMS
LEVEE
DROP STRUCTURE
INTERIOR DRAINAGE
CULVERT
500 0 1000 2000
SCALE IN FEET

2

[redacted]



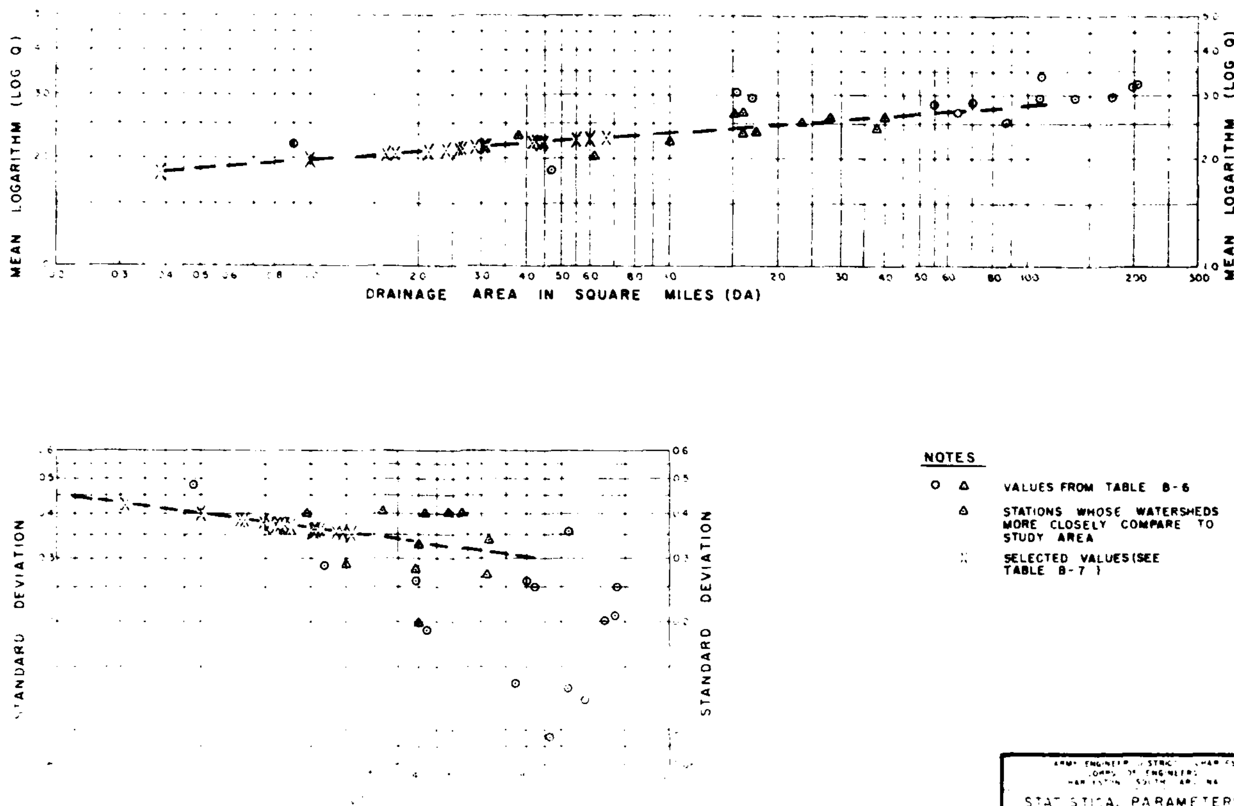
U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

INTERIOR DRAINAGE PLAN

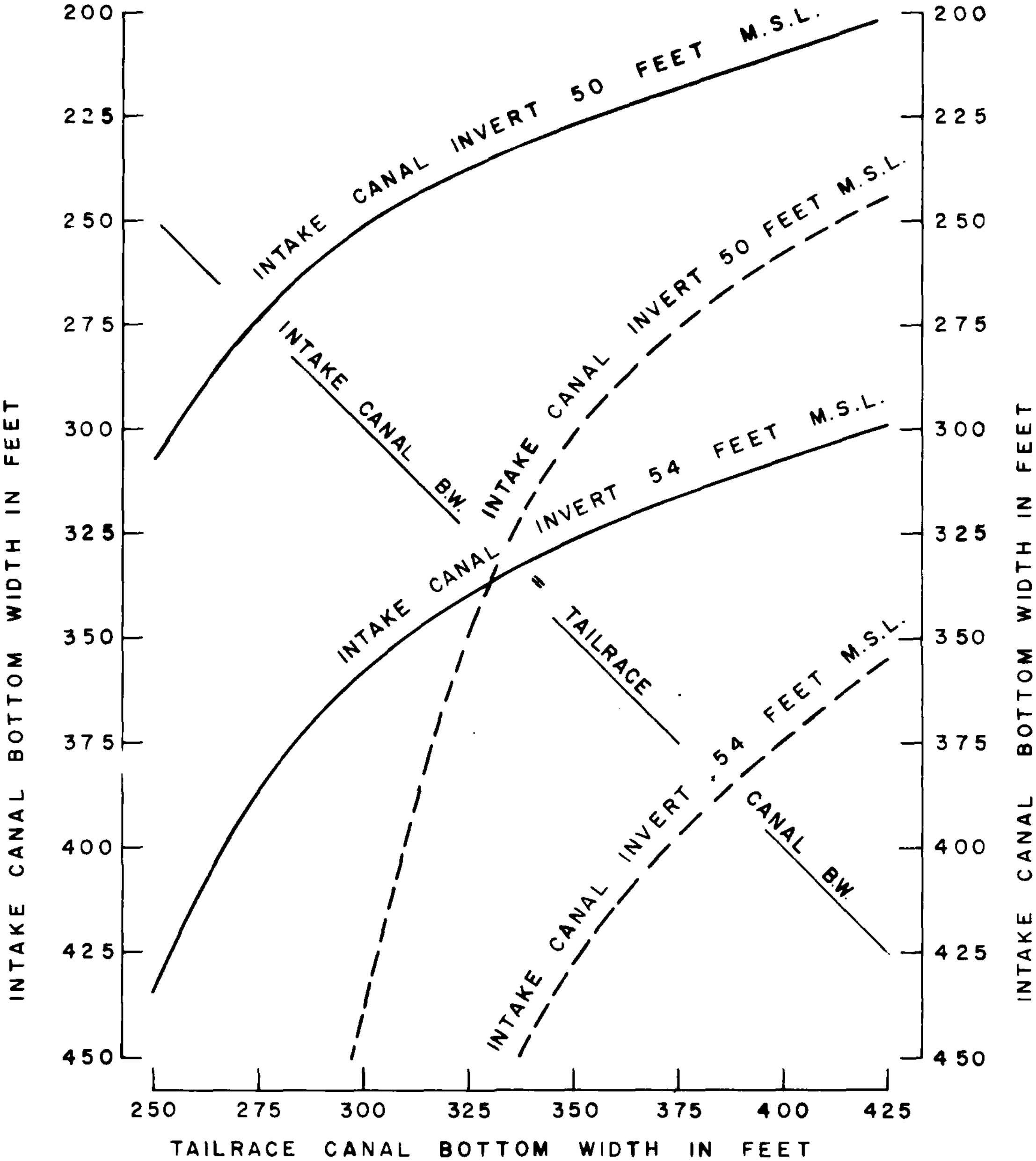
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B 2	
DATE MAY 1976		FILE NO	

PLATE B-



STATISTICAL PARAMETERS FOR FREQUENCY ANALYSIS



LEGEND

——— TAILRACE CANAL INVERT AT EL. 0.0 FEET M.S.L.

– – – – TAILRACE CANAL INVERT AT EL. 3.5 FEET M.S.L.

NOTE

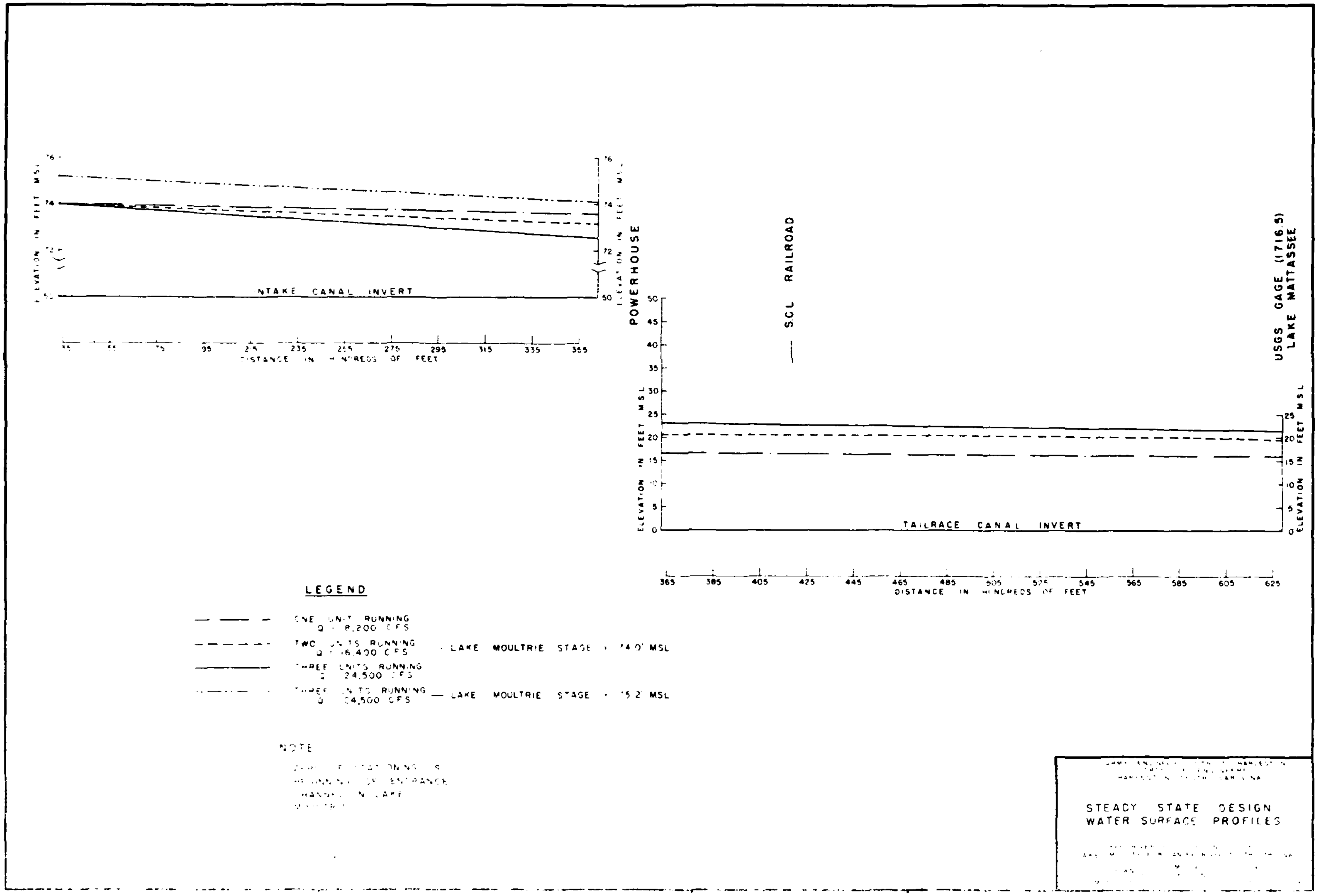
GRAPH WAS DEVELOPED FOR COMPARISON OF DESIGNS AND DOES NOT INCLUDE THE EFFECTS OF BRIDGES.

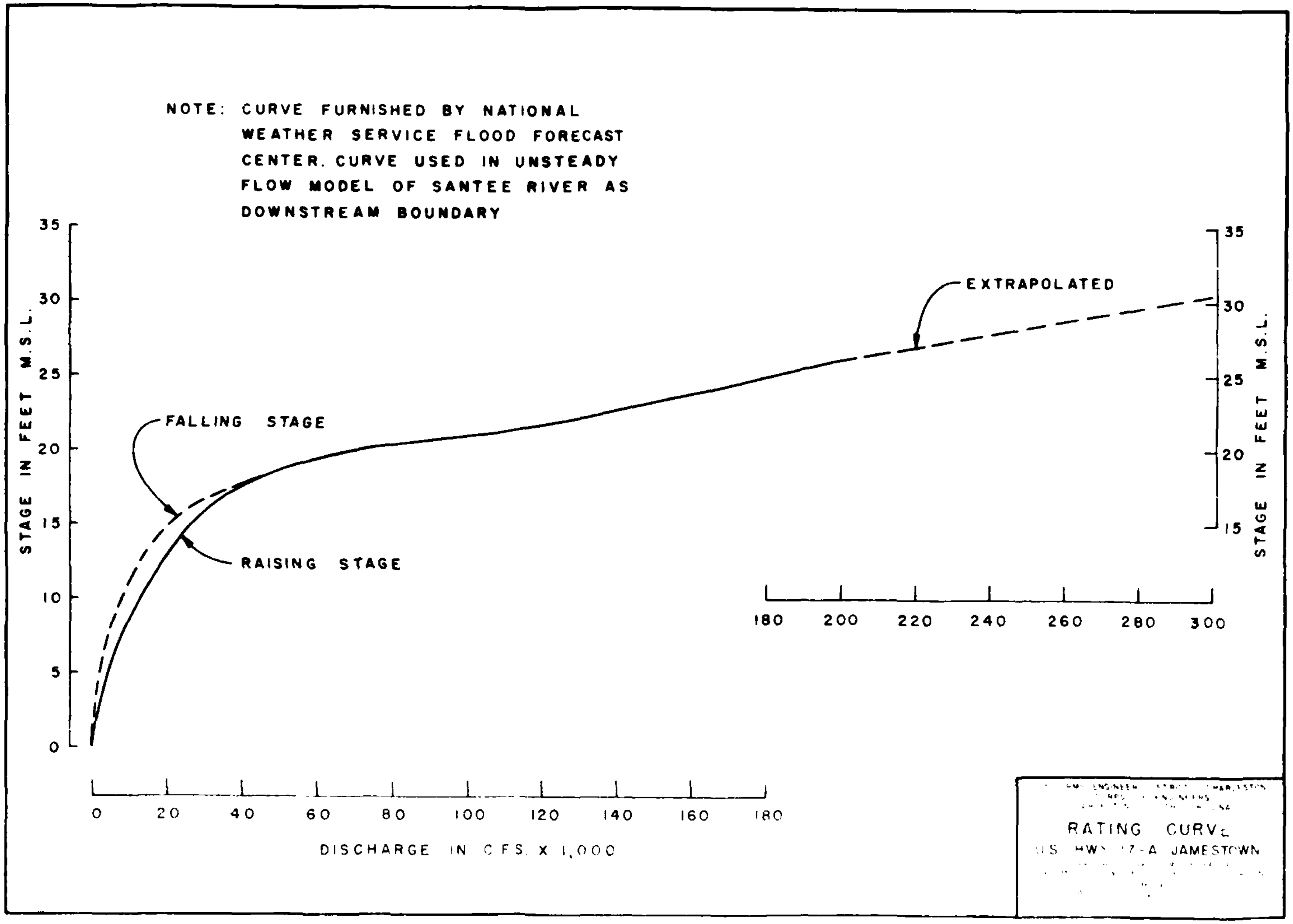
U S ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

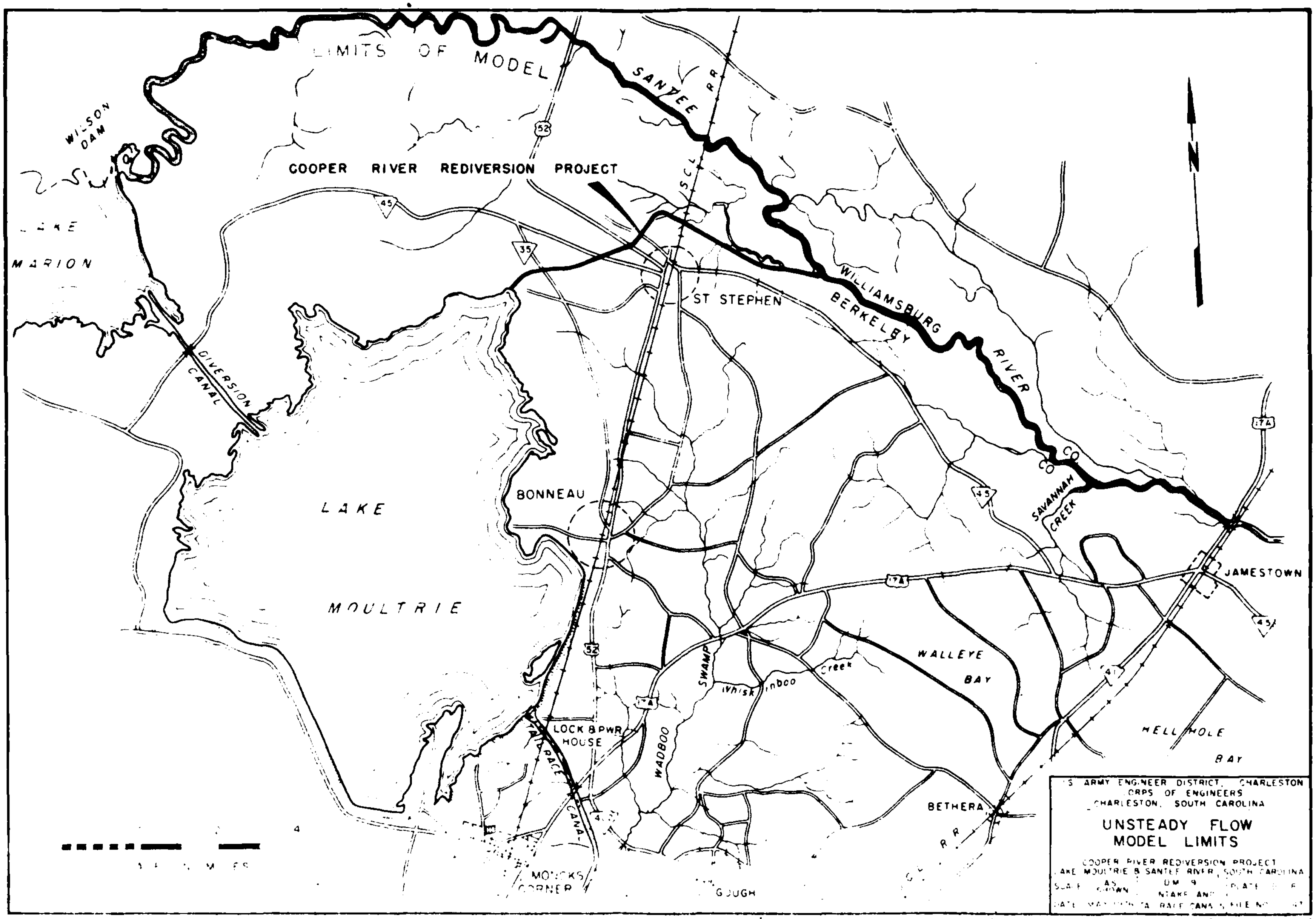
TAILRACE CANAL BOTTOM WIDTHS
vs.
INTAKE CANAL BOTTOM WIDTHS
FOR INDICATED INVERT ELEVATION

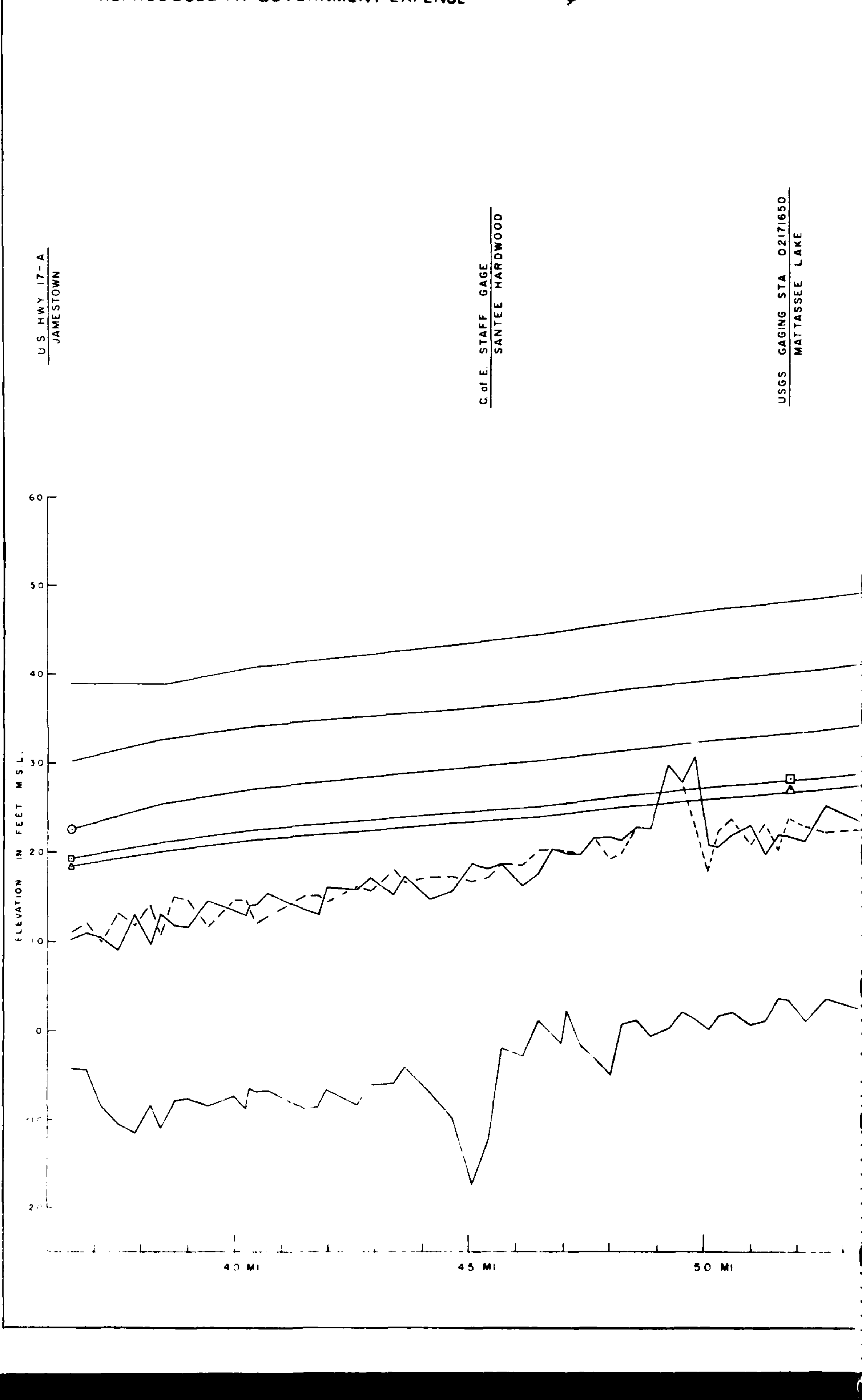
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B - 14
DATE MAY 1976		FILE NO 10043

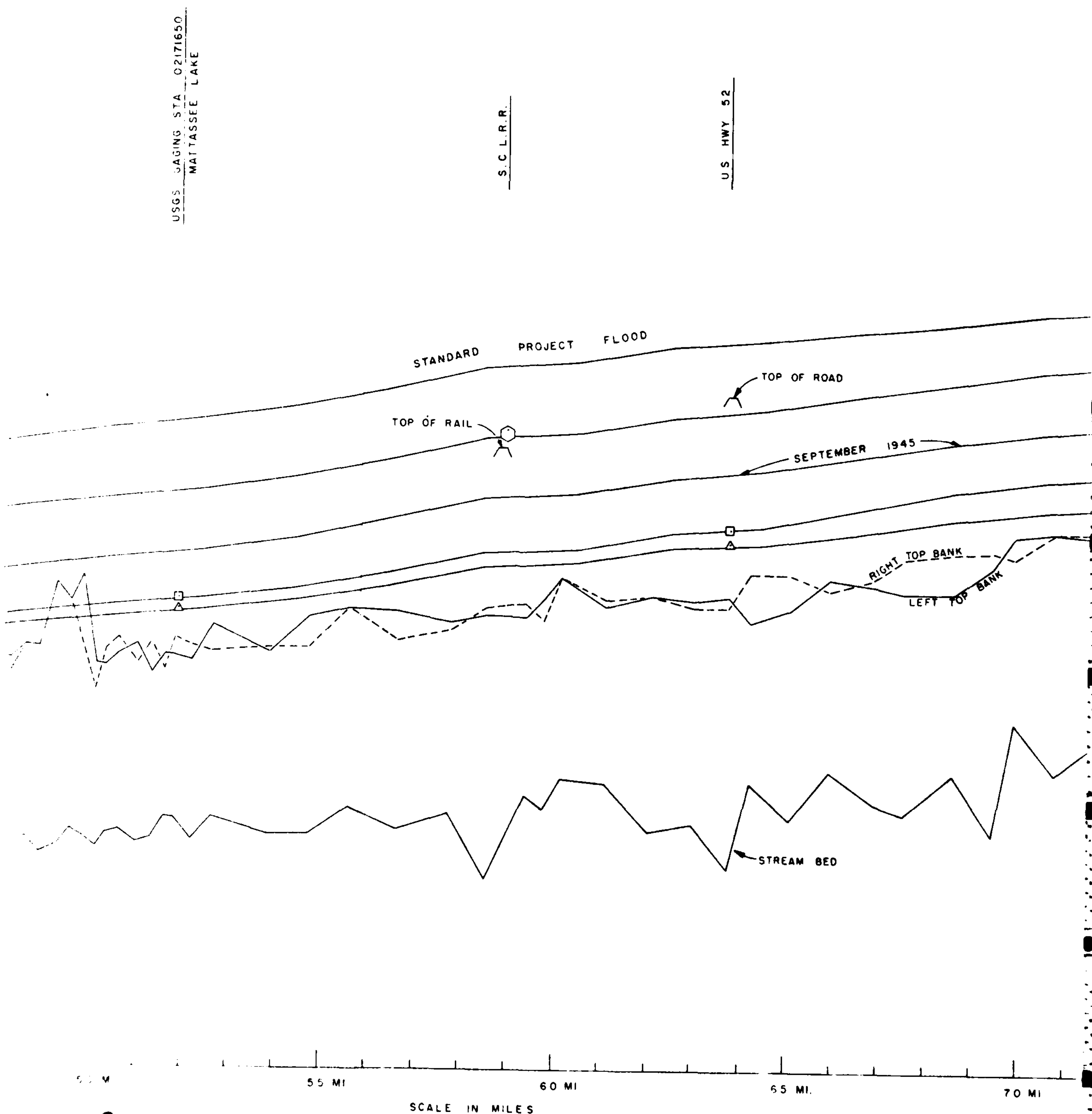

POWERHOUSE
INTAKE CANAL INVERT
ELEVATION IN FEET MSL
DISTANCE IN HUNDREDS OF FEET
S.C.L. RAILROAD
USGS GAGE (1716.5)
LAKE MATTASSEE
TAILRACE CANAL INVERT
LEGEND
ONE UNIT RUNNING
TWO UNITS RUNNING — LAKE MOULTRIE STAGE
THREE UNITS RUNNING
THREE UNITS RUNNING — LAKE MOULTRIE STAGE
NOTE
STEADY STATE DESIGN
WATER SURFACE PROFILES


NOTE: CURVE FURNISHED BY NATIONAL WEATHER SERVICE FLOOD FORECAST CENTER. CURVE USED IN UNSTEADY FLOW MODEL OF SANTEE RIVER AS DOWNSTREAM BOUNDARY
FALLING STAGE
RAISING STAGE
EXTRAPOLATED
STAGE IN FEET M.S.L.
0
5
10
15
20
25
30
35
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
DISCHARGE IN C.F.S. X 1,000
RATING CURVE
US HWY 17-A JAMESTOWN

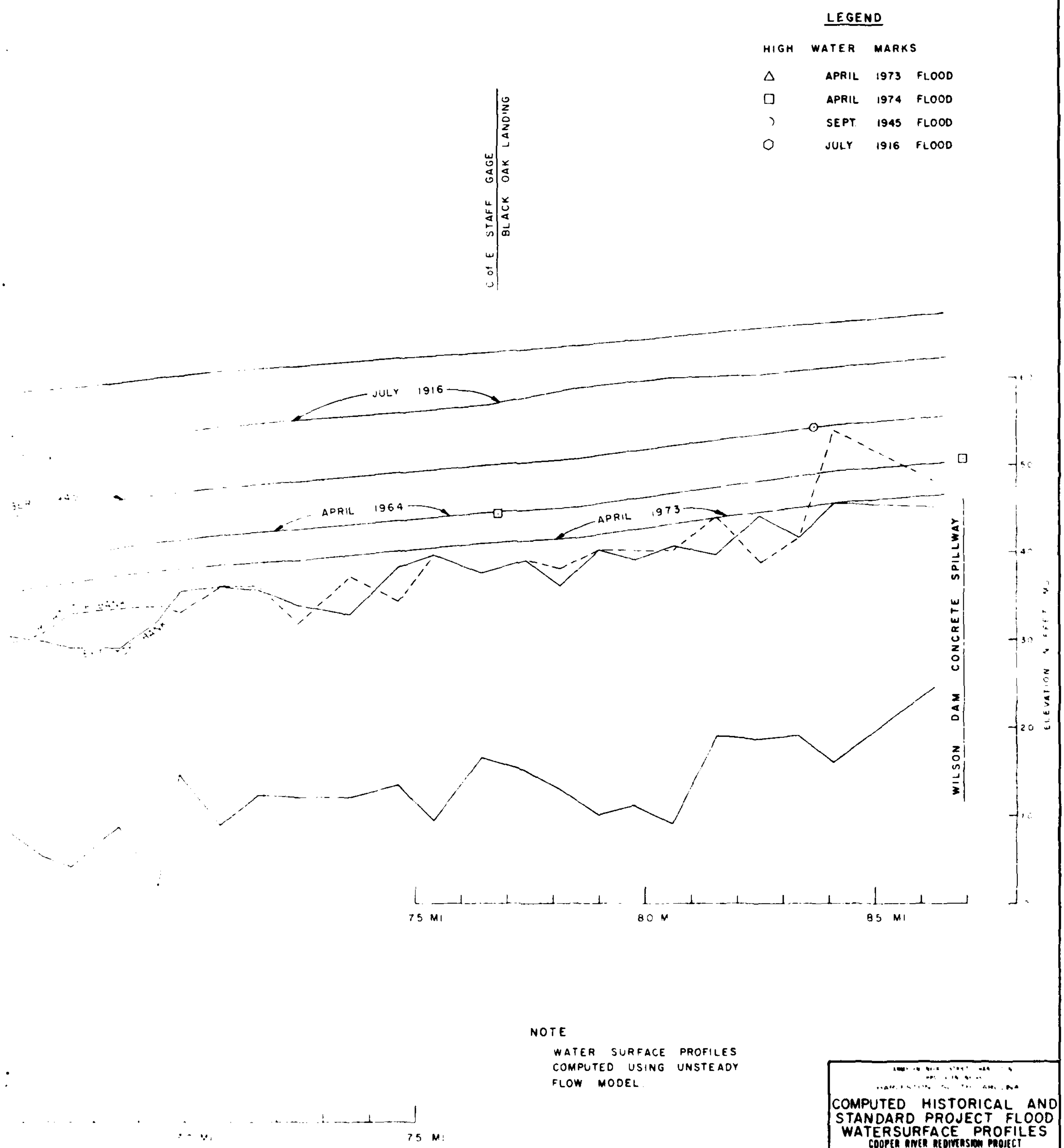

LIMITS OF MODEL
SANTEE
COOPER RIVER REDIVERSION PROJECT
WILSON DAM
LAKE MARION
DIVERSION CANAL
LAKE MOULTRIE
ST STEPHEN
BONNEAU
WILLIAMSBURG
BERKELEY
RIVER
SAVANNAH CREEK
JAMESTOWN
WALLEYE BAY
HELL HOLE BAY
BETHERA
WADBOO
SWAMP
Whiskinboo Creek
LOCK & PWR HOUSE
TAILRACE CANAL
MONCKS CORNER
GOUGH
N
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA
UNSTEADY FLOW
MODEL LIMITS
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA



U S HWY 17-A
JAMESTOWN
C. of E. STAFF GAGE
SANTEE HARDWOOD
USGS GAGING STA 02171650
MATTASSEE LAKE
ELEVATION IN FEET M.S.L.
60
50
40
30
20
10
0
40 MI
45 MI
50 MI


USGS GAGING STA 02171650
MATTASSEE LAKE
S.C.L.R.R.
US HWY 52
STANDARD PROJECT FLOOD
TOP OF ROAD
TOP OF RAIL
SEPTEMBER 1945
RIGHT TOP BANK
LEFT TOP BANK
STREAM BED
55 MI
60 MI
65 MI.
70 MI
SCALE IN MILES

2

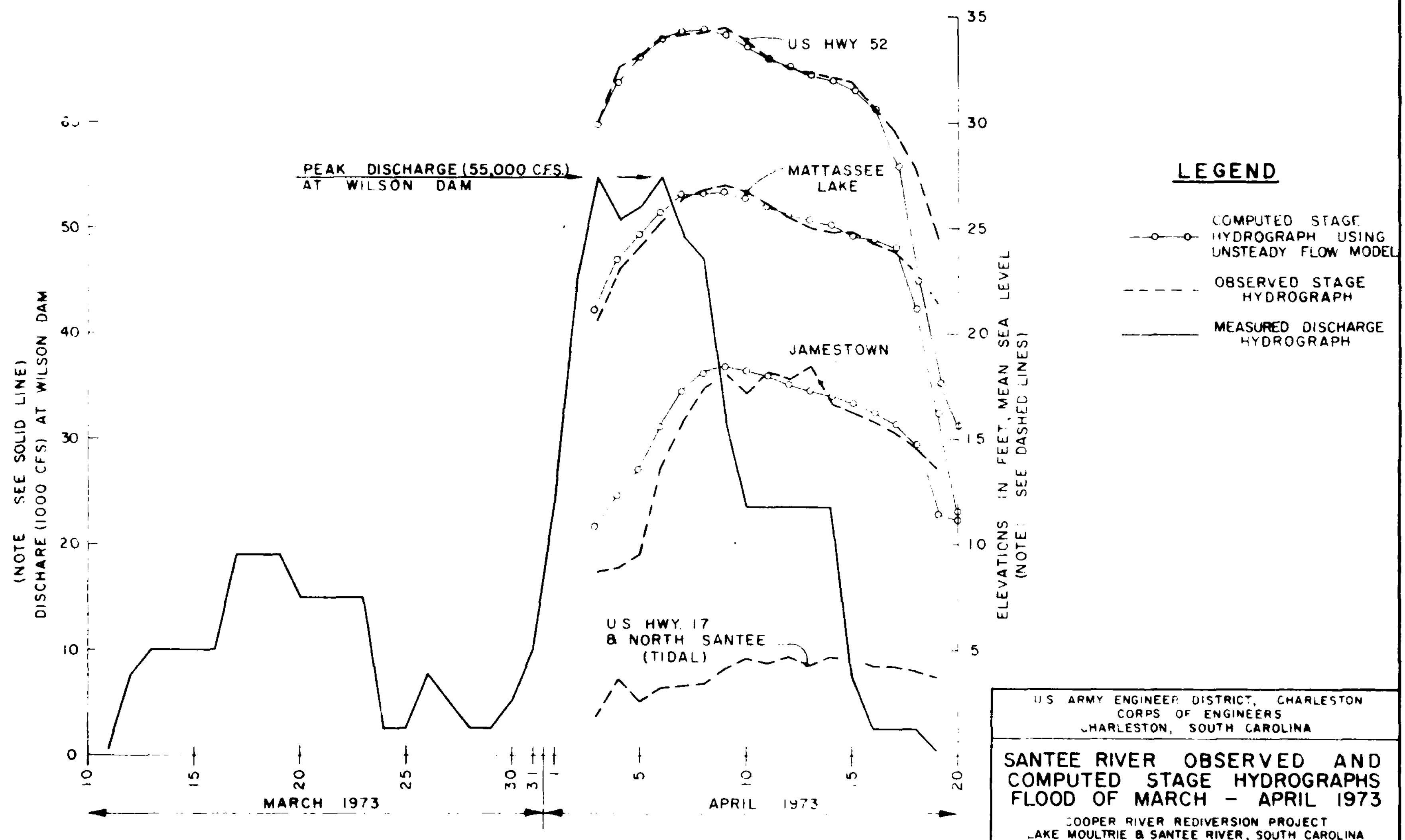


NOTE
WATER SURFACE PROFILES COMPUTED USING UNSTEADY FLOW MODEL.

COMPUTED HISTORICAL AND STANDARD PROJECT FLOOD WATERSURFACE PROFILES
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

PLATE

3

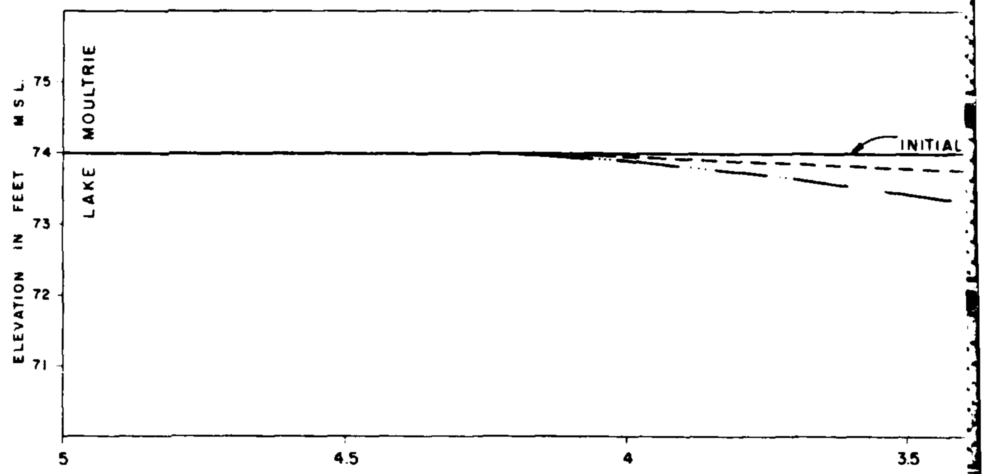


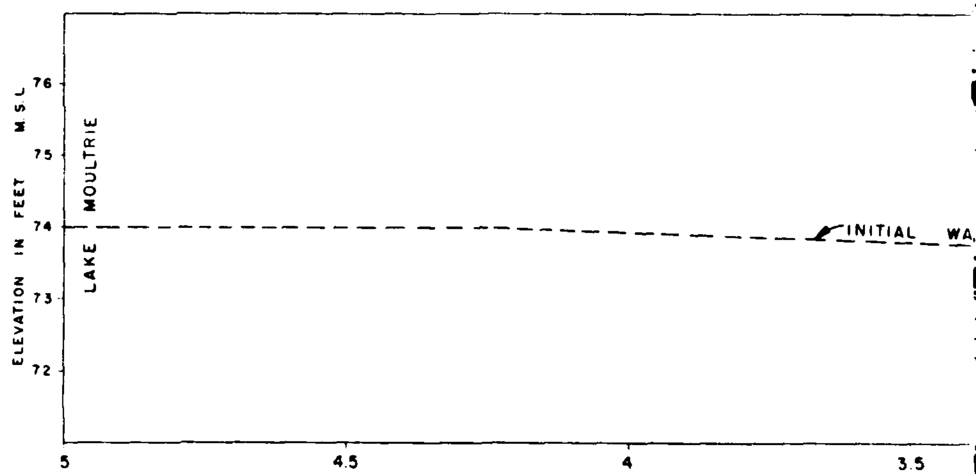
US ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

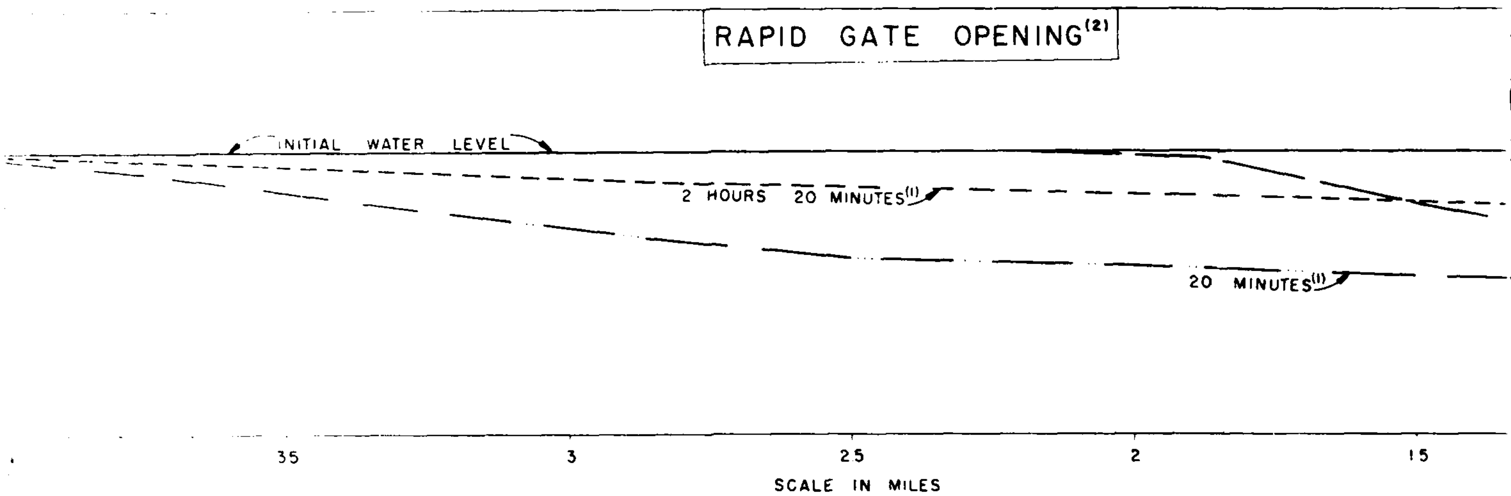
SANTEE RIVER OBSERVED AND COMPUTED STAGE HYDROGRAPHS FLOOD OF MARCH - APRIL 1973

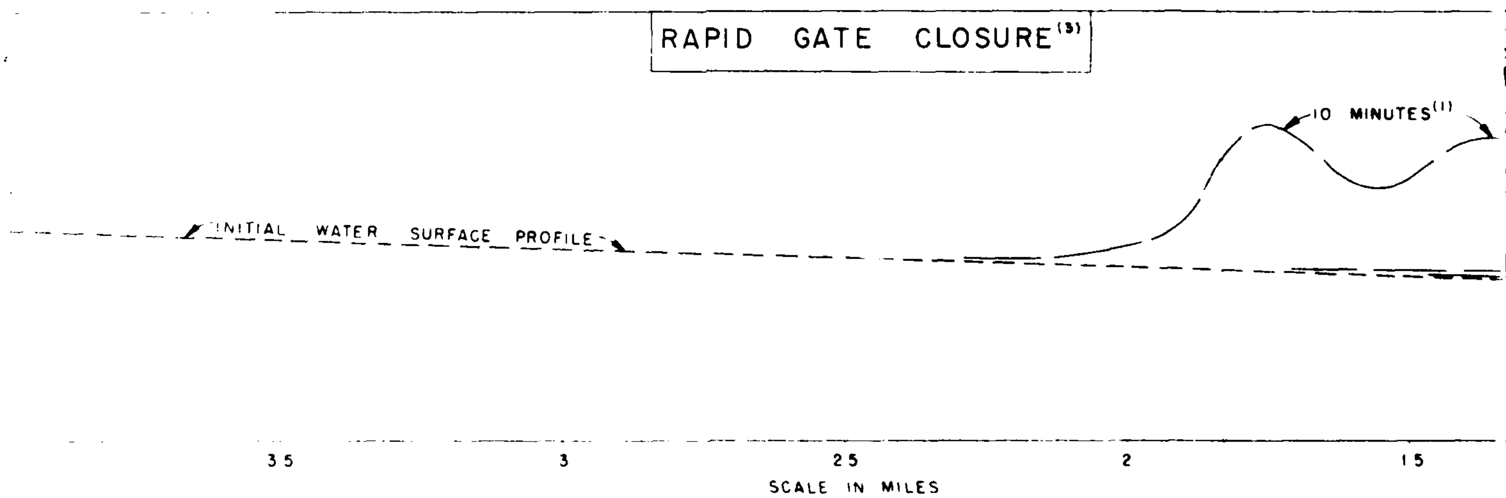
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B-19
DATE MAY 1976		FILE NO 10043


ELEVATION IN FEET M.S.L.
LAKE MOULTRIE
75
74
73
72
71
INITIAL
5
4.5
4
3.5


ELEVATION IN FEET M.S.L.
LAKE MOULTRIE
76
75
74
73
72
INITIAL WA
5
4.5
4
3.5


RAPID GATE OPENING(2)
INITIAL WATER LEVEL
2 HOURS 20 MINUTES(1)
20 MINUTES(1)
3.5
3
2.5
2
1.5
SCALE IN MILES


RAPID GATE CLOSURE(3)
10 MINUTES(1)
INITIAL WATER SURFACE PROFILE
3.5
3
2.5
2
1.5
SCALE IN MILES

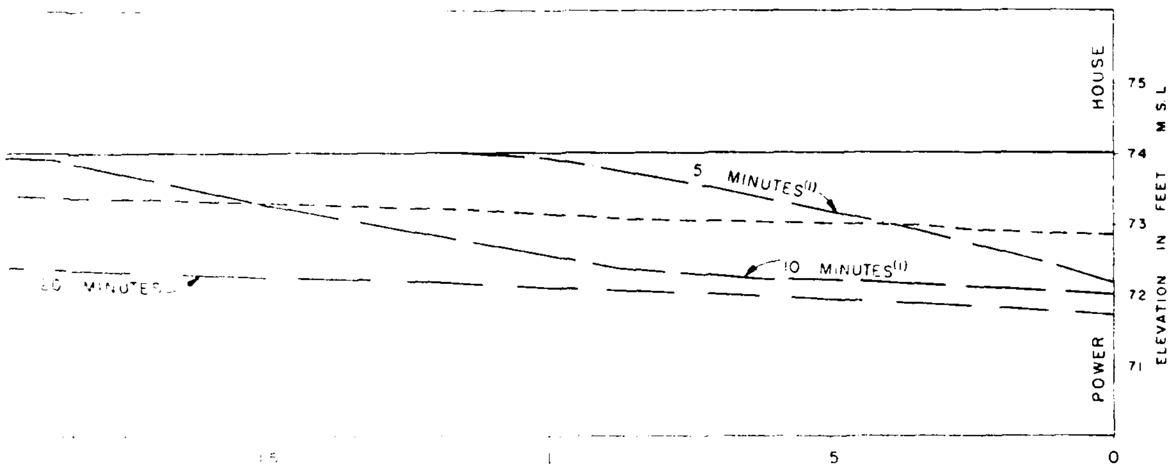
NOTES:

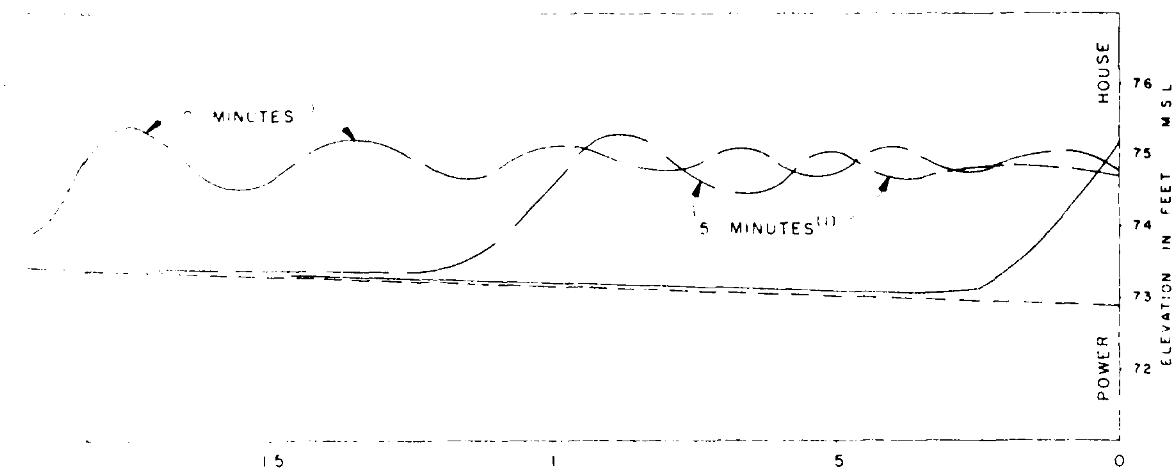
(1)

(2)

(3)

2





NOTES

(1) COMPUTED WATER SURFACE PROFILE USING UNSTEADY FLOW MODEL AT INDICATED TIME FOLLOWING RAPID CLOSURE OR OPENING OF ALL GATES AT THE ST. STEPHEN HYDROPLANT

RAPID GATE OPENING — OPENING OF ALL GATES AT ST. STEPHEN HYDROPLANT IN 5 SECONDS (NO FLOW TO 24,500 CFS IN 5 SECONDS)

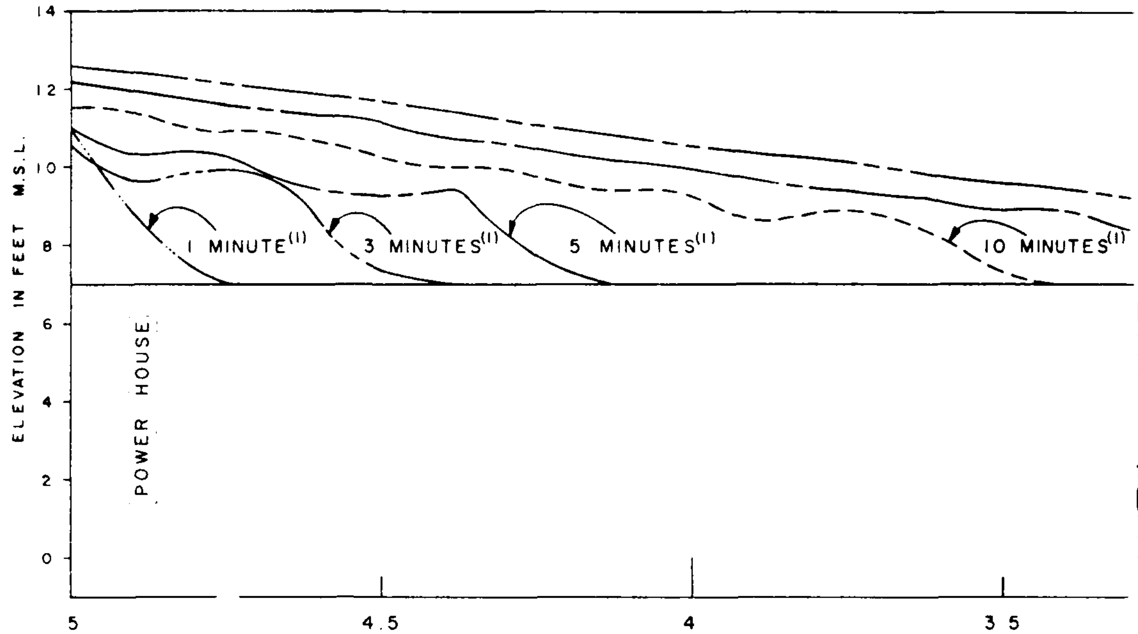
(2) RAPID GATE CLOSURE — CLOSURE OF ALL GATES AT ST. STEPHEN HYDROPLANT IN 5 SECONDS (24,500 CFS TO NO FLOW IN 5 SECONDS)

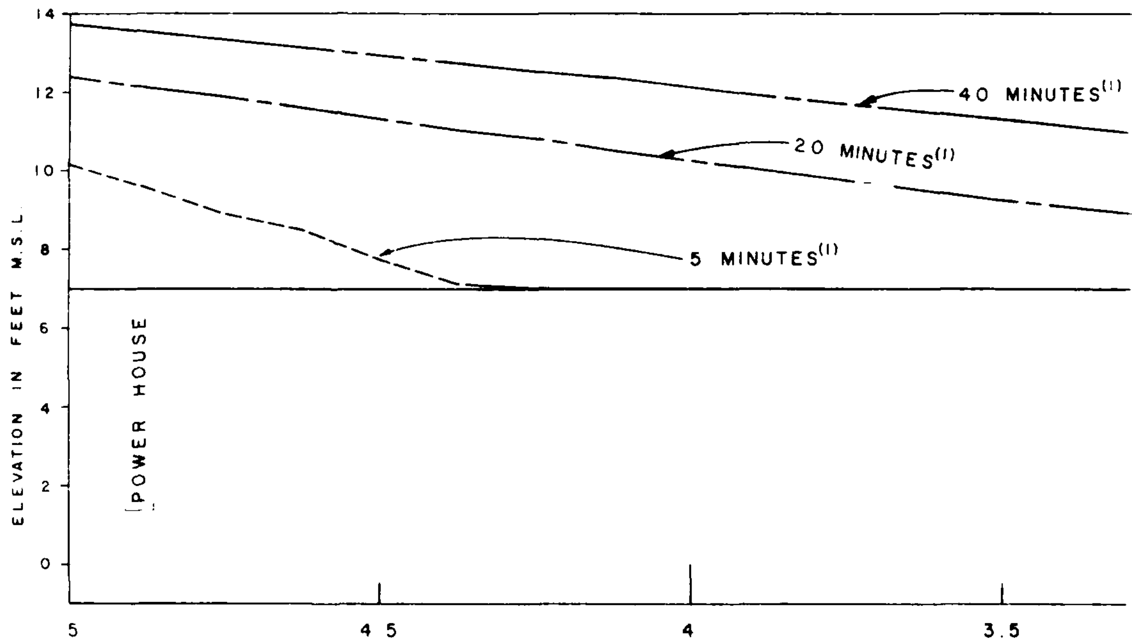
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

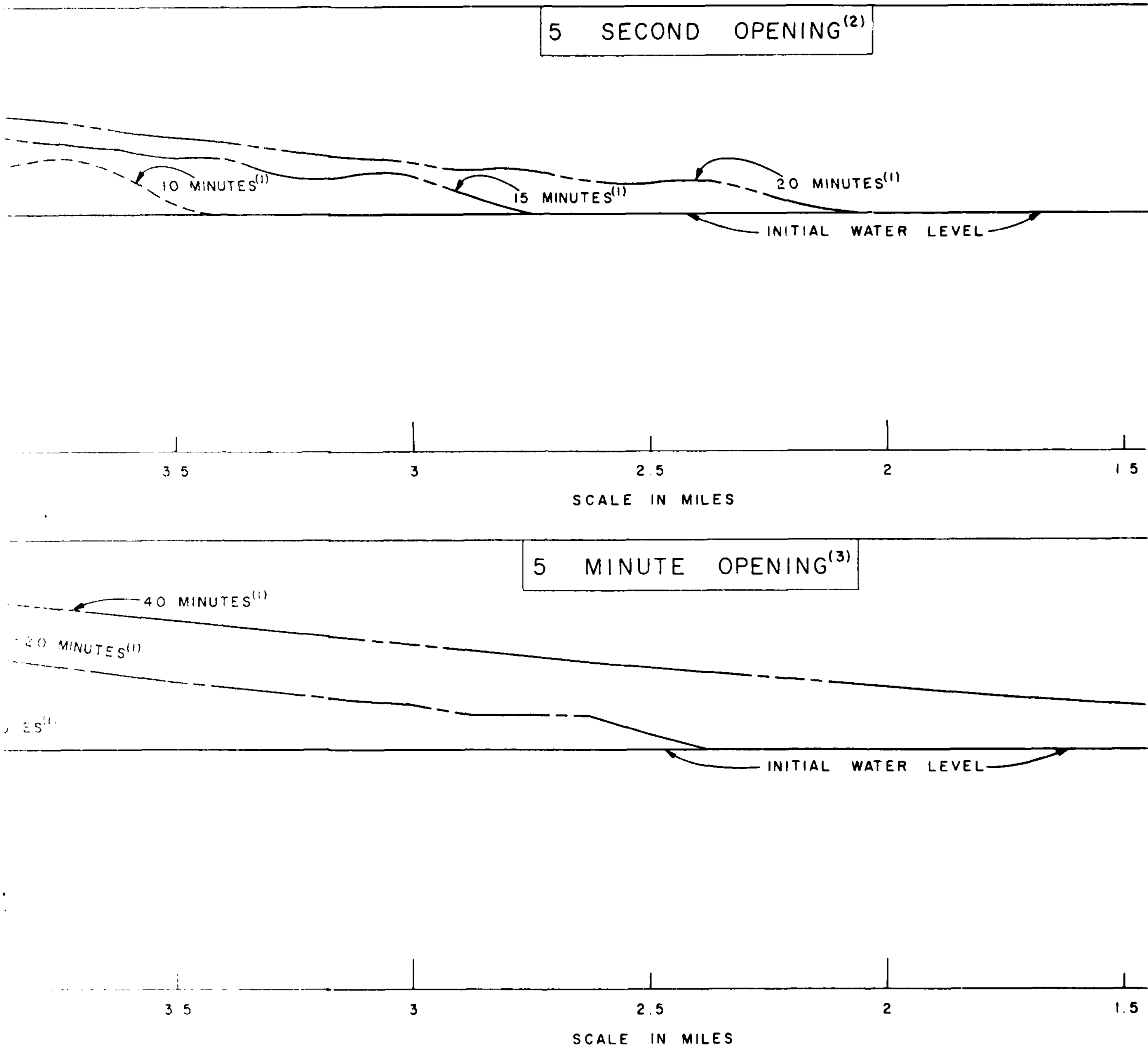
SURGE STUDIES
FOR
INTAKE CANAL
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE: AS SHOWN | INTAKE AND TAILRACE CANALS | PLATE B-20
DATE | FILE NO. 3043

PLATE B-20


ELEVATION IN FEET M.S.L.
POWER HOUSE
1 MINUTE(1)
3 MINUTES(1)
5 MINUTES(1)
10 MINUTES(1)
14
12
10
8
6
4
2
0
5
4.5
4
3.5


ELEVATION IN FEET M.S.L.
POWER HOUSE
40 MINUTES(1)
20 MINUTES(1)
5 MINUTES(1)
14
12
10
8
6
4
2
0
5
4.5
4
3.5



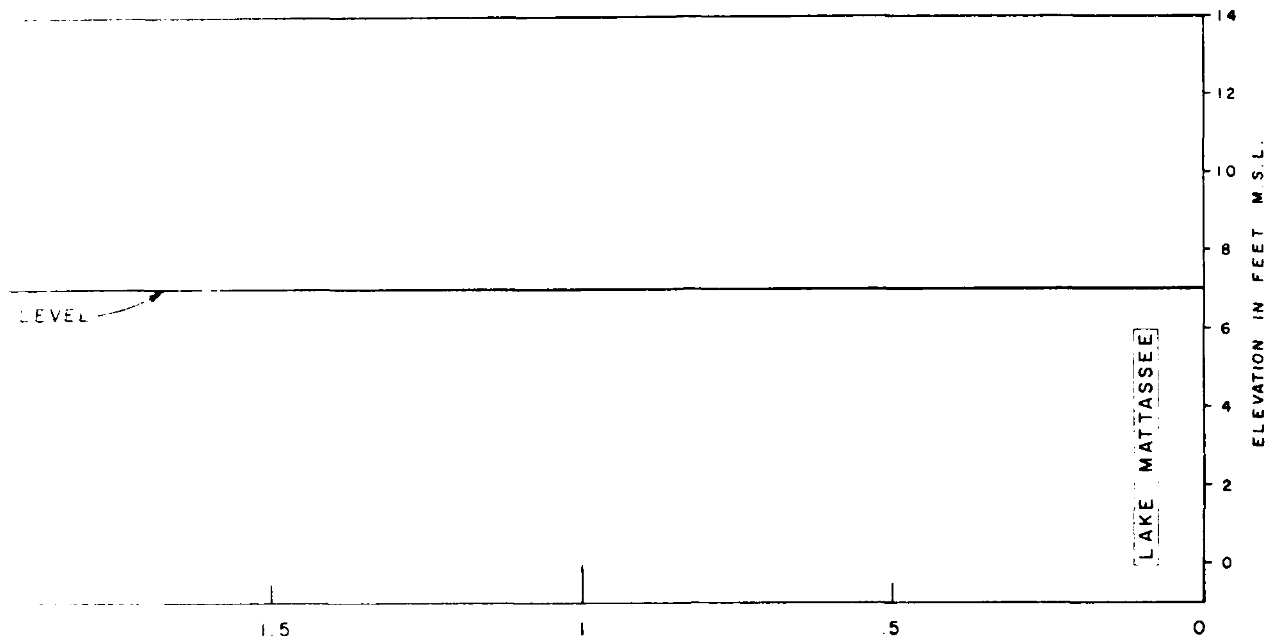
NOTES:

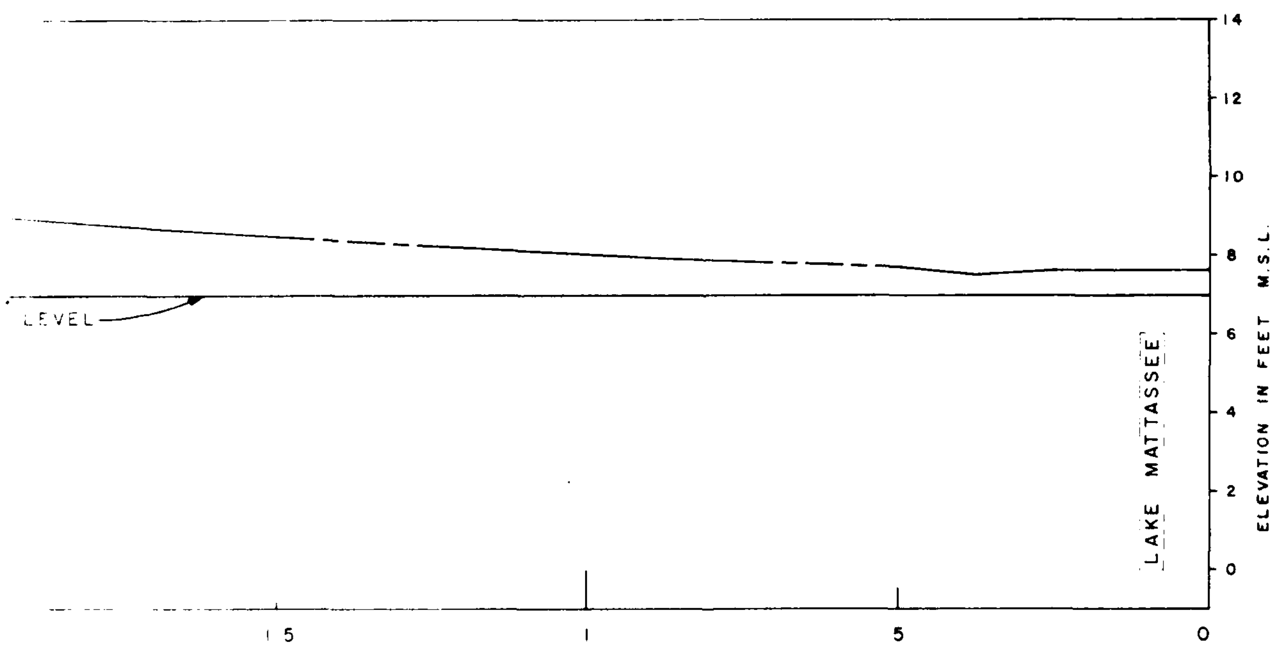
(1) COMPUTED WA
MODEL AT IND
AT THE ST

(2) 5 SECOND OP
STEPHEN HYD
CFS IN 5 S

(3) 5 MINUTE OP
STEPHEN HYD
CFS IN 5

2





NOTES:

(1) COMPUTED WATER SURFACE PROFILE USING UNSTEADY FLOW MODEL AT INDICATED TIME FOLLOWING OPENING OF ALL GATES AT THE ST STEPHEN HYDROPLANT

(2) 5 SECOND OPENING – OPENING OF ALL GATES AT THE ST STEPHEN HYDROPLANT IN 5 SECONDS (NO FLOW TO 24,500 CFS IN 5 SECONDS)

(3) 5 MINUTE OPENING – OPENING OF ALL GATES AT THE ST STEPHEN HYDROPLANT IN 5 MINUTES (NO FLOW TO 24,000 CFS IN 5 MINUTES)

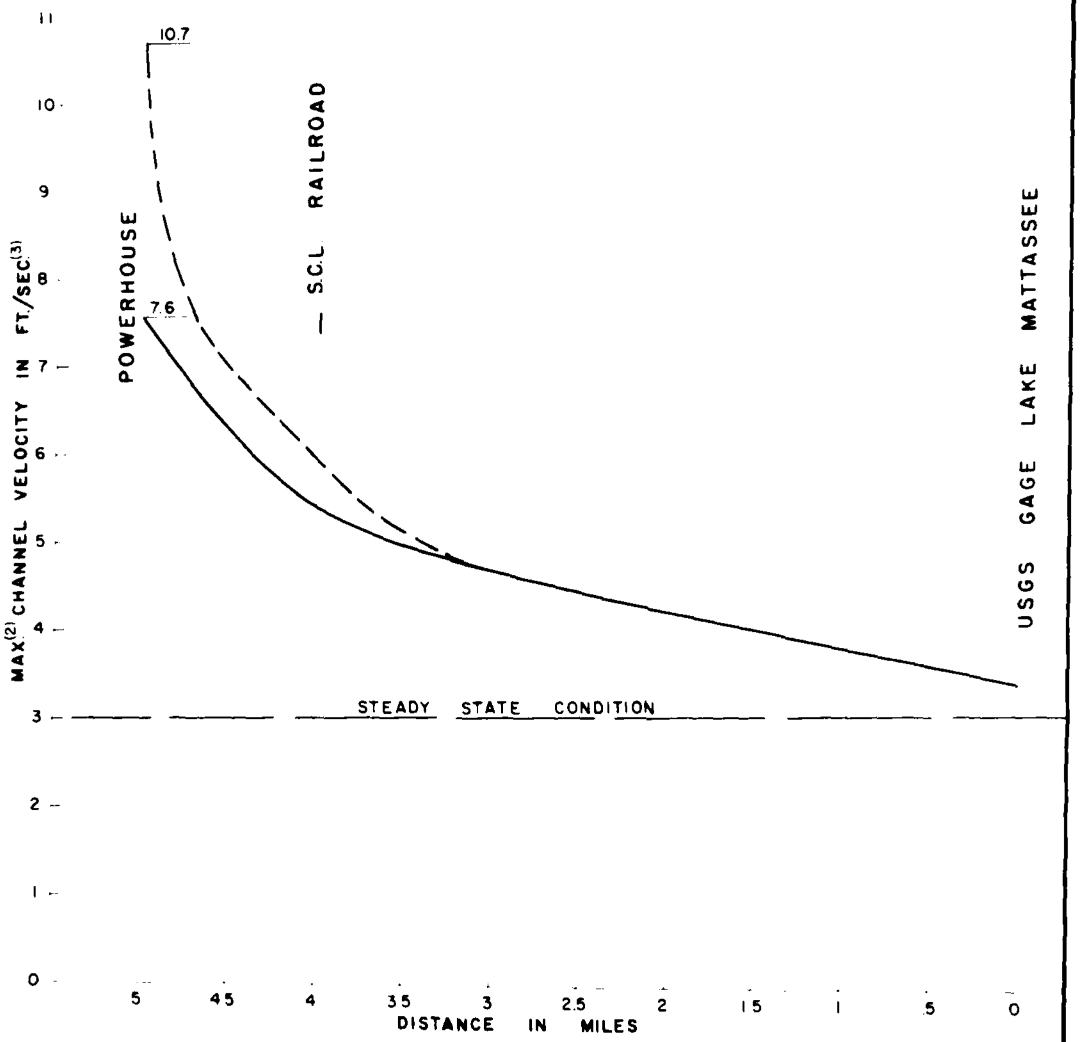
3

U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

SURGE STUDIES FOR TAILRACE CANAL
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B-21	
DATE MAY 1976		FILE NO	10043

PLATE B-21



LEGEND

5 SECOND OPENING (ALL GATES)
NO FLOW TO 24,500 C.F.S.
IN 5 SECONDS AT ST.
STEPHEN HYDROPLANT
(SEE NOTE 1)

5 MINUTE OPENING (ALL GATES)
NO FLOW TO 24,500 CFS
IN 5 MINUTES AT ST.
STEPHEN HYDROPLANT

NOTES:

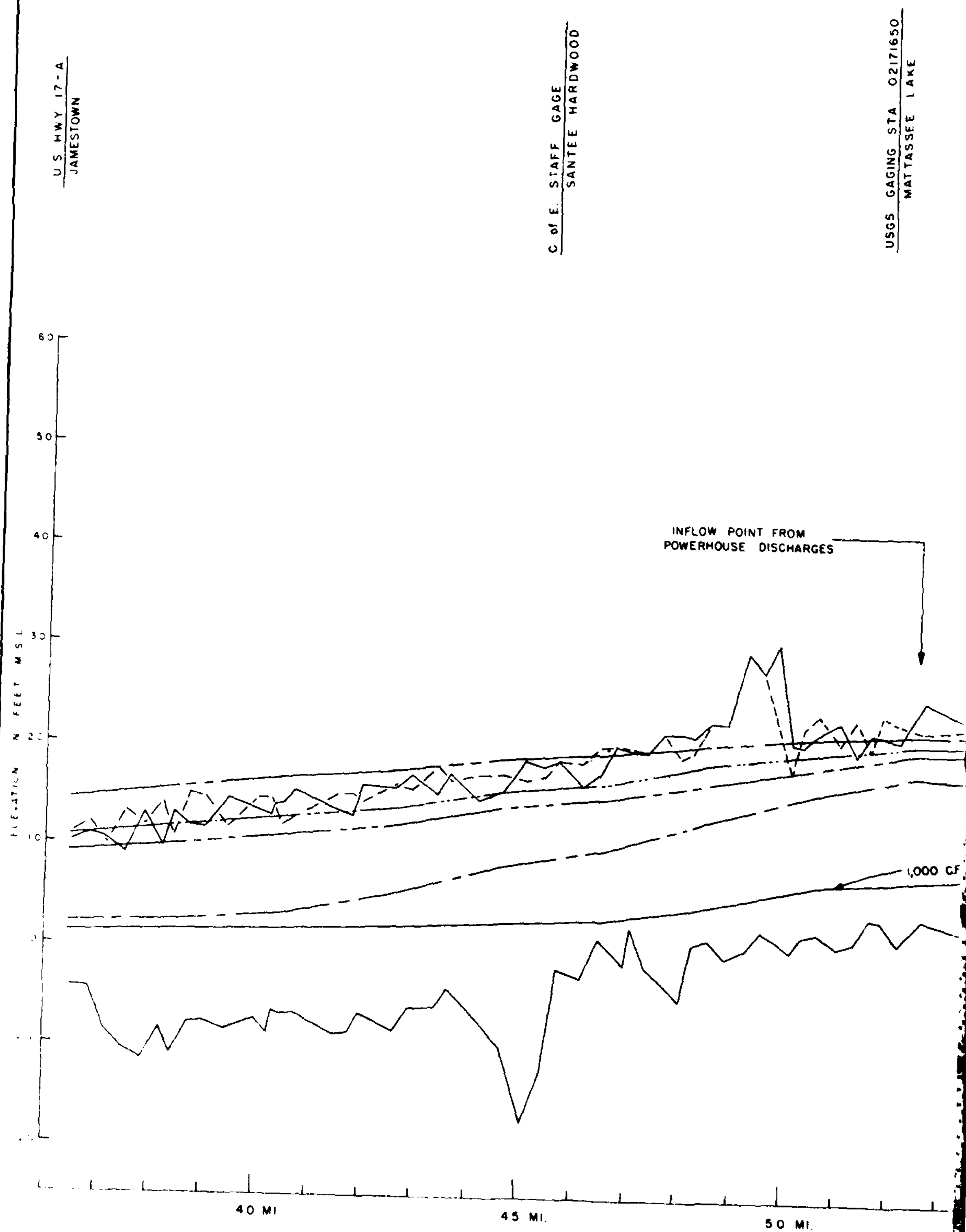
1. THE DASHED CURVE REPRESENTS THE MAXIMUM UPPER LIMIT OF CHANNEL VELOCITIES AS EFFECTED BY RATE OF GATE OPENING
2. MAXIMUM VELOCITY GENERATED AS FLOOD WAVE PROGRESSED DOWNSTREAM.
3. VELOCITIES DETERMINED USING UNSTEADY FLOW MODEL.

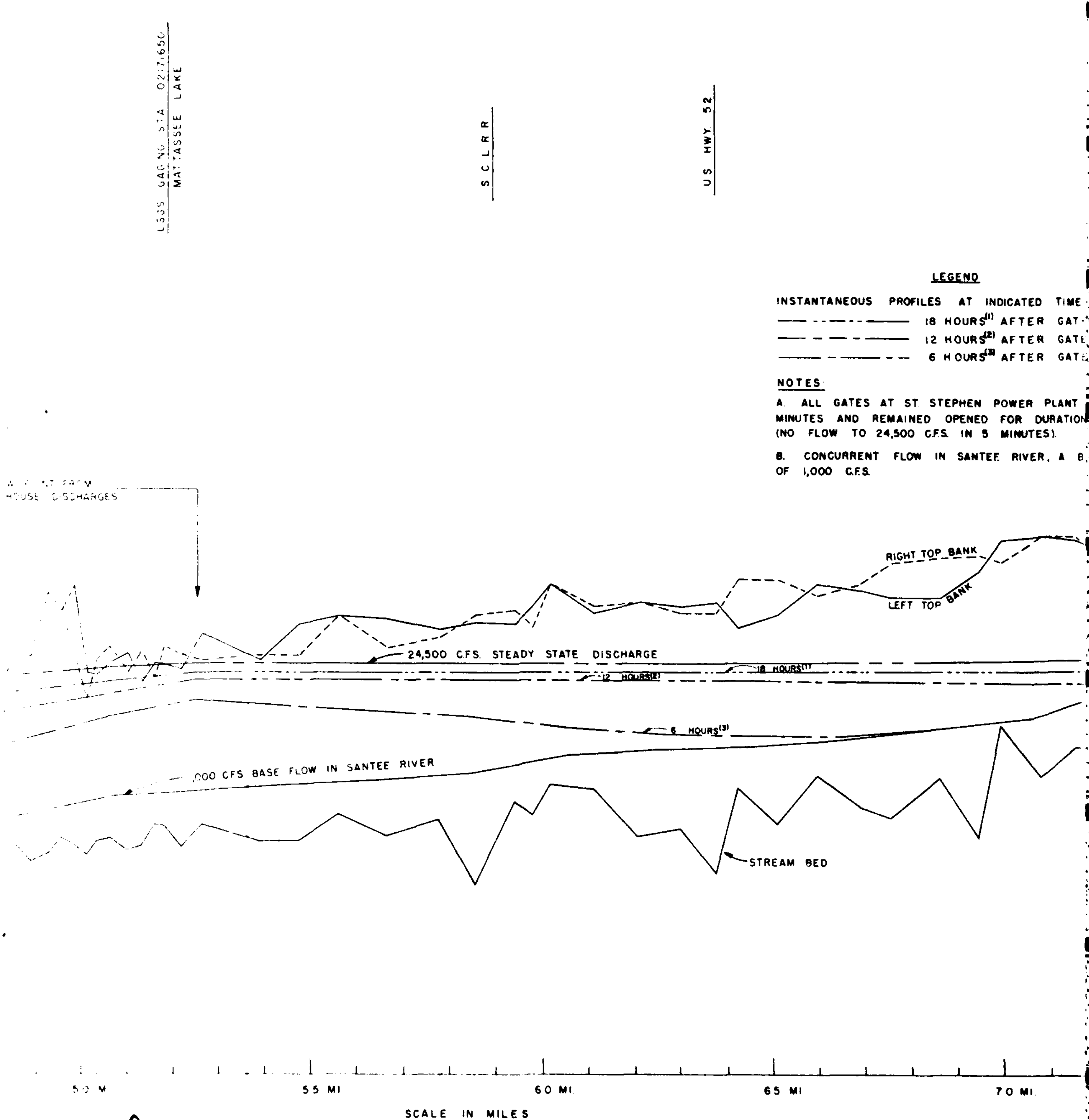
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

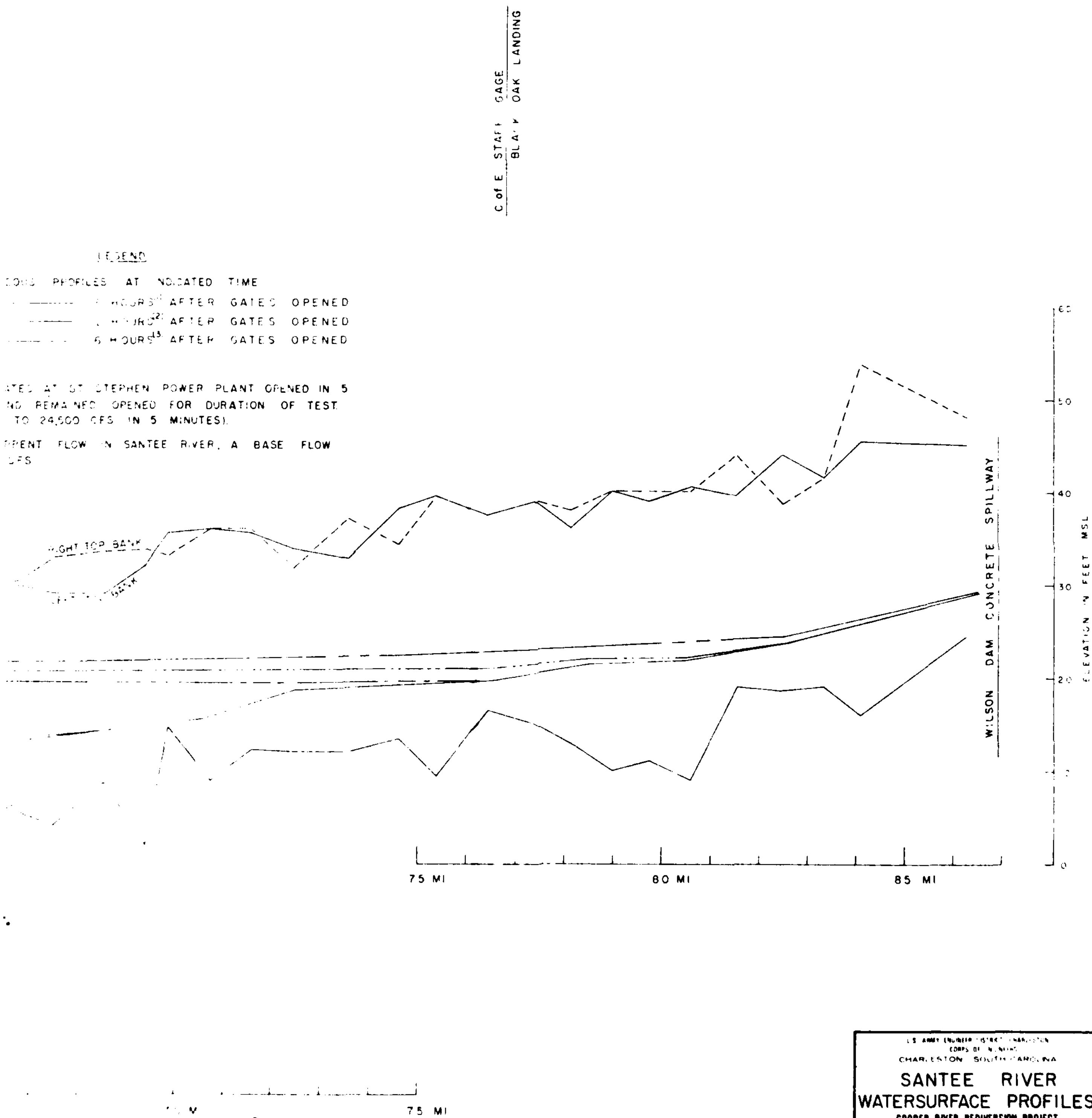
MAXIMUM VELOCITY PROFILE FOR TAILRACE CANAL

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B-[illegible]
DATE MAY 1976		FILE NO [illegible]


U S HWY 17-A
JAMESTOWN
C of E STAFF GAGE
SANTEE HARDWOOD
USGS GAGING STA 02171650
MATTASSEE LAKE
INFLOW POINT FROM
POWERHOUSE DISCHARGES
ELEVATION IN FEET MSL
60
50
40
30
20
10
1,000 CF
40 MI
45 MI.
50 MI.


USGS GAGING STA 02171650
MATTASSEE LAKE
S C L R R
US HWY 52
LEGEND
INSTANTANEOUS PROFILES AT INDICATED TIME
18 HOURS(1) AFTER GAT
12 HOURS(2) AFTER GATE
6 HOURS(3) AFTER GATE
NOTES:
A. ALL GATES AT ST STEPHEN POWER PLANT
MINUTES AND REMAINED OPENED FOR DURATION
(NO FLOW TO 24,500 C.F.S. IN 5 MINUTES).
B. CONCURRENT FLOW IN SANTEE RIVER, A B
OF 1,000 C.F.S.
HOUSE DISCHARGES
RIGHT TOP BANK
LEFT TOP BANK
24,500 C.F.S. STEADY STATE DISCHARGE
18 HOURS(1)
12 HOURS(2)
6 HOURS(3)
,000 CFS BASE FLOW IN SANTEE RIVER
STREAM BED
55 MI
60 MI.
65 MI
70 MI.
SCALE IN MILES



3

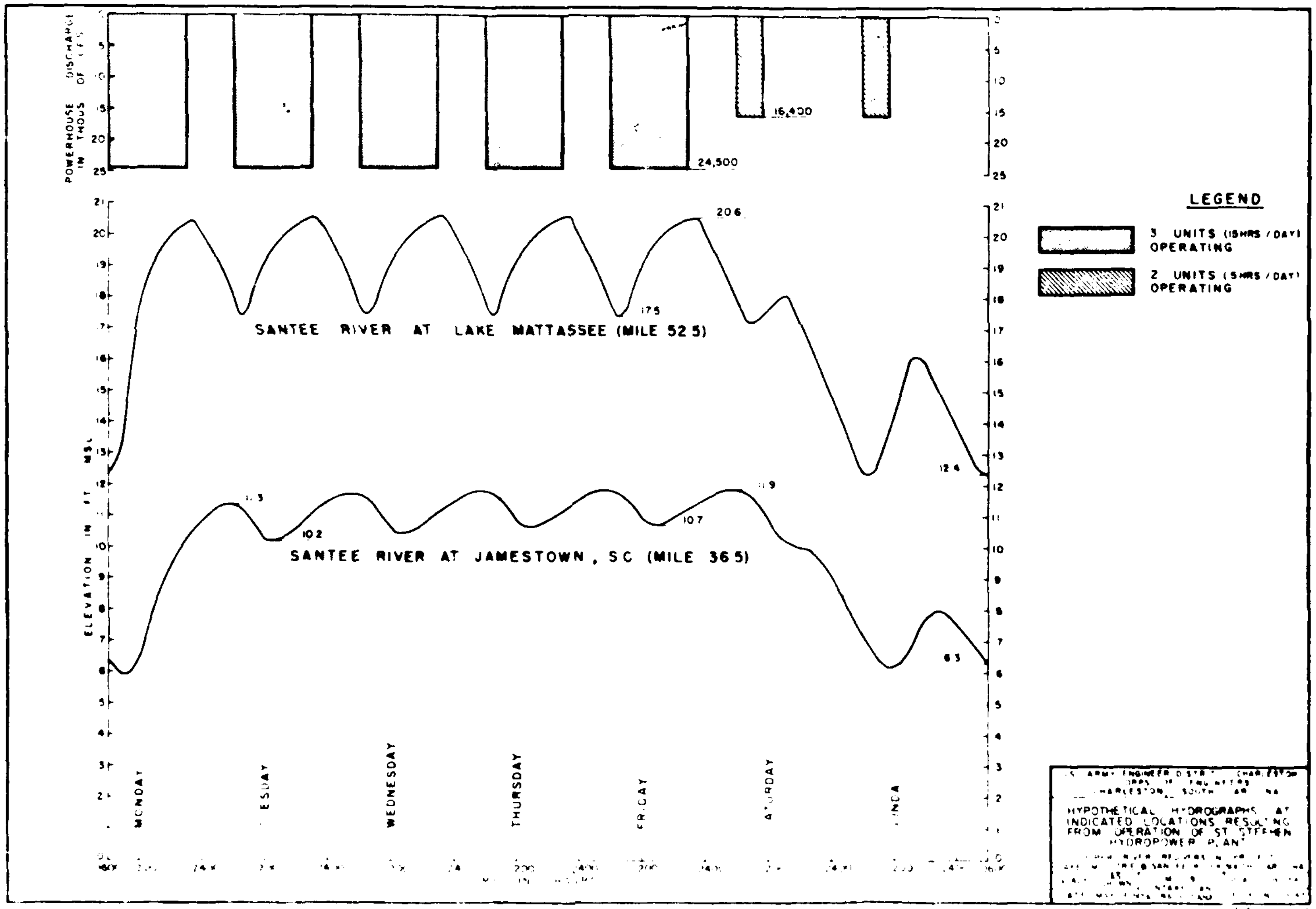
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

SANTEE RIVER
WATERSURFACE PROFILES

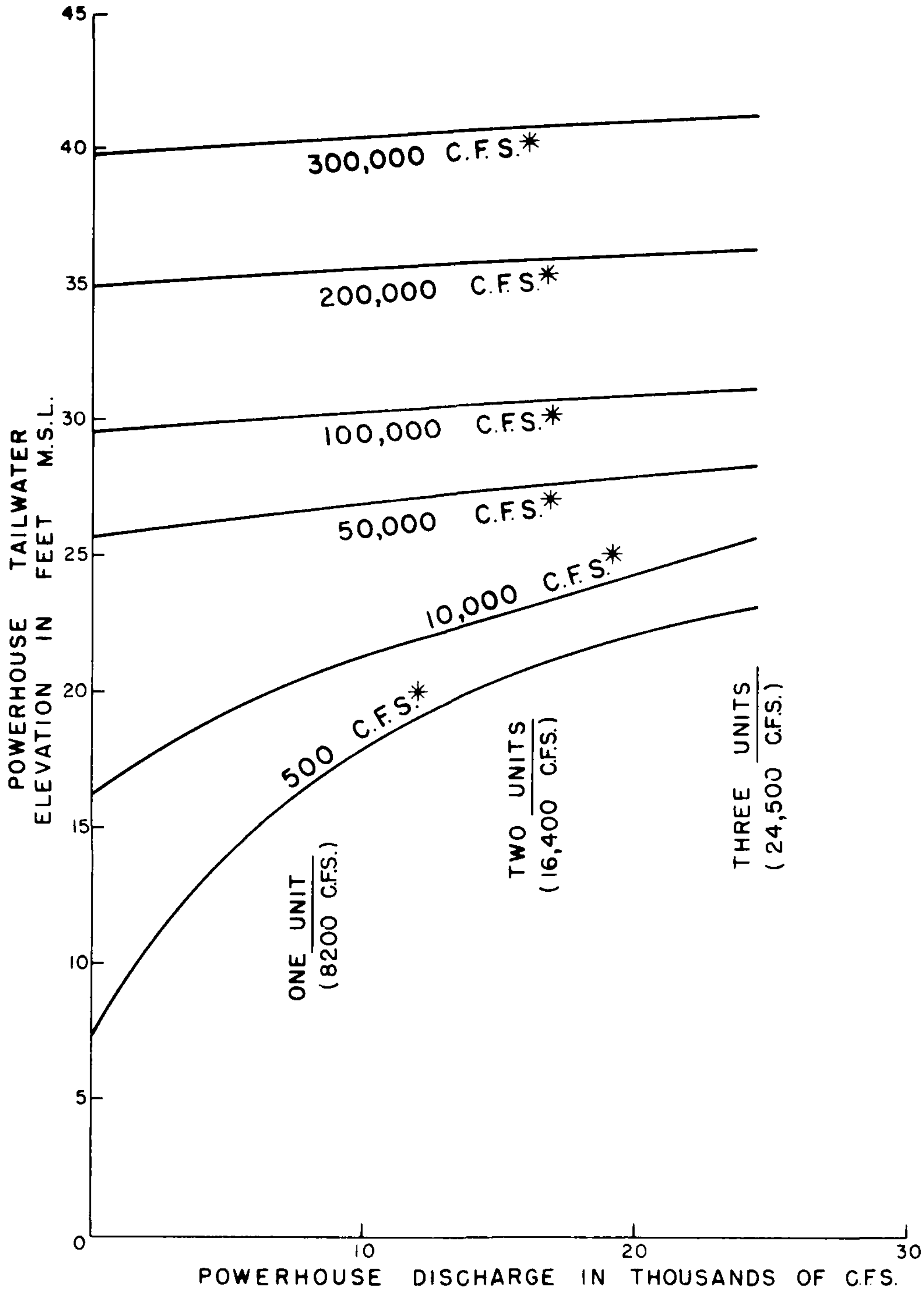
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN — DM — INTAKE AND TAILRACE CANALS — PLATE B-23
DATE MAY 1976 — FILE NO.

PLATE B-23



POWERHOUSE DISCHARGE IN THOUS OF CFS
16,400
24,500
ELEVATION IN FT MSL
206
175
124
SANTEE RIVER AT LAKE MATTASSEE (MILE 52.5)
102
107
63
SANTEE RIVER AT JAMESTOWN, SC (MILE 36.5)
MONDAY
WEDNESDAY
THURSDAY
LEGEND
3 UNITS (15HRS / DAY) OPERATING
2 UNITS (5HRS / DAY) OPERATING
HYPOTHETICAL HYDROGRAPHS AT INDICATED LOCATIONS RESULTING FROM OPERATION OF ST. STEPHEN HYDROPOWER PLANT



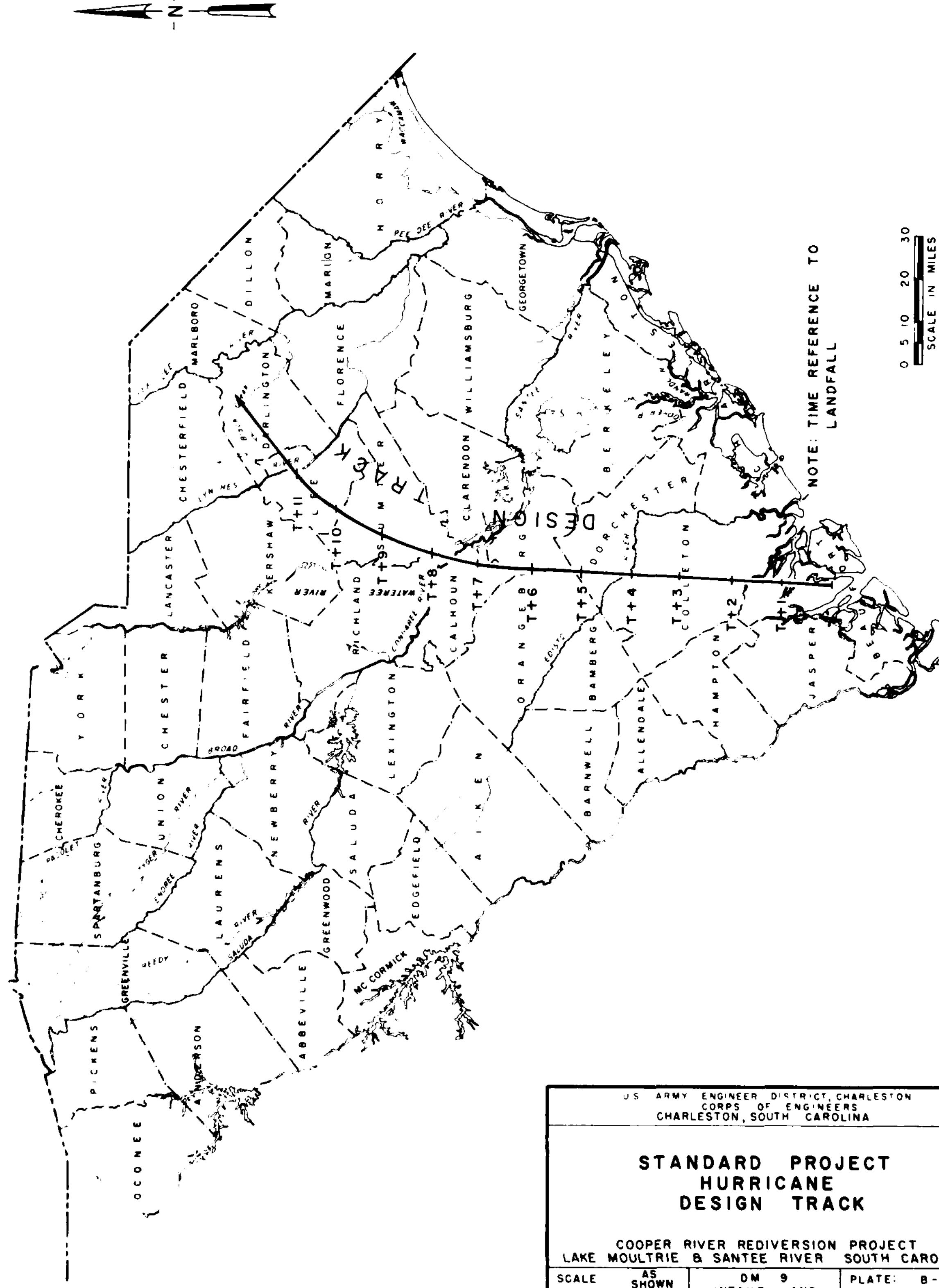
* FLOW IN SANTEE RIVER ABOVE CONFLUENCE OF TAILRACE CANAL

U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

POWERHOUSE TAILWATER RATING CURVES

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER, SOUTH CAROLINA

SCALE AS SHOWN	DM 9	PLATE B-25
DATE MAY 1976	INTAKE AND TAILRACE CANALS	FILE NO. 10043

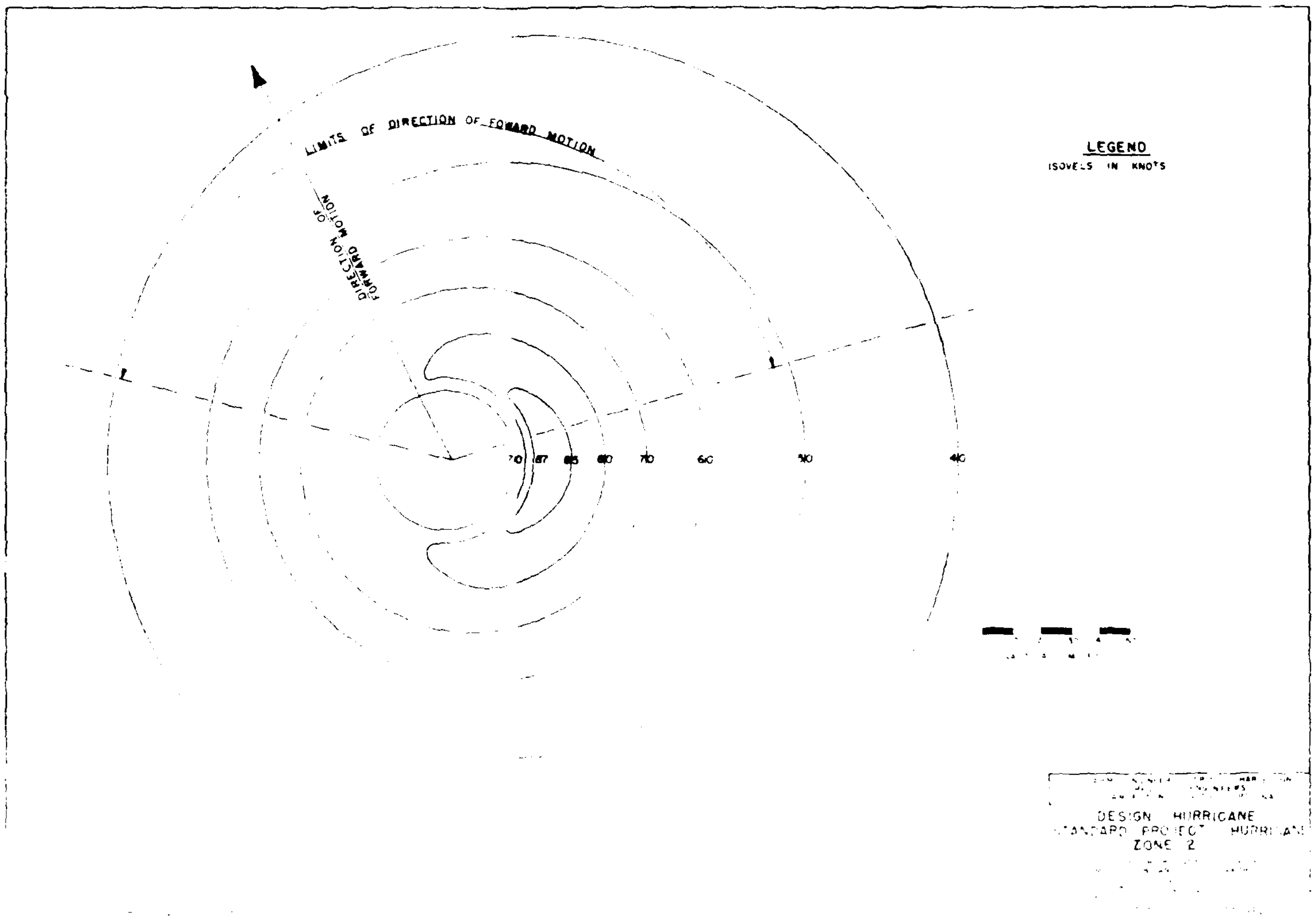


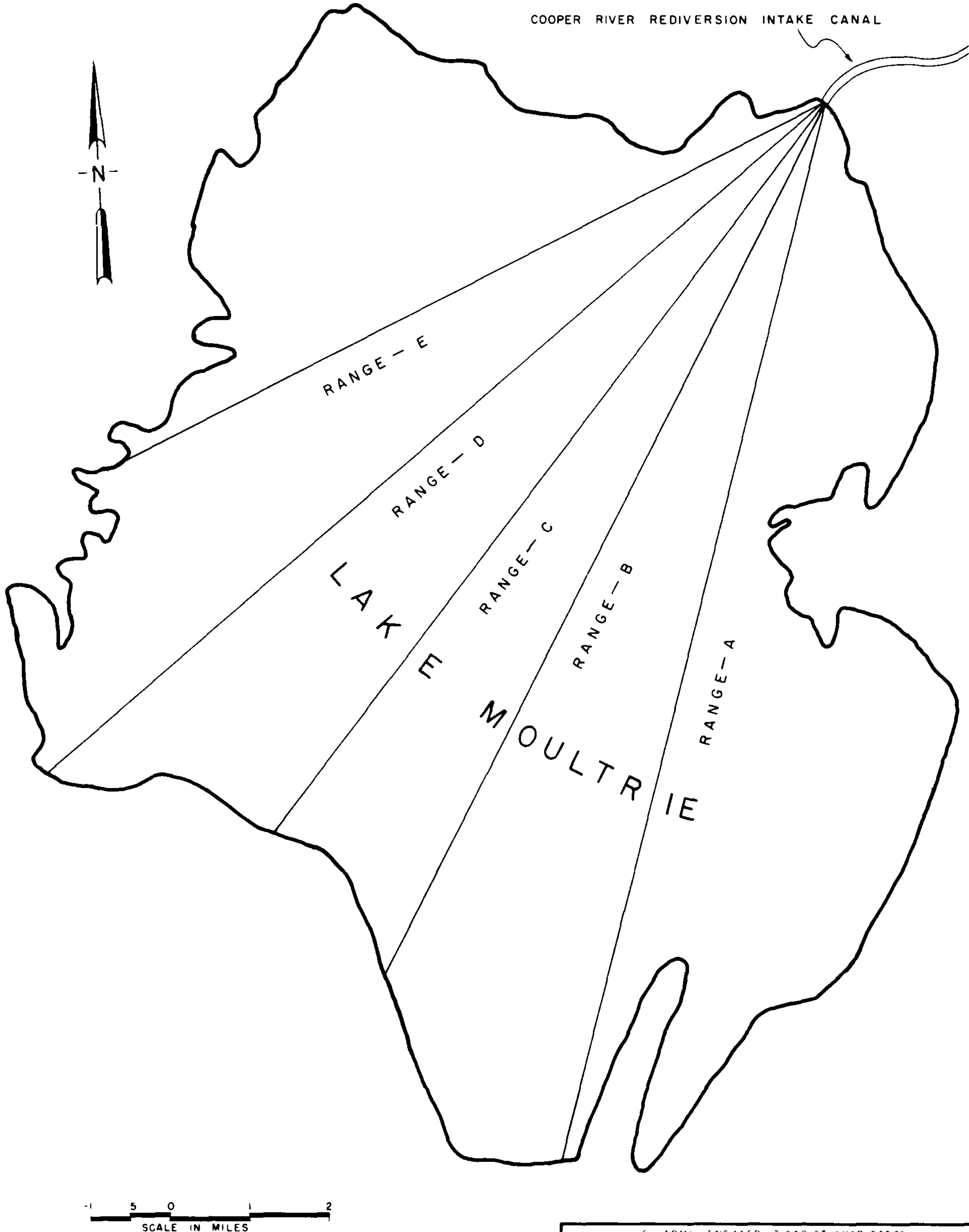
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

STANDARD PROJECT HURRICANE DESIGN TRACK

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE: B-26
DATE: MAY 1976		FILE: 10043


LIMITS OF DIRECTION OF FOWARD MOTION
DIRECTION OF FORWARD MOTION
LEGEND
ISOVELS IN KNOTS
70
87
85
80
70
60
50
40
DESIGN HURRICANE
STANDARD PROJECT HURRICANE
ZONE 2

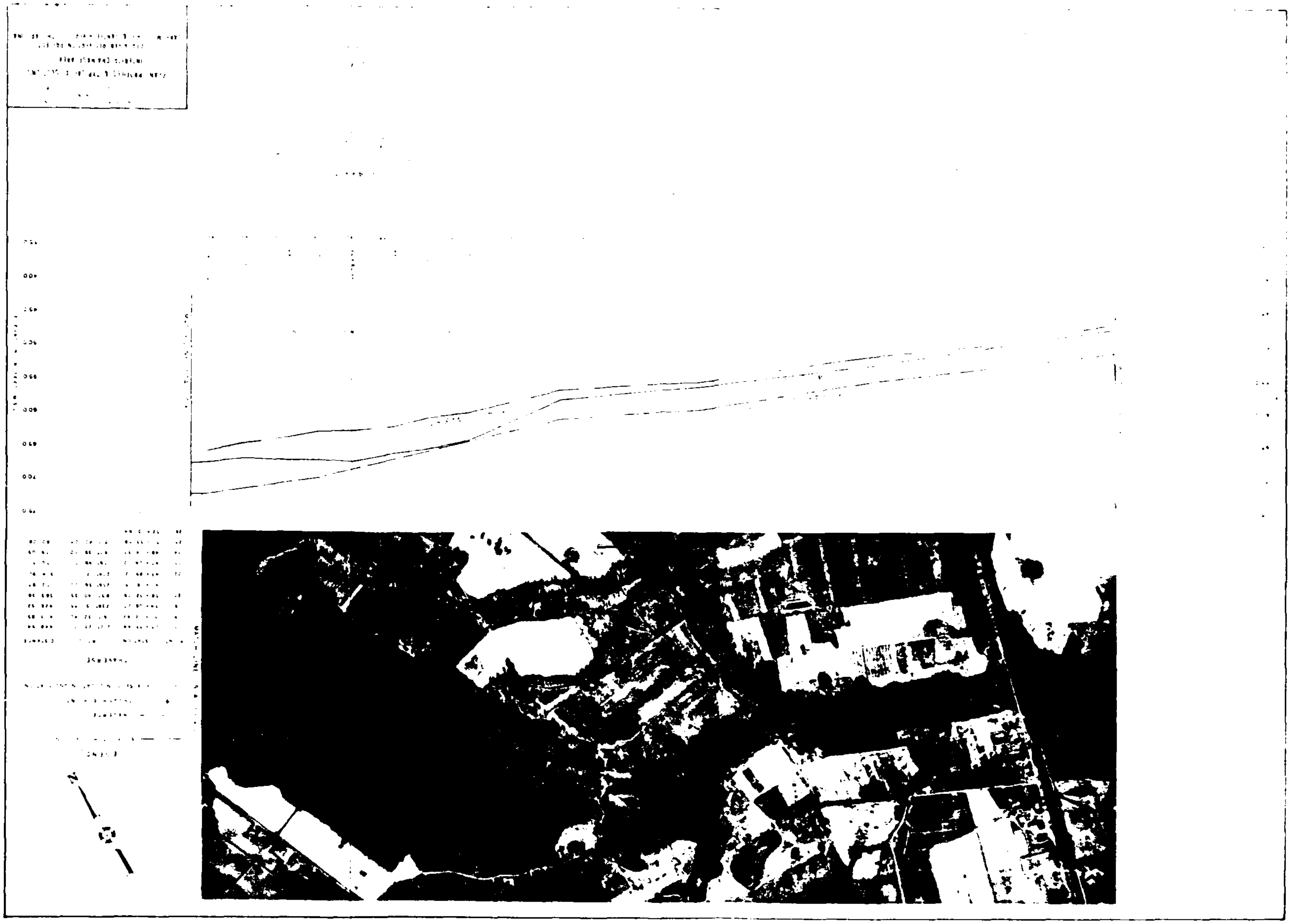


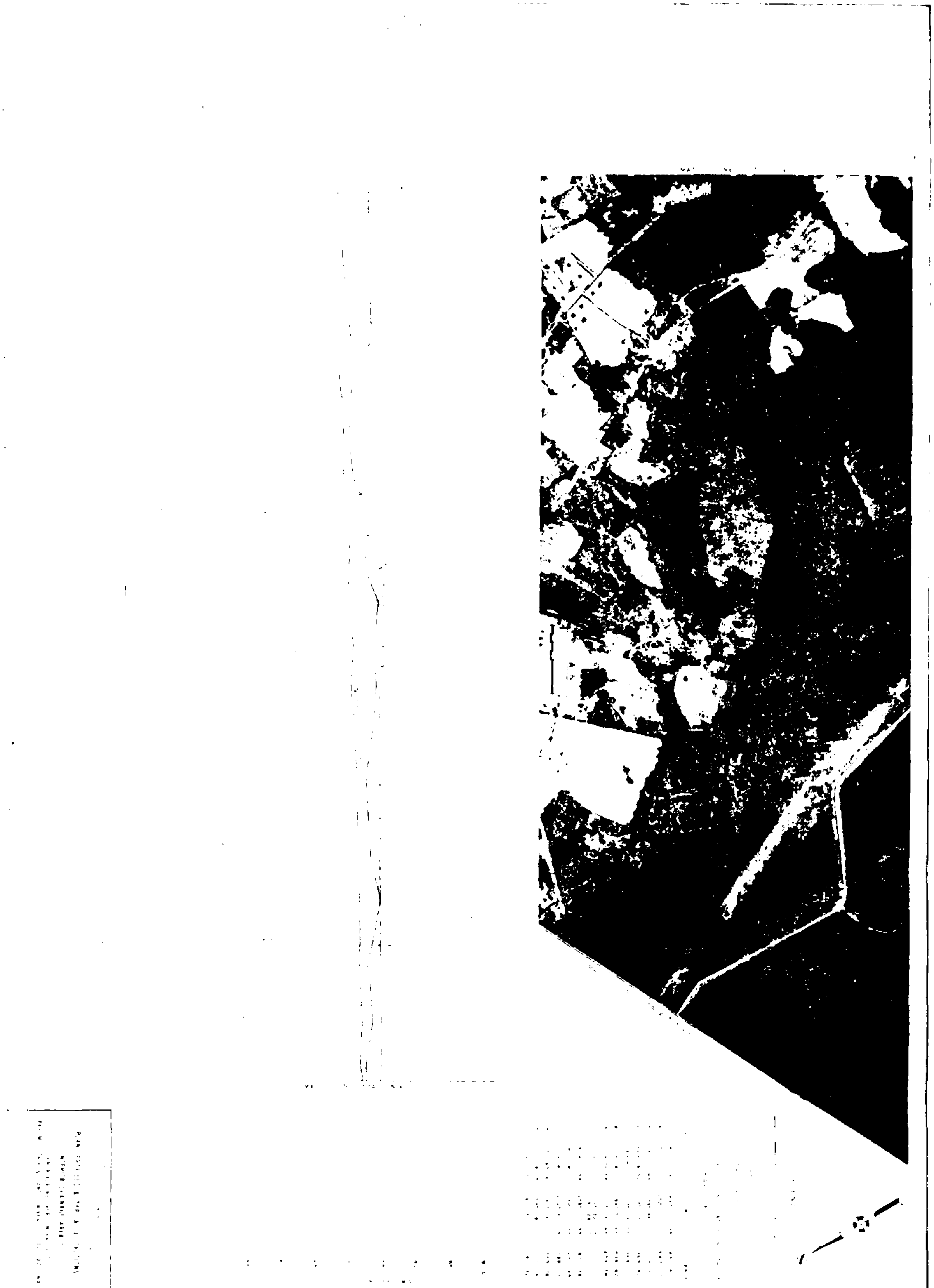
U.S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

RANGES USED IN COMPUTING
WIND SETUP IN
LAKE MOULTRIE

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

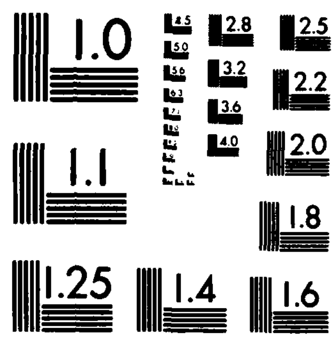
SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE B-28
DATE MAY 1976		FILE 10043



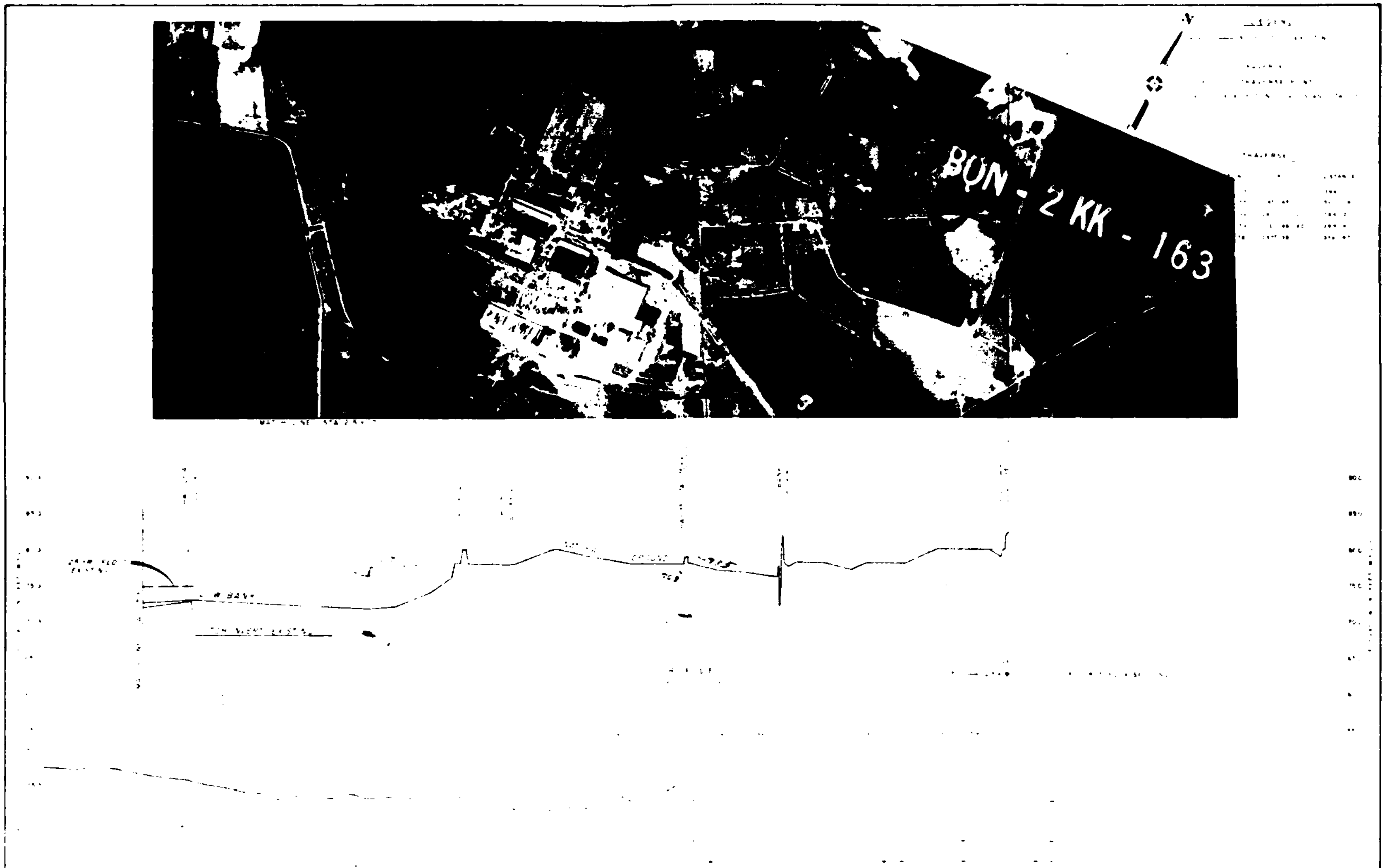


AD-A149 941 COOPER RIVER REDIVERSION PROJECT LAKE MOULTRIE AND SANTEE RIVER SOUTH CAROLINA INTAKE AND TAILRACE CANALS (U) CORPS OF ENGINEERS CHARLESTON SC CHARLESTON DISTRICT JUN 76 4/5

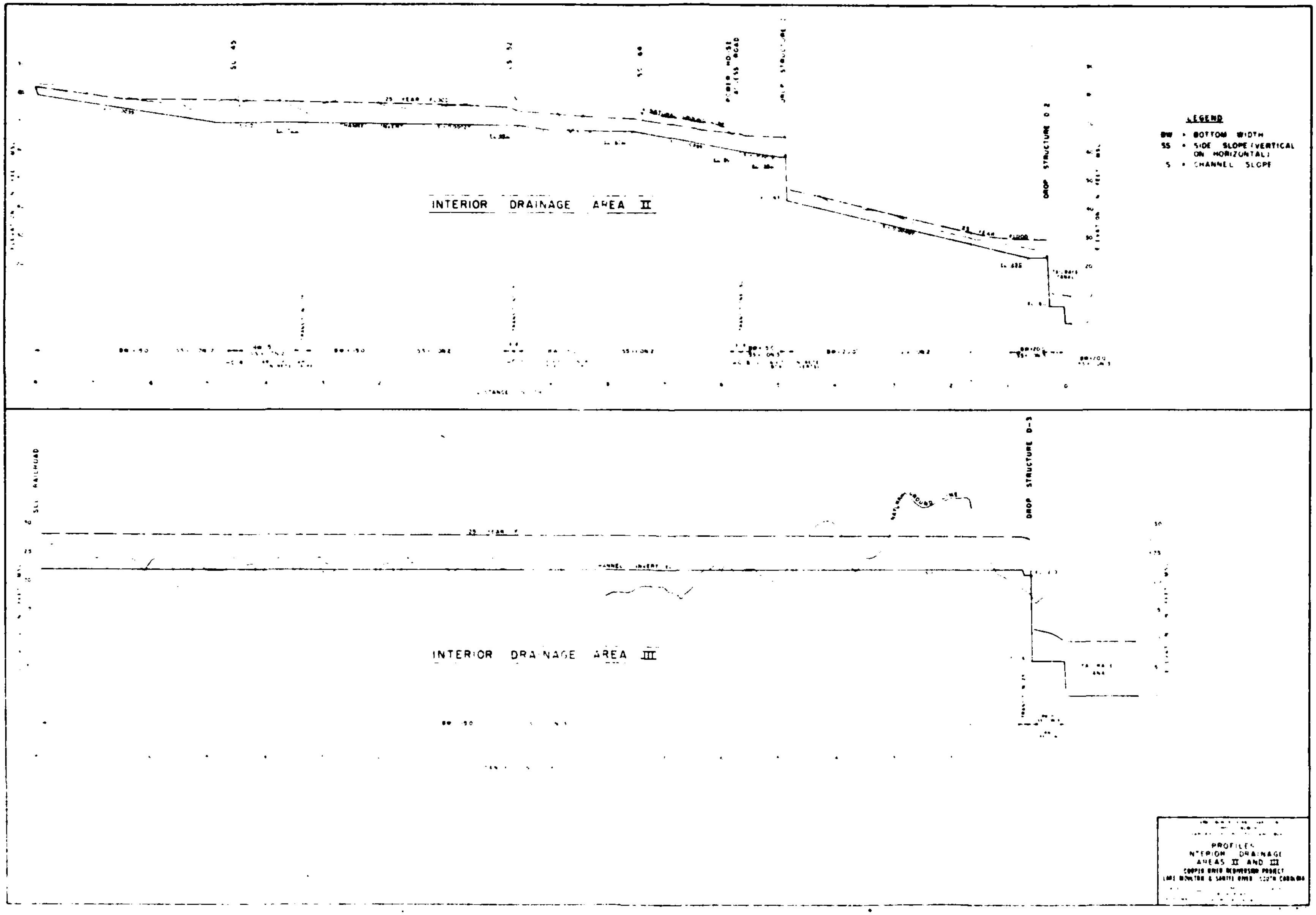
UNCLASSIFIED F/G 13/2 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A



PLAN, PROFILES & TYPICAL X-SECTIONS
INTERIOR DRAINAGE AREA 1


LEGEND
BW = BOTTOM WIDTH
SS = SIDE SLOPE (VERTICAL ON HORIZONTAL)
S = CHANNEL SLOPE
INTERIOR DRAINAGE AREA II
DROP STRUCTURE D-2
INTERIOR DRAINAGE AREA III
DROP STRUCTURE D-3
PROFILES
INTERIOR DRAINAGE
AREAS II AND III
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

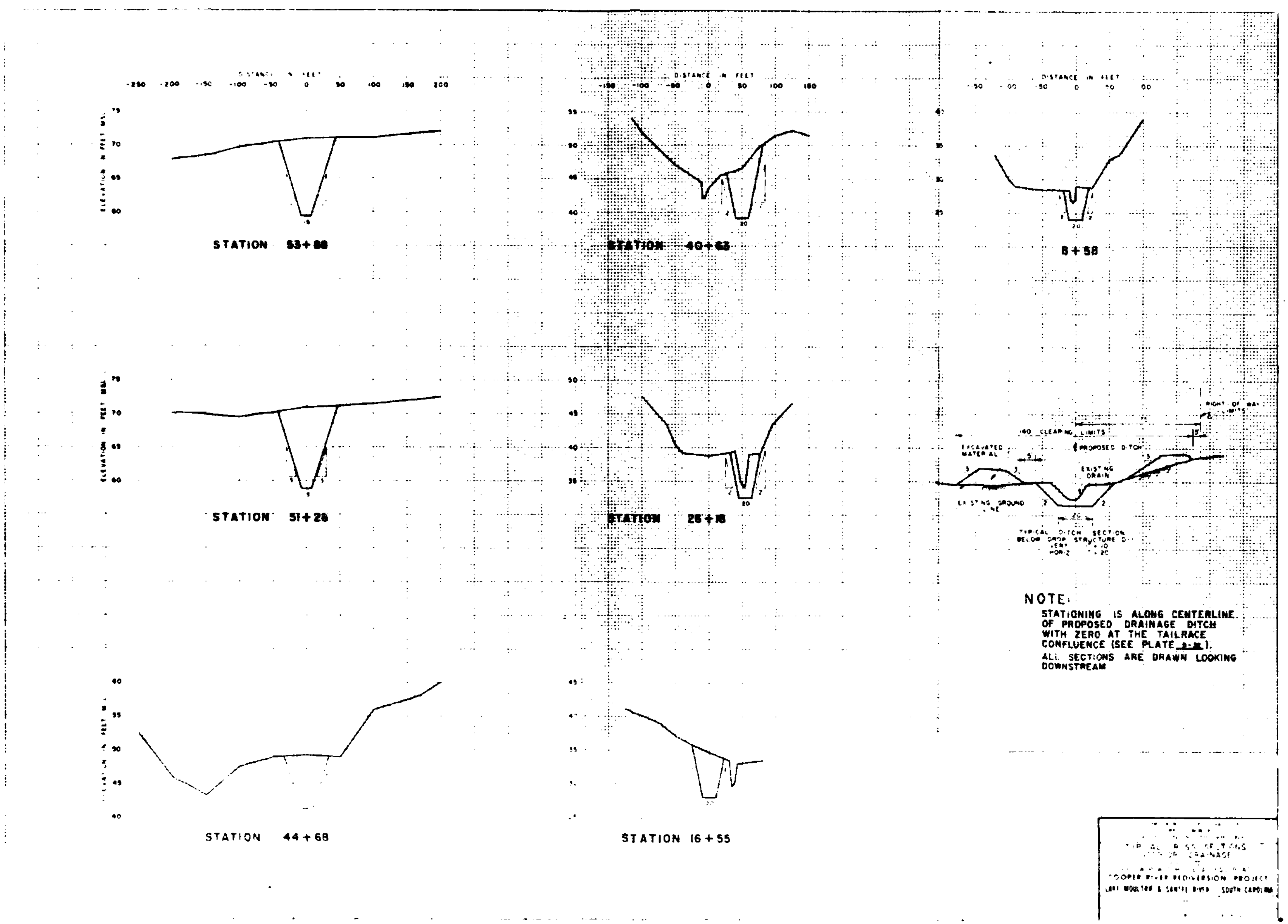

DISTANCE IN FEET
ELEVATION IN FEET MSL
STATION 53+88
STATION 40+63
8+58
STATION 51+28
STATION 26+18
STATION 44+68
STATION 16+55
100 CLEARING LIMITS
RIGHT OF WAY LIMITS
PROPOSED DITCH
EXCAVATED MATERIAL
EXISTING DRAIN
EXISTING GROUND LINE
TYPICAL DITCH SECTION BELOW DROP STRUCTURE D
NOTE:
STATIONING IS ALONG CENTERLINE OF PROPOSED DRAINAGE DITCH WITH ZERO AT THE TAILRACE CONFLUENCE (SEE PLATE).
ALL SECTIONS ARE DRAWN LOOKING DOWNSTREAM
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

Exhibit B-1

Printout Sheets of Operation Studies for St. Stephen and Existing Hydro Facilities

INITIAL STORAGE, 1126.

YEAR 1964	INFLO	TOTAL RELEASE		STOR.	MAR EL.	MOUL EL.	SPILL		STEPH	REL.	STEPH. POWER AVE	PEAK	EN.	PINOP. POWER AVE	PEAK	EN.	SPWY. POWER AVE	PEAK	EN.	TOTAL POWER AVE	PEAK	EN.
	KAF	KAF	KCFS	KAF			KAF	KCFS	KAF	KCFS	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH
1	1518	1229	20.0	1415	74.7	73.3	0	0.0	1014	16.5	60.9	85	45.3	14.4	122	10.7	1.3	2	1.0	76.6	209	57.0
2	1644	1360	24.5	1699	76.5	74.9	0	0.0	1165	21.0	77.9	88	52.3	14.7	127	9.9	1.4	2	1.0	94.0	217	63.1
3	2132	2097	34.1	1734	76.6	75.0	400	6.5	1482	24.1	84.4	85	62.8	14.9	127	11.1	1.4	2	1.0	100.7	214	74.9
4	2750	2742	46.1	1742	76.6	75.1	1136	19.1	1397	23.5	79.0	79	56.9	14.9	128	10.7	1.2	2	0.9	95.1	209	68.5
5	1305	1347	21.9	1700	76.3	75.2	0	0.0	1132	18.4	70.8	90	52.7	14.9	128	11.1	1.5	2	1.1	87.2	220	64.9
6	756	756	12.7	1700	76.0	75.6	0	0.0	548	9.2	37.4	93	26.9	15.0	128	10.8	1.4	2	1.0	53.8	223	38.7
7	925	925	15.0	1700	76.1	75.6	0	0.0	710	11.5	46.4	92	34.6	15.0	128	11.2	1.4	2	1.1	62.9	222	46.8
8	1123	1123	18.3	1700	76.2	75.4	0	0.0	908	14.8	58.3	91	43.4	15.0	128	11.2	1.4	2	1.1	74.7	221	55.6
9	1456	1456	24.5	1700	76.4	75.0	0	0.0	1248	21.0	79.7	89	57.4	14.9	127	10.8	1.4	2	1.0	96.1	218	69.2
10	3345	3301	53.7	1744	76.8	75.1	1667	27.1	1419	23.1	75.6	76	56.2	14.9	128	11.1	1.1	2	0.8	91.6	205	68.1
11	1225	1408	23.7	1561	75.6	74.3	0	0.0	1200	20.2	76.2	87	54.9	14.8	125	10.7	1.4	2	1.0	92.5	214	66.6
12	1438	1532	24.9	1466	75.1	73.5	0	0.0	1317	21.4	79.0	84	58.8	14.7	123	10.9	1.4	2	1.1	95.1	209	70.8
SUM	19617	19277					3203		13540				602.0			130.1			12.1			744.2

YEAR 1965	INFLO	TOTAL RELEASE		STOR.	MAR EL.	MOUL EL.	SPILL		STEPH	REL.	STEPH. POWER AVE	PEAK	EN.	PINOP. POWER AVE	PEAK	EN.	SPWY. POWER AVE	PEAK	EN.	TOTAL POWER AVE	PEAK	EN.
	KAF	KAF	KCFS	KAF			KAF	KCFS	KAF	KCFS	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH
1	1870	1691	27.5	1645	76.4	74.4	0	0.0	1476	24.0	85.3	86	63.5	14.7	125	10.9	1.4	2	1.1	101.4	213	75.5
2	1465	1410	25.4	1700	76.5	74.9	0	0.0	1216	21.9	81.8	88	55.0	14.8	127	10.0	1.5	2	1.0	98.1	217	65.9
3	2116	2082	33.9	1734	76.8	75.0	384	6.2	1483	24.1	84.6	85	62.9	14.9	127	11.1	1.4	2	1.0	100.8	214	75.0
4	1870	1869	31.4	1735	76.8	75.0	218	3.7	1443	24.2	85.8	86	61.8	14.9	127	10.7	1.4	2	1.0	102.1	215	73.5
5	1082	1117	18.2	1700	76.2	75.4	0	0.0	902	14.7	57.6	91	42.9	14.9	128	11.1	1.5	2	1.1	74.0	221	55.1
6	1407	1407	23.6	1700	76.4	75.1	0	0.0	1199	20.1	76.9	89	55.4	15.0	126	10.8	1.4	2	1.0	93.3	219	67.2
7	1187	1187	19.3	1700	76.2	75.3	0	0.0	972	15.8	61.7	91	45.9	14.9	128	11.1	1.4	2	1.1	78.1	221	58.1
8	1106	1106	18.0	1700	76.2	75.4	0	0.0	891	14.5	57.2	91	42.5	15.0	128	11.1	1.4	2	1.1	73.6	221	54.8
9	748	908	15.3	1540	75.1	74.6	0	0.0	699	11.8	46.7	90	33.6	14.9	126	10.7	1.4	2	1.0	63.1	218	45.4
10	745	890	14.5	1395	74.2	73.7	0	0.0	675	11.0	43.1	87	32.1	14.7	123	10.9	1.4	2	1.0	59.2	212	44.0
11	662	800	13.5	1257	73.2	72.8	0	0.0	592	10.0	38.6	85	27.8	14.5	120	10.4	1.4	2	1.0	54.4	207	39.2
12	509	710	11.6	1055	71.7	71.3	0	0.0	495	8.1	30.8	81	22.9	14.3	116	10.6	1.3	2	1.0	46.4	199	34.5
SUM	14767	15178					602		12042				546.3			129.6			12.4			688.2

YEAR 1966	INFLO	TOTAL RELEASE		STOR.	MAR EL.	MOUL EL.	SPILL		STEPH	REL.	STEPH. POWER AVE	PEAK	EN.	PINOP. POWER AVE	PEAK	EN.	SPWY. POWER AVE	PEAK	EN.	TOTAL POWER AVE	PEAK	EN.
	KAF	KAF	KCFS	KAF			KAF	KCFS	KAF	KCFS	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH	MW	MW	MKWH
1	783	692	11.3	1146	72.4	72.0	0	0.0	477	7.8	29.5	83	21.9	14.2	118	10.5	1.3	2	1.0	44.9	203	33.4
2	1431	1039	18.7	1538	75.4	74.2	0	0.0	844	15.2	57.0	88	38.3	14.5	125	9.7	1.4	2	0.9	72.9	215	49.0
3	2145	1961	31.9	1722	76.8	74.8	256	4.2	1488	24.2	84.6	85	63.0	14.8	127	11.0	1.4	2	1.0	100.8	214	75.0
4	617	639	10.7	1700	76.0	75.7	0	0.0	431	7.2	29.6	94	21.3	15.0	128	10.8	1.4	2	1.0	46.0	224	33.1
5	609	609	9.9	1700	75.9	75.7	0	0.0	394	6.4	26.5	94	19.7	15.1	128	11.2	1.4	2	1.1	43.0	224	32.0
6	631	631	10.6	1700	76.0	75.7	0	0.0	423	7.1	29.3	94	21.1	15.1	128	10.8	1.4	2	1.0	45.8	224	33.0
7	526	584	9.5	1642	75.6	75.4	0	0.0	369	6.0	24.8	93	18.5	15.0	128	11.2	1.4	2	1.1	41.3	223	30.7
8	508	633	10.3	1517	74.9	74.7	0	0.0	418	6.8	27.7	91	20.6	14.9	126	11.1	1.4	2	1.0	44.1	219	32.8
9	561	674	11.3	1424	74.3	74.0	0	0.0	466	7.8	31.3	89	22.6	14.7	124	10.6	1.4	2	1.0	47.5	215	34.2
10	628	734	11.9	1317	73.6	73.2	0	0.0	519	8.4	33.2	86	24.7	14.6	122	10.9	1.4	2	1.0	49.2	210	36.6
11	727	792	13.3	1252	73.2	72.7	0	0.0	584	9.8	37.8	85	27.2	14.5	120	10.4	1.3	2	1.0	53.6	207	38.6
12	727	853	13.9	1176	72.3	71.7	0	0.0	638	10.4	39.3	82	29.2	14.3	117	10.6	1.3	2	1.0	54.9	201	40.9
SUM	9913	9842					256		7050				328.2			128.9			12.1			469.2

YEAR 1947	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR EL.	MOUL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP.POWER AVE MW	PINOP.POWER PEAK MW	PINOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1067	942	15.3	1271	75.5	72.7		0.0	727	11.8	40.4	84	33.0	14.5	120	10.6	1.3	2	1.0	60.0	206	44.7
2	1090	901	16.2	1469	74.6	74.0	0	0.0	706	12.7	48.6	88	32.7	14.5	124	9.8	1.4	2	0.9	64.5	214	43.4
3	823	675	11.0	1617	75.5	75.2	0	0.0	460	7.5	30.1	92	22.4	14.8	126	11.0	1.4	2	1.0	46.3	222	34.5
4	400	367	6.5	1633	75.5	75.4	0	0.0	179	3.0	12.5	94	9.0	15.0	128	10.8	1.4	2	1.0	28.9	224	20.8
5	468	426	6.9	1673	75.7	75.6	0	0.0	210	3.4	14.3	95	10.7	15.0	128	11.2	1.4	2	1.1	30.8	225	22.9
6	550	532	8.9	1700	75.9	75.8	0	0.0	324	5.4	22.6	95	16.3	15.0	128	10.8	1.4	2	1.0	39.1	225	28.1
7	801	801	13.0	1700	76.0	75.6	0	0.0	566	9.5	38.8	93	28.9	15.0	128	11.2	1.4	2	1.1	55.3	223	41.1
8	1719	1662	27.4	1737	76.6	75.0	0	0.0	1467	23.9	88.5	88	65.9	15.0	127	11.1	1.4	2	1.1	104.9	217	78.1
9	1076	1151	19.3	1662	76.0	75.1	0	0.0	943	15.8	61.7	90	44.4	14.9	128	10.7	1.4	2	1.0	78.1	220	56.2
10	658	913	14.9	1407	74.2	73.8	0	0.0	698	11.4	44.7	87	33.3	14.8	123	11.0	1.4	2	1.0	60.9	213	45.3
11	678	819	13.8	1266	73.3	72.8	0	0.0	610	10.3	39.7	85	28.6	14.5	120	10.5	1.4	2	1.0	55.6	207	40.1
12	1330	1264	20.6	1332	74.1	72.8	0	0.0	1049	17.1	63.0	83	46.9	14.4	120	10.7	1.4	2	1.0	78.8	206	58.6
SUM	10698	10493					0		1959				472.0			129.5			12.3			513.7

YEAR 1948	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR EL.	MOUL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP.POWER AVE MW	PINOP.POWER PEAK MW	PINOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	2351	1975	32.1	1706	76.8	74.6	274	4.5	1486	24.2	82.9	85	61.7	14.6	126	10.9	1.4	2	1.0	98.9	213	73.6
2	940	1086	19.6	1562	75.4	74.6	0	0.0	891	16.1	61.9	88	41.6	14.8	126	10.0	1.4	2	1.0	78.1	216	52.5
3	1122	984	16.0	1700	76.1	75.5	0	0.0	769	12.5	49.5	92	36.8	14.9	128	11.1	1.4	2	1.1	65.8	222	49.0
4	707	707	11.9	1700	76.0	75.7	0	0.0	499	8.4	34.2	94	24.7	15.0	128	10.8	1.4	2	1.0	50.7	224	36.5
5	688	688	11.2	1700	76.0	75.7	0	0.0	473	7.7	31.6	94	23.5	15.0	128	11.2	1.4	2	1.1	48.1	224	35.8
6	1169	1169	19.6	1700	76.2	75.3	0	0.0	961	16.1	63.3	91	45.6	15.0	128	10.8	1.4	2	1.0	79.8	221	57.4
7	1101	1101	17.9	1700	76.2	75.4	0	0.0	886	14.4	56.9	91	42.3	15.0	128	11.1	1.4	2	1.1	73.5	221	54.5
8	751	834	13.6	1617	75.5	75.1	0	0.0	619	10.1	40.6	92	30.2	15.0	128	11.1	1.4	2	1.1	56.9	221	42.4
9	362	595	10.0	1384	74.0	73.8	0	0.0	367	6.5	26.3	88	18.9	14.6	124	10.6	1.4	2	1.0	42.4	214	30.6
10	474	605	9.8	1253	73.1	72.9	0	0.0	390	6.3	25.1	86	18.7	14.5	121	10.8	1.3	2	1.0	41.0	209	30.5
11	691	725	12.2	1219	73.0	72.5	0	0.0	517	8.7	33.5	84	24.1	14.4	119	10.4	1.3	2	1.0	49.2	206	35.5
12	640	773	12.6	1086	72.0	71.5	0	0.0	557	9.1	34.4	81	25.6	14.2	116	10.6	1.3	2	1.0	50.0	200	37.2
SUM	10996	11242					274		6433				393.7			129.4			12.3			535.4

YEAR 1949	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR EL.	MOUL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP.POWER AVE MW	PINOP.POWER PEAK MW	PINOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1071	905	14.7	1252	73.5	72.6	0	0.0	690	11.2	42.1	84	31.3	14.5	120	10.6	1.3	2	1.0	57.7	206	42.9
2	1371	1070	19.3	1554	75.5	74.3	0	0.0	875	15.8	59.4	88	39.9	14.6	125	9.8	1.4	2	0.9	75.3	215	50.6
3	1583	1437	23.4	1700	76.4	75.0	0	0.0	1222	19.9	75.1	89	55.9	14.8	127	11.0	1.4	2	1.1	91.4	218	68.0
4	2149	2110	35.5	1735	76.8	75.0	476	8.0	1430	24.0	83.9	84	60.4	14.9	127	10.7	1.4	2	1.0	100.1	213	72.1
5	918	953	15.5	1700	76.1	75.5	0	0.0	738	12.0	47.9	92	35.6	15.0	128	11.1	1.5	2	1.1	64.3	222	47.8
6	922	922	15.5	1700	76.1	75.5	0	0.0	714	12.0	48.1	92	34.6	15.0	128	10.8	1.4	2	1.0	64.5	222	46.5
7	746	746	12.1	1700	76.0	75.7	0	0.0	531	8.6	35.2	93	26.2	15.0	128	11.2	1.4	2	1.1	51.7	223	38.5
8	740	863	14.0	1631	75.6	75.2	0	0.0	647	10.5	42.5	92	31.6	15.0	128	11.2	1.4	2	1.1	58.9	222	43.8
9	813	905	15.2	1539	75.1	74.6	0	0.0	697	11.7	46.5	90	33.5	14.9	126	10.7	1.4	2	1.0	62.8	218	45.2
10	728	878	14.3	1389	74.1	73.6	0	0.0	663	10.8	42.3	87	31.5	14.7	123	10.9	1.4	2	1.0	58.4	212	43.5
11	764	864	14.5	1289	73.5	72.9	0	0.0	656	11.0	42.5	85	30.6	14.5	121	10.5	1.4	2	1.0	58.4	208	42.1
12	932	1014	16.5	1207	73.0	72.2	0	0.0	799	13.0	48.8	82	36.3	14.4	119	10.7	1.3	2	1.0	64.6	203	48.0
SUM	12791	12670					476		9660				447.5			129.2			12.2			588.9

YEAR 1970	INFLOW KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR MODL EL.	MAR MODL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PTNOP.POWER AVE MW	PTNOP.POWER PEAK MW	PTNOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1012	945	15.4	1275	73.5	72.7	0	0.0	751	11.9	40.9	84	33.4	14.3	120	10.7	1.3	2	1.0	60.5	206	45.0
2	912	777	14.0	1308	74.3	73.7	0	0.0	583	10.5	40.5	87	27.2	14.5	123	9.7	1.4	2	0.9	56.4	213	37.9
3	1125	855	13.6	1694	76.1	75.5	0	0.0	620	10.1	40.1	93	29.8	14.8	128	11.0	1.4	2	1.0	56.3	223	41.9
4	957	955	16.0	1700	76.1	75.5	0	0.0	746	12.5	50.1	92	36.1	15.0	128	10.8	1.4	2	1.0	66.6	222	47.9
5	729	729	11.9	1700	76.0	75.7	0	0.0	514	8.4	34.2	94	25.4	15.0	128	11.2	1.4	2	1.1	50.6	224	37.7
6	447	454	7.6	1693	75.9	75.7	0	0.0	245	4.1	17.3	95	12.4	15.1	128	10.8	1.4	2	1.0	33.7	225	24.3
7	457	534	8.7	1617	75.4	75.3	0	0.0	318	5.2	21.5	93	16.0	15.0	128	11.2	1.4	2	1.1	37.9	223	28.2
8	1040	971	15.8	1686	76.0	75.4	0	0.0	756	12.3	49.0	92	36.5	15.0	128	11.1	1.4	2	1.1	65.4	222	48.7
9	722	881	14.8	1527	75.0	74.6	0	0.0	672	11.3	45.0	90	32.4	14.9	126	10.7	1.4	2	1.0	61.3	218	44.2
10	264	570	9.3	1237	73.0	72.8	0	0.0	359	5.8	23.3	86	17.3	14.6	120	10.9	1.4	2	1.0	39.3	208	29.2
11	925	870	14.6	1292	73.0	72.9	0	0.0	662	11.1	42.5	85	30.6	14.4	121	10.4	1.3	2	1.0	58.3	208	42.0
12	609	800	13.0	1100	72.1	71.6	0	0.0	585	9.5	36.3	82	27.0	14.3	117	10.6	1.3	2	1.0	51.9	200	38.6
SUM	9219	9326					0		6792				324.2			129.2			12.2			465.6

YEAR 1971	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR MODL EL.	MAR MODL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PTNOP.POWER AVE MW	PTNOP.POWER PEAK MW	PTNOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1241	1027	16.7	1314	73.6	72.8	0	0.0	812	13.2	49.3	84	36.6	14.3	121	10.6	1.3	2	1.0	64.9	207	48.3
2	1753	1367	24.6	1700	76.0	74.8	0	0.0	1172	21.1	77.8	88	52.3	14.6	127	9.8	1.4	2	0.9	93.9	217	63.1
3	2074	2040	33.2	1734	76.8	75.0	340	5.5	1485	24.1	84.8	85	63.1	14.9	127	11.1	1.4	2	1.0	101.0	214	75.2
4	1181	1215	20.4	1700	76.3	75.3	0	0.0	1006	16.9	65.6	90	47.2	14.9	128	10.8	1.5	2	1.0	82.0	220	59.0
5	1400	1400	22.8	1700	76.4	75.1	0	0.0	1185	19.3	73.9	89	55.0	14.9	128	11.1	1.4	2	1.1	90.3	219	67.2
6	862	862	14.5	1700	76.1	75.6	0	0.0	654	11.0	44.1	93	31.8	15.0	128	10.8	1.4	2	1.0	60.5	223	43.6
7	783	783	12.7	1700	76.0	75.6	0	0.0	568	9.2	37.6	93	28.0	15.0	128	11.2	1.4	2	1.1	54.1	223	40.2
8	1023	1023	16.6	1700	76.1	75.5	0	0.0	808	13.1	52.4	92	39.0	15.0	128	11.2	1.4	2	1.1	68.8	222	51.2
9	708	861	14.8	1527	75.0	74.6	0	0.0	673	11.3	45.1	90	32.4	14.9	126	10.7	1.4	2	1.0	61.4	218	44.2
10	1353	1287	20.9	1593	75.7	74.6	0	0.0	1071	17.4	66.6	88	49.6	14.8	126	11.0	1.4	2	1.1	82.8	216	61.6
11	1271	1339	22.5	1526	75.4	74.1	0	0.0	1130	19.0	71.7	86	51.6	14.8	125	10.6	1.4	2	1.0	87.8	213	63.2
12	1650	1651	26.8	1525	75.6	73.7	0	0.0	1435	23.3	84.9	84	63.2	14.7	123	10.9	1.4	2	1.1	101.0	210	75.1
SUM	15299	14874					340		12000				549.7			129.8			12.4			692.0

YEAR 1972	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR MODL EL.	MAR MODL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PTNOP.POWER AVE MW	PTNOP.POWER PEAK MW	PTNOP.POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1946	1752	28.5	1720	76.8	74.8	38	0.6	1499	24.4	85.8	87	63.8	14.7	127	11.0	1.4	2	1.1	102.0	215	75.9
2	1661	1628	29.3	1733	76.8	75.0	81	1.5	1353	24.4	86.6	87	58.2	14.9	127	10.0	1.4	2	1.0	102.9	216	69.2
3	1003	1036	16.8	1700	76.1	75.5	0	0.0	821	13.3	52.8	92	39.3	14.9	128	11.1	1.5	2	1.1	69.2	222	51.5
4	1030	1030	17.3	1700	76.1	75.5	0	0.0	822	13.8	54.8	91	39.4	15.0	128	10.8	1.4	2	1.0	71.2	221	51.3
5	1316	1316	21.4	1700	76.3	75.2	0	0.0	1101	17.9	69.3	90	51.6	15.0	128	11.1	1.4	2	1.1	85.7	220	63.8
6	1294	1294	21.7	1700	76.3	75.2	0	0.0	1086	18.2	70.3	90	50.7	14.9	128	10.8	1.4	2	1.0	86.7	220	62.5
7	1009	1009	16.4	1700	76.1	75.5	0	0.0	794	12.9	51.3	92	38.2	15.0	128	11.1	1.4	2	1.1	67.7	222	50.4
8	917	945	15.4	1672	75.9	75.4	0	0.0	729	11.9	47.5	92	35.3	15.0	128	11.2	1.4	2	1.1	63.9	222	47.6
9	558	763	12.8	1466	74.6	74.3	0	0.0	554	9.3	37.4	89	26.9	14.9	125	10.7	1.4	2	1.0	53.6	216	38.6
10	560	732	11.9	1316	73.6	73.2	0	0.0	517	8.4	33.2	86	24.7	14.6	122	10.9	1.4	2	1.0	49.2	210	36.6
11	723	768	13.2	1251	73.2	72.7	0	0.0	580	9.7	37.6	85	27.1	14.5	120	10.4	1.3	2	1.0	53.4	207	38.5
12	1853	1612	26.1	1501	75.5	73.5	0	0.0	1387	22.6	81.2	84	60.4	14.5	123	10.8	1.4	2	1.0	97.1	206	72.2
SUM	13870	13894					118		11242				515.6			129.9			12.4			657.9

YEAR 1973	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR POOL EL.	MAR POOL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP. POWER AVE MW	PINOP. POWER PEAK MW	PINOP. POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1928	1919	23.1	1511	75.3	73.9	0	0.0	1204	19.6	72.6	86	54.1	14.6	124	10.9	1.4	2	1.1	88.7	212	66.0
2	2383	2170	39.1	1722	76.0	74.8	653	11.8	1323	23.8	80.9	82	54.4	14.8	127	9.9	1.3	2	0.9	97.0	211	65.2
3	2216	2202	35.8	1737	76.8	75.0	510	8.3	1477	24.0	83.6	84	62.2	14.9	127	11.1	1.4	2	1.0	99.8	213	74.3
4	2850	2843	47.8	1795	76.8	75.1	1243	20.9	1392	23.4	78.3	78	56.3	14.9	128	10.7	1.2	2	0.9	94.4	208	67.9
5	1213	1256	20.[illegible]	1700	76.2	75.3	0	0.0	1041	16.9	65.7	90	48.9	14.9	128	11.1	1.5	2	1.1	82.1	220	61.1
6	2227	2192	36.6	1735	76.8	75.0	558	9.4	1426	24.0	83.5	83	60.1	14.9	127	10.8	1.3	2	1.0	99.6	213	71.6
7	1069	1104	18.0	1700	76.2	75.0	0	0.0	869	14.5	56.9	91	42.3	14.9	128	11.1	1.5	2	1.1	73.3	221	54.5
8	914	943	15.3	1671	75.9	75.4	0	0.0	727	11.8	47.4	92	35.2	15.0	128	11.2	1.4	2	1.1	63.8	222	47.5
9	1028	1075	18.1	1624	75.7	75.0	0	0.0	867	14.6	57.2	90	41.2	14.9	127	10.8	1.4	2	1.0	73.6	219	53.0
10	554	819	13.3	1359	73.9	73.5	0	0.0	604	9.8	38.8	87	28.9	14.7	123	11.0	1.4	2	1.0	55.0	212	40.9
11	481	656	11.0	1184	72.6	72.3	0	0.0	448	7.5	29.4	84	21.1	14.4	119	10.4	1.3	2	1.0	45.1	205	32.5
12	900	923	15.0	1161	72.0	71.9	0	0.0	708	11.5	43.2	82	32.2	14.3	118	10.6	1.3	2	1.0	58.8	202	43.8
SUM	17263	17403					2944		12105				537.0			129.5			12.0			678.5

YEAR 1974	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR POOL EL.	MAR POOL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP. POWER AVE MW	PINOP. POWER PEAK MW	PINOP. POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	1860	1481	24.1	1540	75.7	73.9	0	0.0	1266	20.6	74.6	86	55.5	14.4	124	10.7	1.4	2	1.0	90.4	211	67.3
2	1902	1722	31.0	1721	76.8	74.8	181	3.3	1346	24.2	84.7	86	56.9	14.8	127	9.9	1.4	2	0.9	100.9	214	67.8
3	1013	1034	16.8	1700	76.1	75.5	0	0.0	818	13.3	52.6	92	39.1	14.9	128	11.1	1.5	2	1.1	69.0	222	51.3
4	1609	1609	27.0	1700	76.6	74.8	0	0.0	1401	23.5	88.1	87	63.4	14.9	127	10.8	1.4	2	1.0	104.5	216	75.2
5	1037	1037	16.9	1700	76.1	75.5	0	0.0	822	13.4	52.8	92	39.3	14.9	128	11.1	1.4	2	1.1	69.2	222	51.5
6	743	743	12.5	1700	76.0	75.7	0	0.0	535	9.0	36.6	93	26.4	15.0	128	10.8	1.4	2	1.0	53.1	223	38.2
7	876	876	14.2	1700	76.0	75.6	0	0.0	661	10.7	43.4	93	32.3	15.0	128	11.2	1.4	2	1.1	59.9	223	44.6
8	1000	1000	16.3	1700	76.1	75.5	0	0.0	785	12.8	51.0	92	37.9	15.0	128	11.2	1.4	2	1.1	67.4	222	50.2
9	777	927	15.6	1550	75.2	74.7	0	0.0	719	12.1	48.0	90	34.6	14.9	126	10.7	1.4	2	1.0	64.3	216	46.3
10	559	773	12.6	1336	73.7	73.4	0	0.0	557	9.1	35.9	87	26.7	14.7	122	10.9	1.4	2	1.0	51.9	211	38.6
11	561	707	11.9	1210	72.9	72.5	0	0.0	499	8.4	32.6	84	23.5	14.4	119	10.4	1.3	2	1.0	48.4	206	34.6
12	999	1006	16.4	1203	73.0	72.2	0	0.0	791	12.9	48.2	82	35.8	14.3	118	10.7	1.3	2	1.0	63.8	203	47.5
SUM	12956	12914					181		10199				471.4			129.5			12.3			613.3

YEAR 1975	INFLO KAF	TOTAL RELEASE KAF	TOTAL RELEASE KCFS	STOR. KAF	MAR POOL EL.	MAR POOL FL.	SPILL KAF	SPILL KCFS	STEPH KAF	REL. KCFS	STEPH. POWER AVE MW	STEPH. POWER PEAK MW	STEPH. POWER EN. MKWH	PINOP. POWER AVE MW	PINOP. POWER PEAK MW	PINOP. POWER EN. MKWH	SPWY. POWER AVE MW	SPWY. POWER PEAK MW	SPWY. POWER EN. MKWH	TOTAL POWER AVE MW	TOTAL POWER PEAK MW	TOTAL POWER EN. MKWH
1	2068	1647	26.8	1624	76.3	74.1	0	0.0	1432	23.3	83.6	86	62.2	14.5	125	10.8	1.4	2	1.0	99.5	212	74.0
2	1780	1677	30.2	1726	76.8	74.9	134	2.4	1349	24.3	85.5	86	57.4	14.8	127	9.9	1.4	2	1.0	101.7	215	68.3
3	3228	3210	52.2	1745	76.8	75.1	1570	25.5	1424	23.2	76.1	76	56.6	14.9	128	11.1	1.1	2	0.8	92.2	206	68.6
4	1662	1673	28.1	1734	76.8	75.0	11	0.2	1453	24.4	87.5	87	63.0	14.9	127	10.7	1.5	2	1.1	103.9	217	74.8
5	1862	1862	30.3	1735	76.8	75.0	152	2.5	1494	24.3	86.4	86	64.3	14.9	127	11.1	1.4	2	1.1	102.7	216	76.4
6	1549	1624	27.3	1700	76.6	74.8	0	0.0	1415	23.8	87.6	87	65.1	14.9	127	10.7	1.5	2	1.1	103.9	216	74.6
7	1409	1409	22.9	1700	76.4	75.1	0	0.0	1194	19.4	70.0	89	55.1	14.9	128	11.1	1.5	2	1.1	90.4	219	67.2
8	937	958	15.6	1679	76.0	75.4	0	0.0	743	12.1	48.2	92	35.8	15.0	128	11.1	1.4	2	1.1	64.6	222	48.0
9	1143	1157	19.4	1665	76.0	75.1	0	0.0	949	15.9	62.3	90	44.9	15.0	128	10.8	1.4	2	1.0	78.7	220	56.7
10	1225	1293	21.0	1597	75.7	74.7	0	0.0	1078	17.5	67.0	88	50.2	14.9	126	11.1	1.4	2	1.1	83.7	217	62.3
11	1263	1349	22.7	1531	75.3	74.1	0	0.0	1141	19.2	72.3	86	52.1	14.8	125	10.6	1.4	2	1.0	88.5	213	63.7
12	970	1203	19.6	1302	73.7	72.8	0	0.0	988	16.1	60.5	83	45.0	14.6	120	10.8	1.4	2	1.0	76.5	206	56.9
SUM	19160	19001					1667		14660				649.6			120.9			12.3			791.8

Exhibit B-2

Standard Project Flood Determination

Prepared by:
The Savannah District, Corps of Engineers

COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

STANDARD PROJECT FLOOD DETERMINATION

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TEXT

TABLES

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COOPER RIVER REDIVERSION PROJECT

LAKE MOULTRIE AND SANTEE RIVER, SOUTH CAROLINA

STANDARD PROJECT FLOOD DETERMINATION

1. Purpose of Study - The purpose of this study is to develop a standard project flood inflow hydrograph for Lake Marion on the Santee River near Pineville, South Carolina and to present a description and analysis of the major storms and floods which have occurred in the Santee River Basin. The standard project flood and related data are pertinent to the design of the proposed powerhouse and intake and exit canals to be built as part of the Cooper River Rediversion Project.

2. Scope of Study - This study includes the following information:

a. A description of the Santee River Basin.

b. A description of the storms and floods of record in the Santee River Basin.

c. The rationale for selection of the standard project storm.

d. The methods of analysis used to develop the standard project flood.

e. The results of the analysis.

3. Description of the Drainage Basin - The Santee River Basin is located in North Carolina and South Carolina. The basin extends diagonally in a northwest direction from the coast of South Carolina between Georgetown and Charleston to the North Carolina state line and into the western part of North Carolina. The major portion of the basin lies in the central part of South Carolina. The maximum length of the basin is about 275 miles and the maximum width about 115 miles. The total drainage

area of the basin is about 15,700 square miles, of which about 10,400 are in South Carolina and 5,300 are in North Carolina. The basin lies in three well defined physiographic regions, commonly known as the Mountain Region, Piedmont Plateau, and the Coastal Plain. The northwesterly boundary is, in general, the crest of the Blue Ridge Mountain Range, with usual elevations of 4,000 to 5,000 feet msl, and an offshoot of the Black Mountain Range culminating in Mount Mitchell, with an elevation of 6,711 feet msl. The basin is shown on exhibit 1.

4. Stream Characteristics - In the Mountain Region and the Piedmont Plateau the streams have steep slopes and channel capacities that are relatively greater than those in the Coastal Plain region. The stream slopes flatten in the Coastal Plain . The stream valleys are wide and channel capacities are small. Streams in the lower reaches of the Coastal Plain tend to have sluggish flow, and swamps and marshes are predominant. Descriptions of the major streams in the Santee River Basin are given in the following paragraphs:

a. Santee River - The Santee River is formed in the central part of South Carolina by the junction of Congaree and Wateree Rivers, flows southeast, and enters the Atlantic Ocean about 10 miles north of Cape Romain. It has a total length of about 180 miles. The river lies entirely within the Coastal Plain region and exhibits characteristics common to most streams in this region; sluggish flow, wide flat flood plains and swamps and marshes.

b. Wateree - Catawba River - Wateree - Catawba River, the most northerly of the two parent streams of the Santee River, rises on the eastern slope of the Blue Ridge, in McDowell County, North Carolina, and flows first northeast and then east, then bends abruptly southeast and flows in this general direction across the south-central portion of North Carolina and across the north-central part of South Carolina to its junction with the Congaree River. This stream, throughout its course in North Carolina and also through that part of its course in South Carolina above the mouth of Wateree Creek, is known as Catawba River. The total length of the stream is about 450 miles. The greater part of the drainage basin is hilly, and the upper portions are mountainous. Many of the tributary streams rise and flow for almost their entire length in high mountains. Wateree River crosses the fall line about 5 miles above Camden, South Carolina, in rapids about 5 miles in length with a total fall of about 52 feet. The hydroelectric power capability of the river is, for the most part, fully developed by ten major reservoirs which extend along 216 miles. The rated output of the system totals 805 MW with one reservoir, Cowans Ford, contributing 350 MW.

c. Congaree River - The Congaree, the second and most southerly of the two streams, which by their union form the Santee, is formed by the junction of Broad and Saluda Rivers between Lexington and Richland counties, South Carolina. The river flows in a general southeasterly direction for about 60 miles to its junction with the Wateree.

d. Broad River -The Broad River rises on the eastern slope of the Blue Ridge near Hickory Nut Gap, in the southwestern part of McDowell County and the northeastern part of Henderson County, North Carolina, and flows in a general southeasterly direction across a portion of south-central North Carolina and north-central South Carolina to its junction with the Saluda River at Columbia, South Carolina. The length of the river is about 240 miles. In general character the basin closely resembles the Catawba. It lies entirely above the fall line, but has not been developed to any great extent for hydroelectric power purposes.

e. Saluda River - The Saluda River is formed in western South Carolina by the junction of the north, south and middle forks, and flows southeast to its junction with Broad River, the length of the stream being about 110 miles. The three forks are mountain streams, and the character of the drainage basin is similar to that of the Broad River. Two large hydropower projects are located on the Saluda River. Lake Greenwood is located near Chappells, South Carolina and controls a drainage area of 1,150 square miles. Lake Murray located near Lexington, South Carolina recaptures flows released from Lake Greenwood and also receives runoff from 1270 square miles of intermediate drainage area.

5. Description of Reservoirs - A brief description of each of the major reservoirs in the Santee River Basin is given in table 1. With the exception of the three small reservoirs in the Broad River Basin all of the projects have hydroelectric power plants. The Duke Power Company owns and operates the projects in the Catawba-Wateree River Basin. Lakes Murray and Marion are owned and operated by South Carolina Electric and Gas Company and the South Carolina Public Service Authority, respectively. A profile of the Catawba-Wateree River showing the Duke Power Company reservoir system is shown on exhibit 2.

6. Major Storms of Record on the Santee River Basin

(a) Storm of August 24-26, 1908. Widespread rains occurred over the South Atlantic States on August 24 to 26, 1908. These were accompanied by heavy downpours over portions of Georgia, and North and South Carolina. At Monroe, North Carolina, 15.58 inches fell in three days. The rain was quite evenly distributed

TABLE I

CATAWBA RIVER BASIN RESERVOIRS

RESERVOIR	FIRST OPERATED	CAPACITY MIL. CU. FT. NORMAL POOL	SPILLWAY TYPE	DRAINAGE AREA SQ. MI.
Bridgewater	May 1919	10,506	Overflow	380
Rhodhiss	Feb. 1925	1,717	Overflow	1,088
Oxford	Apr. 1928	2,278	Gated	1,310
Lookout Shoals	Dec. 1915	474	Overflow	1,449
Cowans Ford	Sep. 1963	18,100	Gated	1,770
Mountain Island	Dec. 1923	1,132	Overflow	1,860
Catawba	Aug. 1925	6,542	Gated	3,020
Fishing Creek	Nov. 1916	1,630	Gated	3,810
Rocky Creek	Apr. 1908	163	Gated	4,360
Wateree	Oct. 1919	7,626	Overflow	4,750
BROAD RIVER BASIN				
Lure		1,533	Overflow	98
Adger		522	Overflow	42
Summit		577	Overflow	42
SALUDA RIVER BASIN				
Greenwood	May 1940	11,751	Gated	1,150
Murray	Aug. 1929	60,113	Gated	2,440
SANTEE BASIN				
Marion	Nov. 1941	63,180	Gated	14,700

and had been preceded by heavy rains on the 19th and 21st of August, which had partially saturated the soil in many places. The Santee and Savannah Rivers experienced the most destructive floods in their histories. The Wateree River at Camden, South Carolina reached a peak discharge of 366,000 cfs on August 26, 1908.

(b) Storm of July 14-16, 1916. This storm was produced by a tropical hurricane which entered Charleston, South Carolina on the morning of July 14, 1916. The maximum rainfall was recorded in the mountains of North Carolina. Altapass, North Carolina recorded 23.77 inches in three days, with 23.22 inches of rain falling between 2 P.M. of July 15 and 2 P.M. of July 16. The average rainfall over the entire Santee Basin was approximately 8.4 inches. The floods produced by this storm caused the lower reaches of many streams to flood before their headwaters due to the westerly direction of the storm. This diminished flood peaks to some extent. The rains of this storm, however, fell on soil that had been saturated by another tropical storm which passed inland from the Gulf of Mexico on July 9 to 13th. Flood stages attained during this storm exceeded those of August, 1908, on the Catawba River at Mt. Holly, North Carolina, and at Catawba, South Carolina. On July 18, 1916, a discharge of 382,000 cfs was recorded at Rocky Creek Dam. A discharge of 180,000 cfs was recorded at Lookout Shoals Reservoir on July 16, 1916, prior to the failure of an earth dike. The highest stage known (44.1 ft.) at Catawba, North Carolina, occurred partially because of a dike failure at Lookout Shoals Dam. The Wateree River at Camden, South Carolina gage recorded a peak discharge of 400,000 cfs during this flood.

(c) Storm of August 10-17, 1940. This hurricane storm moved inland in the vicinity of Beaufort, South Carolina and Savannah, Georgia on August 11, 1940. As the hurricane proceeded overland with decreasing intensity, it curved northward along the Appalachian Mountains, thence eastward, passing out into the Atlantic Ocean below Norfork, Virginia on August 16. Precipitation greater than 15 inches for the entire storm and 8 inches during a single day was recorded at many points. On drainage areas less than 5,000 square miles, the average rainfall was less than that recorded in July of 1916, but it exceeded that recorded in the earlier storm over areas greater than 5,000 square miles. The intensity of hourly precipitation recorded was generally not too high during the August, 1940 storm, but the excess runoff from many mountainous streams, especially in the Catawba River Basin, indicates intensities greater than those recorded.

The floods were severe on the headwaters of the Catawba River where peak discharges of 1,400 cfs per square mile from drainage areas of more than 50 square miles were recorded in the vicinity of Grandfather's Mountain and Blowing Rock, North Carolina. The Duke Power Company's nine reservoirs in operation in 1940 materially reduced destructive flooding, especially Bridgewater and Rhodhiss Reservoirs located in the mountainous drainage area. The precipitation on the Catawba Basin was greatest along the Appalachian Mountains with a maximum of 15 inches recorded. The average precipitation over the 4,750 square mile drainage basin above Wateree Reservoir, however, was only 7.1 inches. This was due to the gradual tapering off of precipitation on the lower portion of the basin with some areas receiving only 3 inches.

7. Selection of Model Storm for Standard Project Storm Estimate - Studies made by the Hydrometeorological Section of the U. S. Weather Bureau and the Duke Power Company were used to compile a list of the greatest storms which have occurred in the southeastern part of the country. These storms (shown with related data in table II), the draft report, "All-Season Probable Maximum Precipitation, United States East of the 105th Meridian for Areas from 1,000 to 20,000 Square Miles and Durations from 6 to 72 Hours", prepared by the National Weather Service, the maximum possible precipitation estimate for the Savannah River Basin above Hartwell Dam and EM 1110-2-1411 "Standard Project Flood Determinations" were used to determine the most severe depth-duration-area relationship and isohyetal pattern of any storm that is considered reasonably characteristic of the Santee River Basin. A comparative summary of depth-duration-area data for the storms analyzed is shown in table II. The depth-area relationship for the generalized storm data and the 1916 storm for a 72-hour duration is shown on exhibit 3. As shown in these data revised probable maximum precipitation estimates from the report "All-Season Probable Maximum Precipitation, United States East of the 105th Meridian for Areas from 1,000 to 20,000 Square Miles and Durations from 6 to 72 Hours" are considerably greater than previous probable maximum precipitation estimates. This is due to the Yankeetown, Florida storm of 3-7 September 1950. The outstanding rains of this storm are not included in earlier estimates of probable maximum and standard project storm rainfall.

The Santee River Basin is within an area where strong orographic controls exist, and the pattern selected for the standard project storm should reflect this orography. The

TABLE II

RAINFALL DEPTH - DRAINAGE AREA - DURATION RELATIONSHIPS
FOR HISTORICAL AND DESIGN STORMS

STORM OF 23-28 SEPTEMBER 1929

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	60	72	96	120
10	8.4	12.9	15.3	16.0	16.0	16.4	19.0	19.6	19.6	20.0	20.0
100	8.1	12.4	14.1	15.1	15.6	16.3	18.4	19.2	19.3	19.7	19.7
200	7.9	12.1	13.7	14.6	15.3	16.1	18.1	18.9	19.0	19.6	19.6
500	7.5	11.6	12.9	13.9	14.7	15.9	17.3	18.5	18.7	19.1	19.1
1,000	7.1	10.9	12.2	13.1	14.0	15.3	16.5	17.8	18.0	18.2	18.2
2,000	6.5	10.0	11.3	12.1	12.9	14.1	15.3	16.3	16.5	16.8	16.8
5,000	5.2	7.8	9.1	9.8	10.6	11.8	13.4	14.2	14.3	14.6	14.7
10,000	3.7	5.6	6.9	7.6	8.5	9.9	11.8	12.5	12.5	12.6	12.7
20,000	2.1	3.6	4.9	5.8	6.7	7.9	9.8	10.5	10.5	10.6	10.7
50,000	1.0	1.9	2.8	3.7	4.5	5.1	6.4	7.0	7.4	7.5	7.7
70,000	0.7	1.5	2.2	3.1	3.8	4.2	5.2	5.7	6.1	6.6	6.7

STORM OF 13-17 JULY 1916

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	60	72	96	108
10	8.0	12.6	17.0	22.2	22.9	23.0	23.2	23.7	23.7	23.8	23.8
100	7.2	12.0	15.6	19.3	20.8	21.1	21.7	22.1	22.1	22.2	23.2
200	6.9	11.7	15.0	18.3	19.9	20.3	20.9	21.3	21.4	21.4	21.4
500	6.4	11.1	13.9	16.6	18.3	18.8	19.5	19.8	20.1	20.1	20.1
1,000	5.9	10.4	12.9	15.0	16.7	17.3	18.1	18.4	18.6	18.7	18.7
2,000	5.1	9.3	11.6	13.3	14.9	15.5	16.3	16.6	16.8	16.9	16.9
5,000	3.9	7.4	9.3	10.9	12.0	12.6	13.4	13.6	13.8	14.0	14.0
10,000	3.0	5.5	7.2	8.6	9.4	9.9	10.6	10.8	11.0	11.2	11.2
20,000	2.1	3.8	5.0	5.9	6.6	7.3	8.0	8.2	8.4	8.6	8.6
37,000	1.3	2.2	3.0	3.8	4.7	5.6	7.0	7.5	7.8	8.1	9.1

STORM OF 23-28 AUGUST 1908

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	60	72	96	120
10	8.0	10.2	10.8	11.7	13.0	13.6	14.2	14.9	16.2	18.0	18.0
100	6.4	8.3	9.2	10.8	11.9	12.2	13.8	14.5	15.1	16.6	16.6
200	5.9	7.8	8.7	10.4	11.5	11.8	13.6	14.3	14.8	15.9	15.9
500	5.3	7.0	8.0	9.9	10.9	11.1	13.2	13.9	14.2	15.0	15.0
1,000	4.7	6.3	7.4	9.5	10.4	10.5	12.9	13.5	13.8	14.2	14.2
2,000	4.2	5.6	6.8	8.8	9.7	9.9	12.4	12.9	13.1	13.4	13.4
5,000	3.6	4.7	5.8	7.7	8.5	8.8	11.1	11.8	12.1	12.2	12.2
10,000	3.0	4.0	5.0	6.6	7.4	7.7	9.7	10.6	11.0	11.1	11.2
20,000	2.3	3.2	4.1	5.4	6.1	6.4	8.0	9.0	9.7	9.8	9.9
50,000	1.4	2.1	2.8	3.7	4.3	4.6	5.6	6.6	7.5	7.7	7.8
69,600	1.1	1.7	2.3	3.1	3.5	4.0	4.8	[illegible]	[illegible]	6.8	6.9

STANDARD PROJECT PRECIPITATION IN INCHES

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in sq. mi.	Duration of rainfall in hours				
	6	12	24	48	72
30	9.8	12.5	15.3	17.3	18.2
100	9.5	11.9	14.4	16.4	17.2
200	9.2	11.3	13.5	15.6	16.7
300	9.0	10.9	13.0	15.1	16.0
400	8.9	10.7	12.6	14.7	15.8
500	8.8	10.5	12.3	14.3	15.5
600	8.7	10.4	12.0	14.1	15.2
700	8.6	10.2	11.7	13.9	14.9
800	8.5	10.1	11.6	13.7	14.7
1,000	8.4	9.8	11.2	13.3	14.3
2,000	7.9	9.0	10.0	12.1	13.1
5,000	7.1	7.8	8.2	10.3	11.1
10,000	6.3	6.7	6.9	8.8	9.6
20,000	5.3	5.4	5.5	7.3	8.0

60% PROBABLE MAXIMUM PRECIPITATION IN INCHES

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in sq. mi.	Duration of rainfall in hours				
	6	12	24	48	72
30	14.4	17.3	22.1	24.7	30.8
100	13.9	16.8	21.5	24.1	28.8
200	13.2	16.3	20.8	23.6	27.3
300	13.0	15.8	20.3	23.0	26.3
400	12.6	15.5	19.7	22.7	25.5
500	12.3	15.1	19.2	22.2	24.9
600	12.0	14.9	18.7	21.8	24.4
700	11.8	14.5	18.3	21.5	23.9
800	11.5	14.2	17.5	21.1	23.5
1,000	11.0	13.7	17.1	20.4	22.8
2,000	9.0	11.4	14.0	17.9	20.5
5,000	6.4	8.4	11.3	14.4	17.1
10,000	4.8	6.7	9.2	12.1	14.3
20,000	3.5	5.1	7.4	10.1	11.5

PROBABLE MAXIMUM PRECIPITATION IN INCHES

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in sq. mi.	Duration of rainfall in hours				
	6	12	24	48	72
30	24.0	28.8	36.8	41.1	51.3
100	23.2	28.0	35.8	40.2	48.0
200	22.2	27.1	34.7	39.3	45.5
300	21.7	26.4	33.8	38.4	43.9
400	21.0	25.9	32.8	37.8	42.5
500	20.5	25.1	32.0	37.0	41.5
600	20.0	24.8	31.1	36.3	40.6
700	19.6	24.1	30.5	35.8	39.9
800	19.1	23.6	29.8	35.1	39.2
1,000	18.4	22.8	[illegible]	34.0	38.0
2,000	15.0	19.0	[illegible]	29.9	34.1
5,000	10.7	14.0	19.8	24.0	28.5
10,000	8.0	11.0	15.3	20.2	23.9
20,000	5.9	8.5	[illegible]	16.5	19.2

following statement concerning orography is taken from the U. S. Weather Bureau maximum possible precipitation study for the area above Hartwell Dam. "The Altapass and Big Meadows storms have large-scale orographic features in common with the basin, and if the centers of these storms are placed on or near the ridge line, their use for patterns is allowable. The major axis of these patterns must be placed parallel to the ridge line of the basin, and may be moved up or down 500 feet from the elevation at which they occurred".

Since the Savannah River Basin is near the Santee River Basin, the above restraints concerning transposition and placement were used as guides in determining a standard project storm pattern. In the above quote the Altapass and Big Meadows storms are the storms of 13-17 July 1916 (SA2-9) and 11-17 October 1942 (SA 1-28A), respectively.

Based upon the meteorological and orographic considerations discussed above, the 13-17 July 1916 storm was selected as the model to be used in determining the time and areal rainfall distribution for the standard project storm for the Santee River Basin. The rainfall depth-duration relationship for this storm corresponds very closely with the depth-duration relationship determined by taking 50 percent of the probable maximum precipitation from the report "All-Season Probable Maximum Precipitation, United States East of the 105th Meridan for Areas From 1,000 to 20,000 Square Miles and Durations from 6 to 72 Hours". In addition, the isohyetal pattern of the storm reflects the marked orographic influence present in the basin. The 1916 storm is shown on exhibit 4.

8. <u>Other Storms Considered</u> - In addition to the storm of 14-16 July 1916 flood hydrographs were determined for the following hypothetic storms:

a. Storm of 14-16 July 1916 with a 20 percent increase in rainfall.

b. Storm resulting from the precipitation depth-duration-drainage area relationship from the report "All-Season Probable Maximum Precipitation, United States East of the 105th Meridian for Areas from 1,000 to 20,000 Square Miles and Durations from 6 to 72 Hours", reduced by 40 percent.

c. Storm resulting from the depth-area-duration relationship shown on plate 8 of EM 1110-2-1411, "Standard Project Flood Determination".

9. Initial Loss - The initial loss preceding large storms in the southeast vary from a minimum of about .05 inch to a maximum of 0.7 inch and is relatively small in comparison with the flood runoff volume. A value of 0.05 inch was used for initial loss in this study.

10. Infiltration Rate - Studies made by the Corps of Engineers in the Saluda and Savannah River Basins indicate the infiltration rate ranges from 0.05 to 0.15 inch per hour depending upon antecedent moisture conditions, slope, and soil type. The Savannah, Saluda, Broad, and Catawba-Wateree Basin ajoin and their topography, soil group, and climate are similar. Based upon this and the assumption that a standard project storm would be preceded by rainfall which would saturate the soil, an infiltration rate of 0.05 inch per hour was adopted for this study.

11. Unit Hydrographs

a. Catawba - Wateree Rivers - The Duke Power Company has made several flood studies on the Catawba-Wateree Rivers in connection with reports required by the Federal Power Commission and the Atomic Energy Commission. In their reports regionalized unit hydrograph data have been developed by the method presented in "Flood Studies", D. L. Miller and R. A. Clark, Design of Small Dams, U. S. Department of the Interior, Bureau of Reclamation, First Edition, 1960. Synthetic unit hydrograph coefficients were derived from the storm of 29, 30 September 1958 (Hurricane Gracie) for nine tributary streams in the Catawba River Basin where stream gaging records were available. The relationship between subarea hydrologic characteristics and lag time is shown on exhibit 5. Because flood peaks increase and lag time shorten with larger flood, the lag time was reduced to more closely represent conditions during a flood of standard project magnitude. This adjusted curve, which was used to develop unit hydrographs for 13 subbasins in the Wateree - Catawba Basin, is also shown on exhibit 5. The unit hydrographs are included in the appendix A.(1)

b. Broad River - Unit hydrograph data for the Broad River Basin were developed from rainfall - runoff records at 12 U.S.G.S. stream gaging stations using the optimization routine of the HEC-1 computer program. The Duke Power Company provided Clark and Snyder coefficients and other unit hydrograph data for subareas in the upper Broad River. These data were developed in connection with the Company's proposed Cherokee Nuclear Station near Gaffney, South Carolina. To supplement these data and broaden the coverage to include the lower Broad River Basin, unit hydrograph studies were made for an additional 4 sites. Unit hydrographs for 16 Broad River Basin subareas were developed and are shown with other pertinent unit hydrograph data in appendix A.(1)

(1) Appendix A not included - on file in Charleston and Savannah Districts.

c. Saluda River - A detailed analysis of the September - October 1929 floods in Saluda River Basin was made by the Corps of Engineers in 1937. This analysis included unit hydrograph derivations and the development of Synder coefficients for the basin. The results of this study, published in EM 1110-2-1405, "Flood Hydrograph Analyses and Computations" 31 August 1959, were used to compute synthetic unit hydrographs for 3 subareas in the Saluda River Basin. Unit hydrograph data for each subarea are shown in appendix A.[1]

12. Method of Analysis

a. 14-16 July 1916 Storm - The unit hydrograph procedure was used to develop a standard project flood hydrograph for the 14,700 square mile area above Lake Marion. The Santee Basin was divided into three main river basins; the Saluda, Broad, and Wateree - Catawba. These basins were then divided into 37 subbasins and unit hydrographs were computed for each subbasin. The subbasins are shown on exhibit 1.

The isohyetal pattern of the July 1916 storm was superimposed on the Santee River Basin and the average rainfall for each subarea was computed. The average hourly precipitation distribution over each subarea was determined from hourly precipitation stations by the Thiessen polygon method. Precipitation stations were plotted on the map of the Santee River Basin. A Thiessen network was then constructed around each station by drawing perpendicular bisectors to the lines connecting stations. The polygons formed are assumed to be boundries of the effective area controlled by the precipitation station. The area of each polygon falling within a subarea was measured and expressed as a percentage of the total subarea. These percentages are used as station weights for distribution of the basin average rainfall computed from isohyetals for each subarea.

Using HEC-1 a flood hydrograph for each subbasin was computed by applying the subbasin rainfall excess to the applicable unit hydrograph. Starting in the uppermost part of the Saluda River Basin subbasin, flood hydrographs were computed, routed downstream, and combined with runoff from other subbasins until a flood hydrograph was developed at the mouth of the river.

This hydrograph was then stored for future use. The process of combining and routing flood hydrographs was then repeated starting in the upper reaches of the Broad River Basin. When a flood hydrograph for the Broad River at its junction with the Saluda River had been developed, the Saluda River hydrograph was recalled from storage and combined with the Broad River hydrograph. This hydrograph was then routed to the mouth of the Congaree River, combined with the runoff from the local area, and stored for future use.

The routing-combining procedure was then repeated for the Wateree - Catawba River until a flood hydrograph at the mouth had been generated. This hydrograph was then combined with the hydrograph from the Congaree, routed to Wilson Dam and combined with the runoff hydrograph for the local area between the confluence of the Congaree and Wateree and Wilson Dam. The HEC-1 input data sequence and operation order for the computations are shown on exhibits 6 through 10. Exhibit 11 is a schematic diagram of the operations.

b. <u>14-16 July 1916 Storm with Rainfall Increased 20 Percent</u> - The same operational procedure explained in paragraph (a) was used for this storm. All rainfall values were increased 20 percent.

c. <u>Storms (b) and (c) from Paragraph 8</u> - Flood hydrographs resulting from storms (b) and (c) were developed using the stream system computational procedure of HEC-1. This routine computes a hydrograph consistent with the given precipitation depth-drainage area relationship by using a series of index hydrographs. The consistent hydrograph for a subbasin is determined by interpolating between the two index hydrographs that encompasses the subbasins drainage area. The procedure of generating index hydrographs, interpolating, routing and combining is continued until a consistent flood hydrograph is developed at the desired site. The hydrograph resulting from storms (b) and (c) are shown on exhibit 12.

13. Results of Study - Flood hydrograph results for the four storms studied are summarized in table III. For comparative purposes, the flood resulting from the revised probable maximum rainfall was computed. Data from this flood and its relationship to the other floods computed are also shown in table III.

Table III
Flood Hydrograph Results

Storm	Peak Flow (cfs)	Total Volume (ac. ft.)	Total Volume (inches)	Time to Peak (days)	Ratio of Peak to PMF Peak
July 1916	508704	4065957	5.15	7.125	30.4
July 1916 20% increase	631259	5135391	6.51	7.25	37.8
60% PMF	976374	7854857	9.95	6.25	58.4
SPF Criteria	630341	4886547	6.19	6.25	37.7
PMF	1671178	14235903	18.04	6.25	-

Appendix A[(1)] is a copy of the computer printout of the 13-16 July 1916 storm with rainfall increased 20 percent. Input data pertinent to all operations; subarea runoff computations, hydrograph routings, and hydrograph combinings are available in the printout. Similar printouts for the other storms are available and will be furnished, if needed.

Recommended Standard Project Flood Hydrograph - The flood hydrograph resulting from the 13-16 July 1916 storm with rainfall increased 20 percent is recommended for adoption as the standard project flood for Lake Marion based on the following:

a. The rainfall depth-area-duration relationship for this storm corresponds closely with that of the revised probable maximum precipitation relationship reduced 50 percent.

b. The isohyetal pattern of the storm reflects the marked orographic influence present in the basin.

c. The flood hydrograph of this storm corresponds closely with the hydrograph resulting from generalized depth-area duration data in EM 1110-2-1411, Standard Project Flood Determinations.

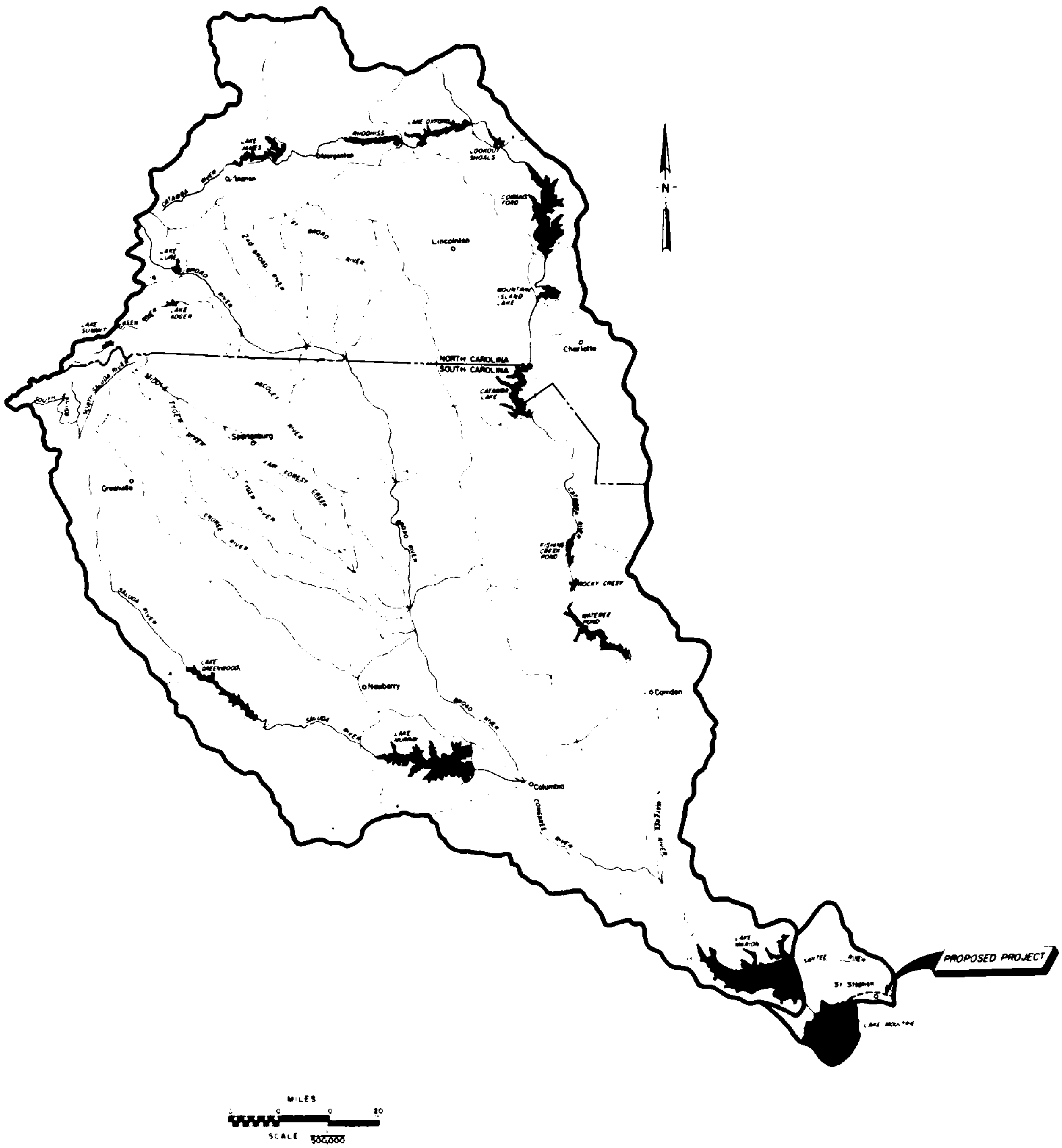
The 3 hour ordinates in cfs for the standard project flood are shown in table IV.

TABLE IV

STANDARD PROJECT FLOOD

INFLOW HYDROGRAPH TO LAKE MARION

Day	Discharge - CFS (EOP Values)							
	0300	0600	0900	1200	1500	1800	2100	2400
1	852	852	910	1265	2036	3145	4823	7559
2	11926	19054	29258	41832	55586	68556	78979	85690
3	88695	89246	88826	88350	88479	89223	89680	89098
4	87310	84677	81978	80186	79923	81688	85886	92539
5	101621	113107	126965	143155	161619	182275	205006	229654
6	256022	283367	312902	342813	373264	403873	434292	464073
7	492807	520020	545313	568184	588170	604832	617779	626631
8	631214	631259	626772	617718	604169	586481	564938	540004
9	512344	482462	451001	418566	385774	353169	321243	290417
10	261037	233359	207583	183816	162107	142450	124793	109046
11	95096	82809	72043	62650	54485	47409	41290	36006
12	31446	27512	24115	21180	18640	16436	14521	12851
13	11393	10115	8993	8004	7132	6360	5677	5069
14	4529	4049	3620	3238	2897	2592	2319	2076
15	1858	1663	1488	1332	1192	1067	955	855
16	765	685	613	549	491	440	394	352
17	315	282	253	226	203	181	162	145
18	130	117	104	94	84	75	67	60
19	54	48	43	39	35	31		

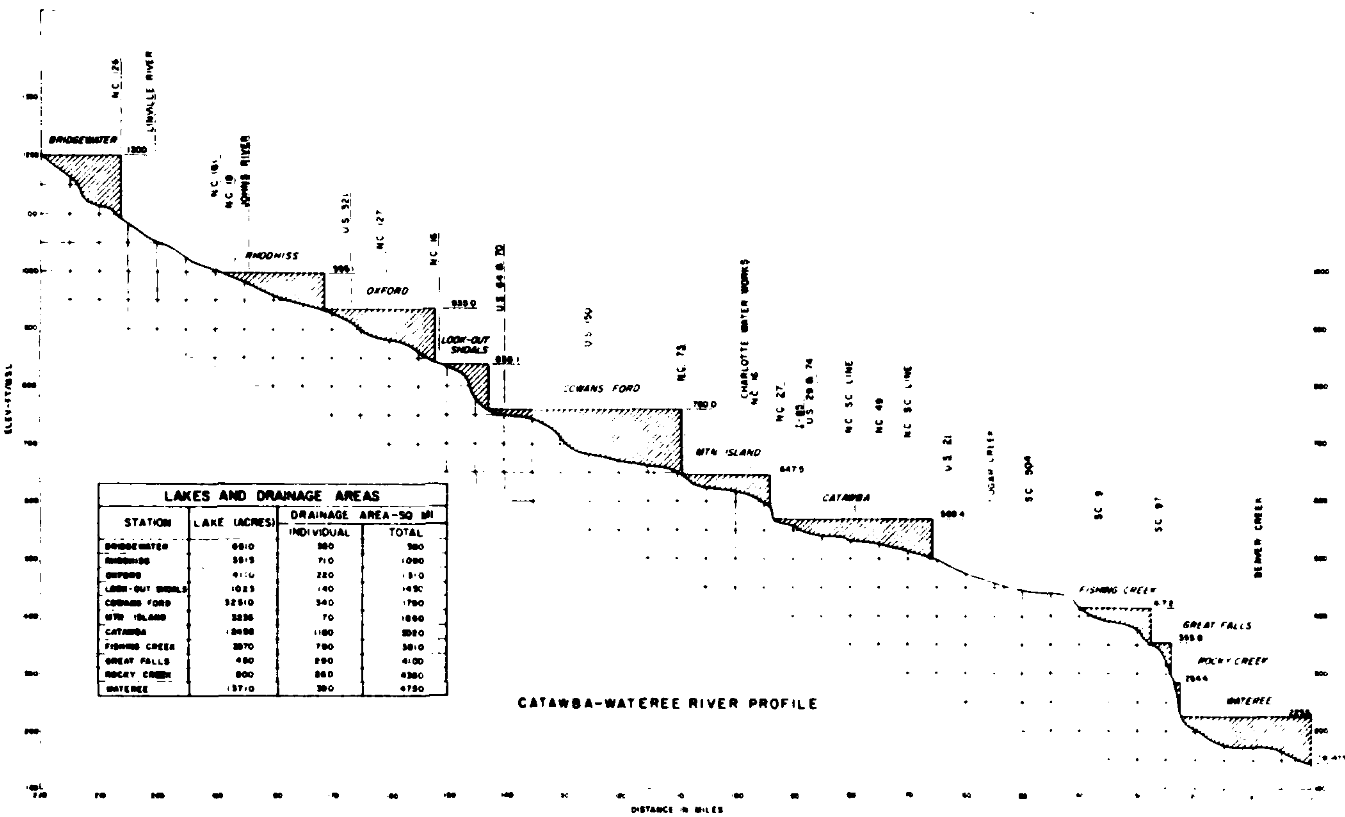


SANTEE RIVER FLOOD STUDY

BASIN
MAP

JUNE 1975

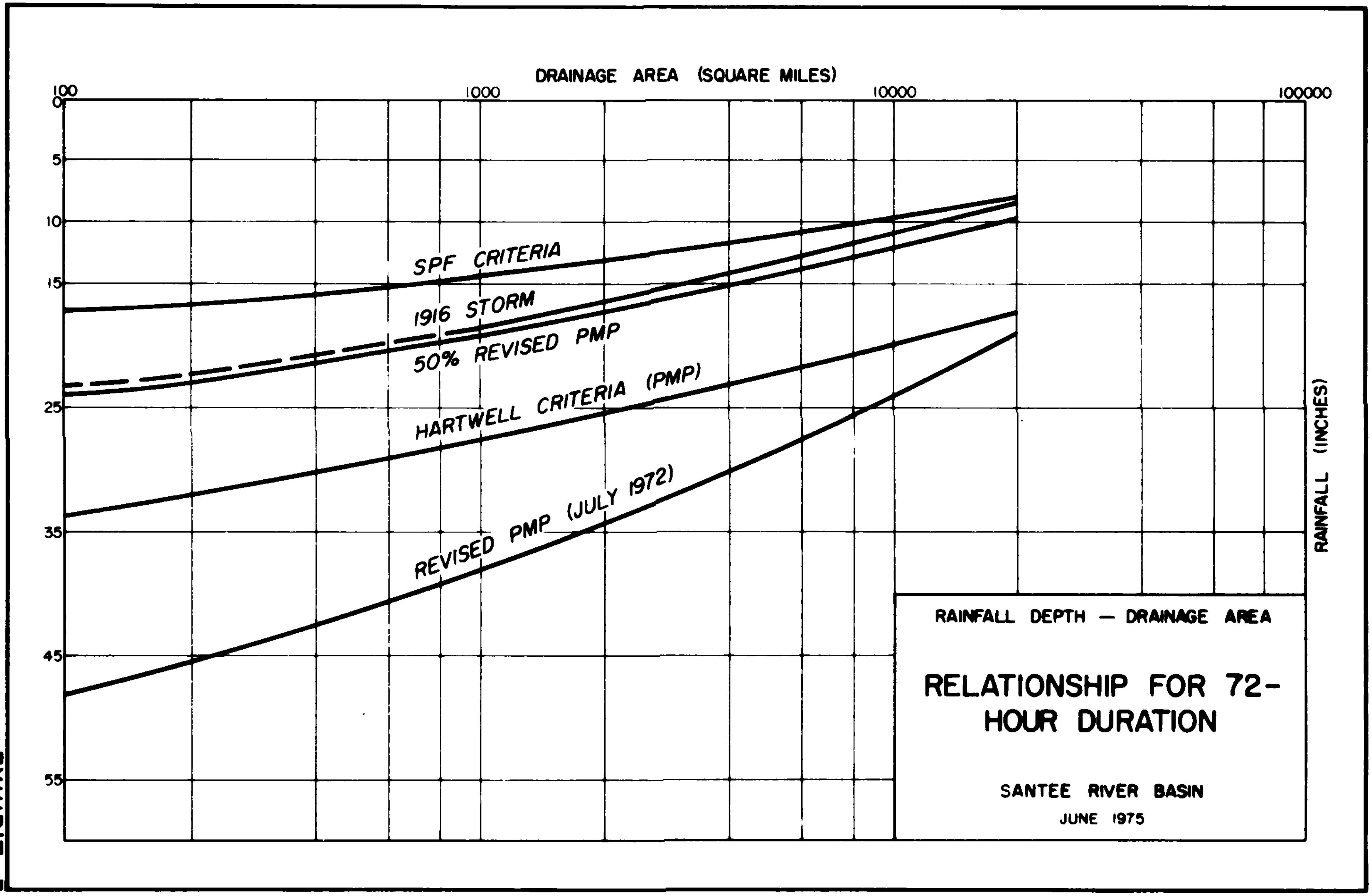
EXHIBIT I



CATAWBA-WATEREE RIVER PROFILE

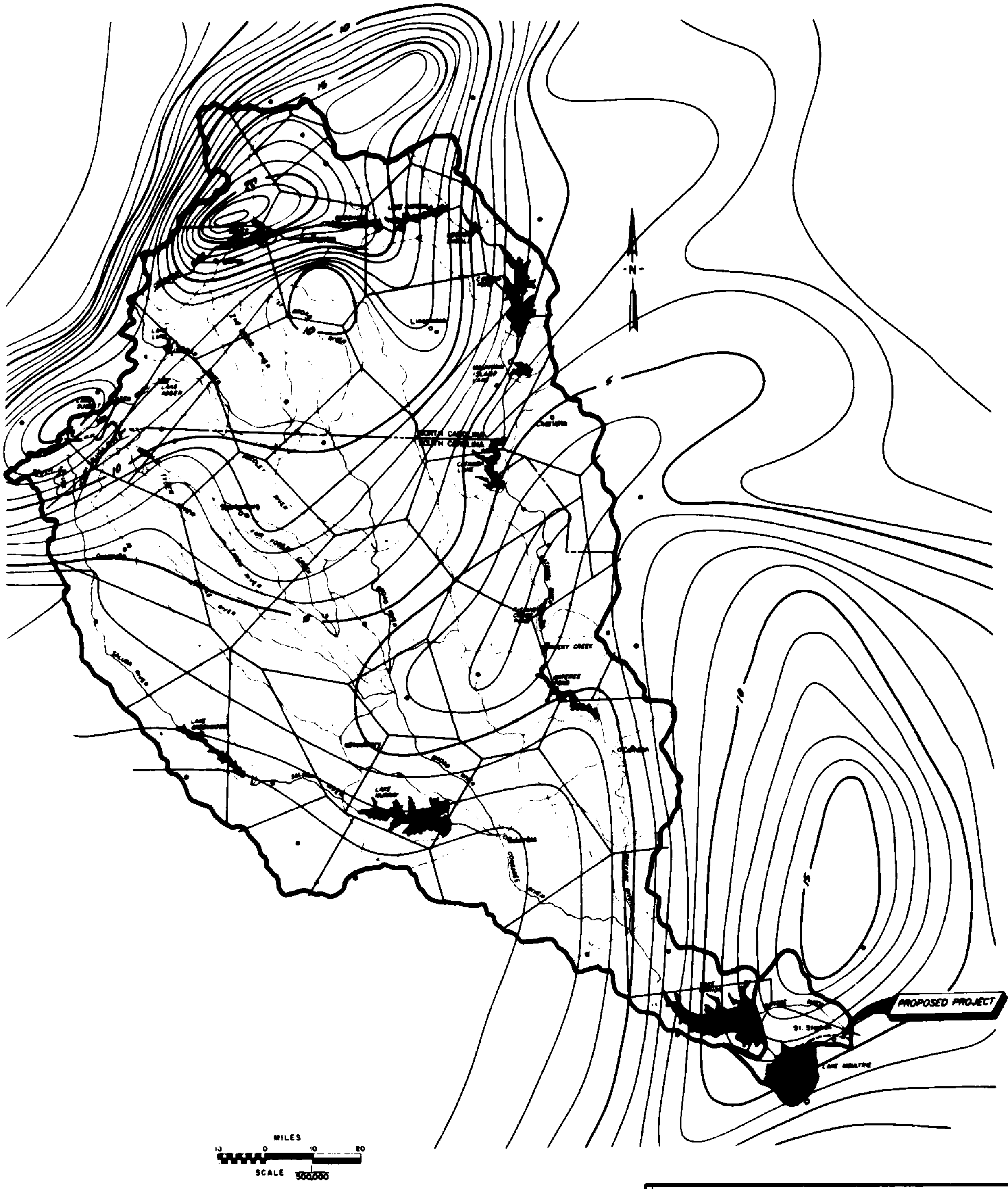
LAKES AND DRAINAGE AREAS			
STATION	LAKE (ACRES)	DRAINAGE AREA-SQ MI	
		INDIVIDUAL	TOTAL
BRIDGEWATER	6810	380	380
RHODHISS	3515	710	1090
OXFORD	4110	220	1310
LOOK-OUT SHOALS	1023	140	1450
COWANS FORD	32510	340	1790
MTN ISLAND	3235	70	1860
CATAWBA	13400	1160	3020
FISHING CREEK	3370	790	3810
GREAT FALLS	480	290	4100
ROCKY CREEK	800	280	4380
WATEREE	13710	370	4750

EXHIBIT 2



DRAINAGE AREA (SQUARE MILES)
100
1000
10000
100000
0
5
10
15
25
35
45
55
RAINFALL (INCHES)
SPF CRITERIA
1916 STORM
50% REVISED PMP
HARTWELL CRITERIA (PMP)
REVISED PMP (JULY 1972)
RAINFALL DEPTH — DRAINAGE AREA
RELATIONSHIP FOR 72-HOUR DURATION
SANTEE RIVER BASIN
JUNE 1975

EXHIBIT 3



LEGEND

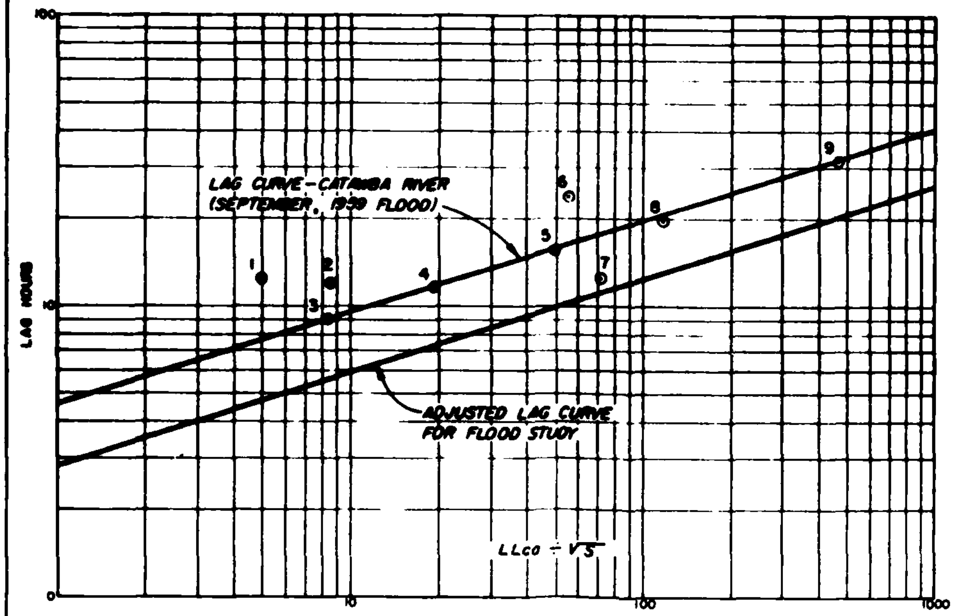
- - - - SUBAREA DRAINAGE DIVIDE
4 SUBAREA DRAINAGE NUMBER (SEE EXHIBIT II)
——— ISOHYETAL LINES FOR TOTAL PRECIPITATION IN INCHES

SANTEE RIVER FLOOD STUDY

ISOHYETAL MAP
FOR
14-16 JULY 1916 STORM

JUNE 1975

EXHIBIT 4



1. LOWER LITTLE RIVER D.A.=34.2 SQ. MI.
2. MOUNTAIN CREEK D.A.=42.4 SQ. MI.
3. LONG CREEK D.A.=31.4 SQ. MI.
4. CATAWBA RIVER (NEAR MARION, N.C.) D.A.=171 SQ. MI.
5. HENRY CREEK D.A.=80 SQ. MI.
6. INDIAN CREEK D.A.=68.4 SQ. MI.
7. LINVILLE RIVER D.A.=66 SQ. MI.
8. ROCKY CREEK D.A.=194 SQ. MI.
9. SOUTH FORK CATAWBA RIVER AT LOWELL, N.C. D.A.=625 SQ. MI.

SANTEE RIVER FLOOD STUDY

LAG CURVES

CATAWBA RIVER BASIN

EXHIBIT 5

EXHIBIT 6

HEC-1 INPUT DATA SEQUENCE

SALUDA RIVER

Operation

1. Title and job specification

2. Compute runoff from storm at Lake Greenwood - Area 14

3. Route hydrograph at 14 to 16 by lag

4. Compute subarea hydrograph, for areas 15 and 16, at 16

5. Combine routed and local flows at 16

6. Route hydrograph at 16 to 17 by lag

Operation

7. Compute subarea hydrograph at 17

8. Combine routed and local flows at 17

9. Route hydrograph at 17 to 131 (confluence of Broad River)

EXHIBIT 7

HEC-1 INPUT DATA SEQUENCE

BROAD RIVER

Operation

1. Title and job specification
2. Compute runoff from storm at Lake Summit on Green River Area 191
3. Route hydrograph through Lake Summit 191 by Modified Puls
4. Route hydrograph at 191 to 192 (Lake Adger) by lag
5. Compute subarea hydrograph at Lake Adger area 192
6. Combine routed and local flows at 192
7. Route flows at 192 through Lake Adger by Modified Puls
8. Route hydrograph at Lake Adger to 193 (confluence of Green & Broad Rivers
9. Compute local flow at 193
10. Combine routed and local flows at 193
11. Compute subarea hydrograph at 18 (Lake Lure)
12. Route hydrograph through Lake Lure
13. Route hydrograph from 18 to Broad River at 193 (confluence of Green & Broad Rivers) by lag
14. Combine hydrograph from Lake Lure 18 with Green River hydrograph at 193
15. Route hydrograph at 193 to Broad River area 20 by lag
16. Compute subarea hydrograph at 20
17. Combine routed and local flows at 20

Operation

18. Route hydrograph from 20 to 21 by lag
19. Compute subarea hydrograph at 21 (2nd Broad River)
20. Combine routed and local flows at 21
21. Route hydrograph from 21 to 22 by lag
22. Compute subarea hydrograph at 22
23. Combine routed and local flows at 22
24. Route hydrograph to 23 by lag
25. Compute subarea hydrograph at 23 (1st Broad River)
26. Combine routed and local flows at 23
27. Route hydrograph from 23 to 25 by lag
28. Compute subarea hydrograph at 24
29. Combine routed and local flows at 24
30. Route hydrograph to 25 by lag
31. Compute subarea hydrograph at 25 (Pacolet River)
32. Combine routed and local flows at 25
33. Route hydrograph to 26 by Muskingum method
34. Compute subarea hydrograph at 26
35. Combine routed and local flows at 26
36. Route hydrograph to 27 by lag
37. Compute subarea hydrograph at 29 (Middle Tyger River)

Operation

38. Route hydrograph to 27 by lag
39. Compute subarea hydrograph at 28 (Fairforest Creek)
40. Route hydrograph to 27 by lag
41. Compute subarea hydrograph 27 (Tyger River)
42. Combine routed hydrographs 26, 28 & 29 with local flows at 27
43. Route hydrograph to 30 by lag
44. Compute subarea hydrograph at 30 (Enoree River)
45. Combine routed hydrograph with local flows at 30
46. Route hydrograph to 31 by Muskingum Method
47. Compute subarea hydrograph at 31
48. Combine routed hydrograph with local flows at 31
49. Route hydrograph to 131 by lag
50. Combine hydrographs from Saluda and Broad Rivers at 131
51. Route hydrograph to 32 (Congaree River) by Muskingum Method

EXHIBIT 8

HEC-1 INPUT DATA SEQUENCE

CONGAREE RIVER

Operation

1. Title and job specification

2. Compute subarea hydrograph at 32 (Congaree River)

3. Combine routed and local flows at 32 (see Broad River)

4. Route hydrograph to 132 by lag

EXHIBIT 9

HEC-1 INPUT DATA SEQUENCE

CATAWBA-WATEREE RIVER

Operation

1. Title and job specification
2. Compute runoff from storm at Bridgewater Area 1
3. Route hydrograph through Bridgewater Lake Area 1 using modified Puls
4. Route hydrograph to 2 (Rhodiss by lag)
5. Compute subarea hydrograph at 2
6. Combine routed and local flows at 2
7. Route hydrograph through Rhodiss Lake by Modified Puls
8. Route hydrograph from 2 to 3 (Oxford Dam) by lag
9. Compute subarea hydrograph at 3
10. Combine routed and local flows at 3
11. Route hydrograph through Oxford Lake by Modified Puls
12. Route from 3 to 4 (Lookout Shoals Dam by lag
13. Compute subarea hydrograph at 4
14. Combine routed and local flows at 4
15. Route hydrograph through Lookout Shoals Lake by Modified Puls
16. Route from 4 to 5 (Cowens Ford Dam) by lag
17. Compute subarea hydrograph at 5

Operation

18. Combine routed and local flows at 5
19. Route hydrograph through Oxford Dam by Modified Puls
20. Route from 5 to 6 (Mtn. Island Dam) by lag
21. Compute subarea hydrograph at 6
22. Combine routed and local flows at 6
23. Route hydrograph through Mtn. Island Lake by Modified Puls
24. Route hydrograph from 6 to 7 (Wylie Dam) by lag
25. Compute subarea hydrograph at 7
26. Combine routed and local flows at 7
27. Route hydrograph through Wylie Lake by Modified Puls
28. Route hydrograph from 7 to 8 (Fishing Creek Dam) by lag
29. Compute subarea hydrograph at 8
30. Combine routed and local flows at 8
31. Route hydrograph through Fishing Creek Lake by Modified Puls
32. Route hydrograph from 8 to 9 (Rocky Creek Dam) by lag
33. Compute subarea hydrograph at 9
34. Combine routed and local flows at 9

Operation

35. Route hydrograph through Rocky Creek Lake by Modified Puls
36. Route hydrograph from 9 to 10 (Wateree Dam) by lag
37. Compute subarea hydrograph at 10
38. Combine routed and local flows at 10
39. Route hydrograph through Wateree Pond by Modified Puls
40. Route hydrograph from 10 to 11 by lag
41. Compute subarea hydrograph at 11 (Camden)
42. Combine routed and local flows at 11
43. Route hydrograph at 11 to 13 by the Muskingum method
44. Compute subarea hydrograph at 12
45. Route hydrograph at 12 to 13 by the lag
46. Compute subarea hydrograph at 13
47. Combine routed and local flows at 13
48. Route hydrograph at 13 to 132 (confluence with Congaree River
49. Combine route hydrograph from Congaree River with Watree at 132
50. Route hydrograph to 33 (Santee River above Lake Marion)

EXHIBIT 10

HEC-1 INPUT DATA SEQUENCE

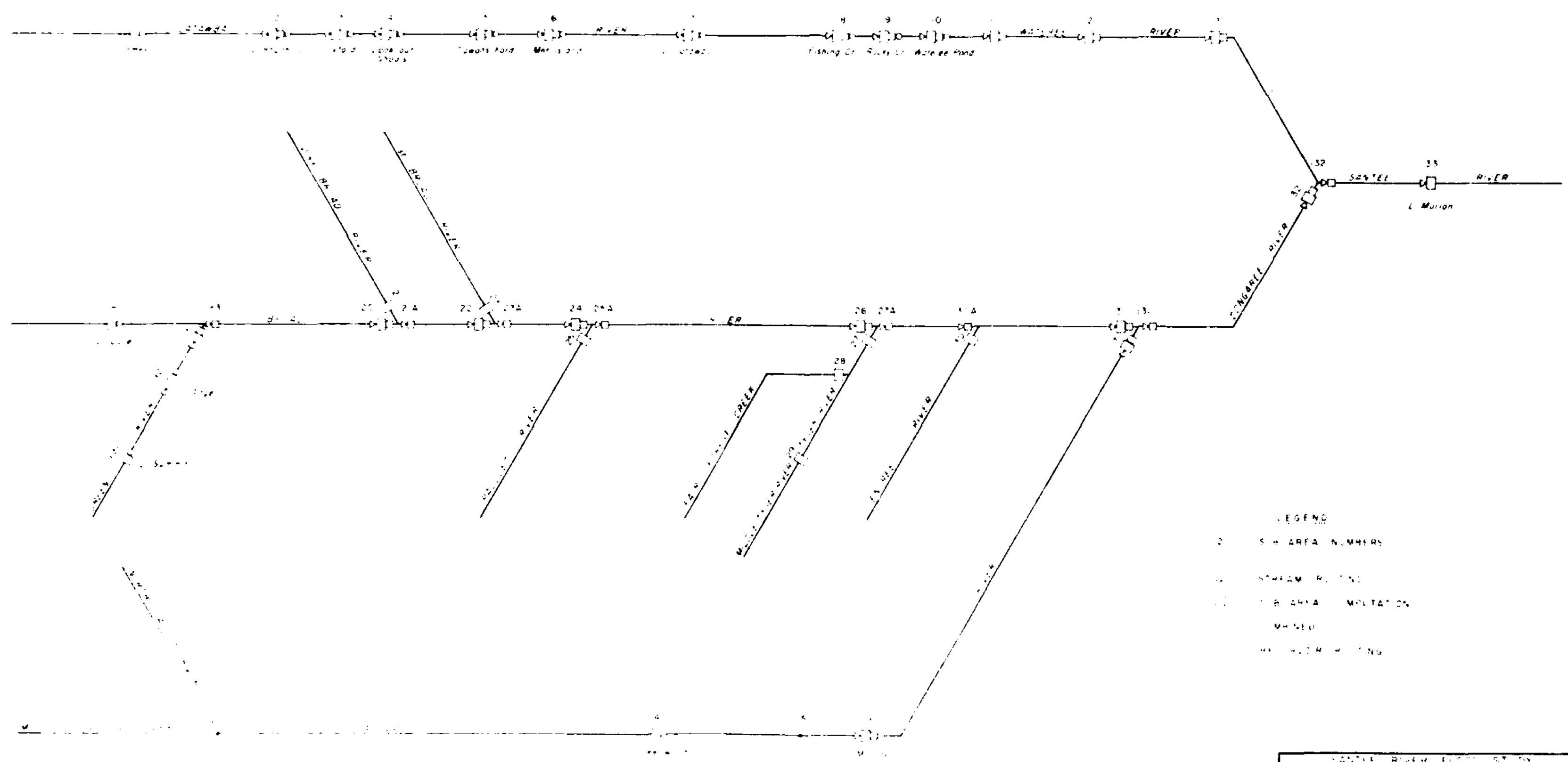
SANTEE RIVER

Operation

1. Title and job specification

2. Compute subarea hydrograph at 33 (Santee River @ Lake Marion)

3. Combine routed hydrograph with local flows at 33



SANTEE RIVER FLOOD STUDY

SCHEMATIC DIAGRAM
OF
OPERATIONS

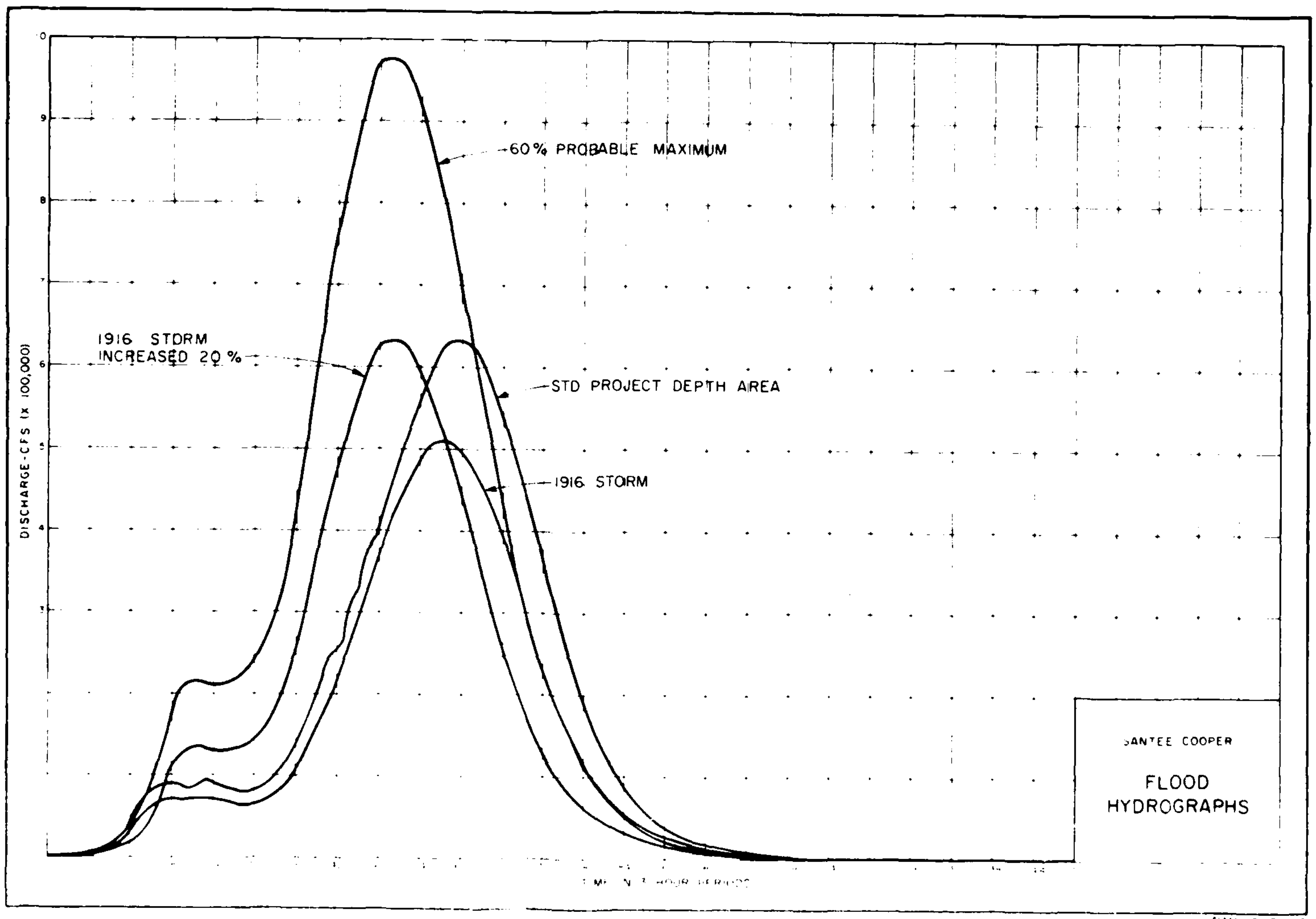

60% PROBABLE MAXIMUM
1916 STORM INCREASED 20%
STD PROJECT DEPTH AREA
1916 STORM
DISCHARGE-CFS (X 100,000)
SANTEE COOPER
FLOOD HYDROGRAPHS

EXHIBIT 12

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX C

ALTERNATE STUDIES

U. S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

APPENDIX C

ALTERNATE STUDIES

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COOPER RIVER REDIVERSION PROJECT
INTAKE AND TAILRACE CANALS

APPENDIX A
ALTERNATE STUDIES

ALTERNATE ALIGNMENT STUDIES

1. General. The intake and tailrace canals alignments approved in the General Design Memorandum, January 1972, were analyzed for economic and environmental suitability under the engineering factors listed below. Also studied were two alternate intake canal alignments and two alternate tailrace canal alignments. Alignments studies are shown on Plate C-1. The purpose of the alternate alignments studies was to determine the most feasible canal alignments under the following engineering criteria:

a. Excavation volume and type of materials. The invert elevation (50 msl in this study) and canal side slopes (1V on 3H) for the intake canal were held constant for each of the three alternative intake canal alignments. Similarly, the invert elevation (0.0 msl in this study) and canal side slopes (1V on 3H) were held constant for each of the three alternative tailrace canal alignments. Excavation volumes were affected by canal length and topography, differences in top of ground elevation and width of natural drainage features. The type of subsurface materials encountered within the canal excavation affected unit costs of excavation.

b. Hydraulic efficiency. It was recognized that a straight canal alignment would be the most efficient from the hydraulic standpoint, however, design studies showed that curves of three degrees or less would not create significant hydraulic losses nor increase design velocities above three feet per second. The addition of curves would lengthen the GDM approved alignment, which had one curve in the intake canal and two curves in the tailrace canal. Curves were also considered to have an intangible aesthetic value which would justify a curved alignment over a straight alignment if economic and hydraulic factors were equal.

c. Excavated material disposal. Disposal alternatives for the intake canal excavated materials were practically the same as recommended in the GDM. In the tailrace canal, disposal alternatives were significantly different against the hillside as compared to alignments located a considerable distance out in the flood plain from the hillside.

d. Real estate costs. Real estate in the intake canal was highly sub-divided and several houses have been built on the GDM alignment since 1972. Low-lying land at the project site generally has fewer human habitants and is primarily wildlife habitat. There are no buildings or dwellings in the tailrace canal alignment areas in the Santee River flood plain.

e. Relocation bridge costs. More favorable orientations of road relocation bridge crossings (than the GDM relocation plan) could be obtained for alignments other than the GDM alignment (Plan A). Naturally, the shortest bridge would be a bridge crossing perpendicular to the canal centerline.

f. Interior drainage. The collector ditch/drop structure concept for interior drainage as approved in the GDM was applied to each alternative alignment for the intake and tailrace canals. After selection of the canal alignments, several interior drainage schemes were investigated as discussed later in this appendix.

g. Environmental impact. The impact of each alternative alignment on fish and wildlife resources was determined through co-ordination with Federal and State fish and wildlife agencies. These agencies recommended against intake canal alignments that would affect low-lying timberland habitat (see Plate C-1) and recommended against tailrace canal alignments that would require widening and/or deepening of the Santee River. They preferred a tailrace canal alignment that would result in the least acreage of Santee River flood plain converted to construction of the canal and related structures. A survey of archeological resources was co-ordinated with and performed by the State of South Carolina. Three important archeological sites (38 BK 83/84 and 38 BK 76) are shown on Plate C-1. Coordination letters are presented in Exhibit B in the main report.

2. Intake canal. Three major alternative alignments for the intake canal (see Plate C-1) were studied, as well as several minor variations in alignment, in order to determine the most feasible alignment to satisfy the above-listed engineering criteria. Rough estimates of approximate excavation volumes and total costs of comparative items prepared for the study of the three major alternatives are compared in Table 1 below.

TABLE 1

INTAKE CANAL ALIGNMENT ALTERNATIVES

Alignment Alternatives	Volume of Excavation 1/	Total Cost 2/
Plan A	7,928,800 C.Y.	$8,140,000
Plan B	8,019,800 C.Y.	$7,670,000
Plan D	7,645,500 C.Y.	$7,520,000

1/ Based on invert EL.50 msl with 1V on 3H excavation slopes.
2/ Excavation volume multiplied by unit excavation cost of $0.60 per C.Y., plus real estate and road relocations costs.

The advantages and disadvantages of each major intake canal alignment alternative are listed below.

a. Straight alignment (Plan A). This intake canal alignment was recommended in the General Design Memorandum, January 1972. The alignment started at Lake Moultrie, curved around the Georgia Pacific plant, then followed a straight line to the powerhouse. The principal advantages of this alignment were (1) most efficient hydraulics (fewer curves) and (2) the shorter canal length requires less acreage for canal excavation and excavated material disposal. Disadvantages of this alignment were (1) greater volume of excavation, (2) more residences and higher-valued real estate property would be affected by this alignment compared to other alignment alternatives and (3) more expensive orientations of highway relocation bridge crossings than other alignment alternatives.

b. Meandering alignment (Plan B). This alignment was developed by drawing the intake canal centerline through the lowest ground elevations within a reasonable distance from the GDM alignment (Plan A). The alignment starts at Lake Moultrie, curves around the Georgia Pacific plant into a small creek basin south of the Plan A alignment. The alignment curves to follow the lower elevations of the basin then curves into a smaller drainage basin near the powerhouse. The advantages of this alignment were (1) lower real estate costs due to predominance of low-lying land and fewer residences with in the canal right-of-way, (2) lower highway relocations costs due to more favorable orientation of bridge crossings and (3) additional curves (meanders) improve the aesthetic appearance of the intake canal. Disadvantages of the meandering alignment were (1) a longer canal and greatest volume of excavation, (2) required the most canal right-of-way acreage and (3) it destroys considerable acreage of low-lying woodland wildlife habitat.

c. Intermediate meandering alignment (Plan D). This alignment is one of several intermediate alignments that were developed to incorporate the best features of Plans A and B. The alignment starts at Lake Moultrie, curves around the Georgia Pacific plant then curves back across lower ground elevations between the Plan A and Plan B alignments to the powerhouse. The principal advantages of the intermediate alignment were (1) least volume of excavation, (2) affects fewer residences than Plan A, (3) no significant difference in interior drainage facilities compared to Plans A and B, (4) material volumes per unit length of alignment are more uniform than for Plan A and (5) contains aesthetic curves (2 to 3 degrees) with no significant additional head or velocity increases. Disadvantages of this alignment were that it (1) is 600 feet longer than Plan A, (2) has less favorable orientations of road relocation bridge crossings than Plan B and (3) it destroys a small acreage of low-lying woodland wildlife habitat.

3. Selected intake canal alignment. The Plan D alignment was selected as the most feasible intake canal alignment to satisfy the aforementioned engineering critiera. Based on preliminary estimates of excavation volume and total cost, the Plan D intake canal alignment would be the most economical alignment.

4. Tailrace canal. Three major alternative alignments, and several minor alignment variations, for the tailrace canal (see Plate C-1) were studied to determine the most feasible alignment to satisfy the above-listed engineering criteria. Rough estimates of approximate excavation volumes and total costs of comparative items for the three major alternatives are compared in Table 2 below.

TABLE 2

TAILRACE CANAL ALIGNMENT ALTERNATIVES

Alignment Alternatives	Volume of Excavation	Total Cost
Plan A	7,751,300 C.Y.	$6,976,200
Plan B	7,164,900 C.Y.	$6,448,400
Plan D	7,151,200 C.Y.	$6,566,000

1/ Based on invert EL.0.0 msl with 1V on 3H excavation slopes.
2/ Volume of excavation multiplied by unit excavation cost for unclassified excavation of $0.90 per C.Y., plus cost of interior drainage facilities.

The advantages and disadvantages of each major alignment alternative are listed below.

a. Straight alignment (Plan A). This alignment was recommended in the General Design Memorandum. The alignment has a three degree curve starting immediately below the powerhouse tailrace slab, then proceeds in a straight line to the junction of Lake Mattassee and the Santee River. Advantages of this alignment are that (1) it has the least number of curves to affect hydraulic efficiency, (2) no interior drainage facilities would be required and (3) little annual maintenance of embankment slopes and disposal areas would be required. Disadvantages of this alignment are that (1) it would result in the largest volume of excavation, (2) some soft rock excavation would be required and (3) canal excavation and disposal of excavated material affects hardwood (cypress) timberland wildlife habitat in the Santee River flood plain.

b. Short route to Santee River (Plan B). This alignment was the subject of a feasibility study in DM No. 3, Entrance Channel in Lake Moultrie. A summary of the study is presented in Exhibit No. 1 to this appendix. The alternative alignment studies presented in this report update the study in Exhibit No. 1. The alignment route followed the shortest distance from the powerhouse to the Santee River, thence along the existing river channel to Lake Mattassee. Principal advantages of this route were (1) less volume of excavation than Plan A and (2) no interior drainage facilities would be required. Disadvantages of the short route were (1) environmentally damaging excavation in the Santee River would be required in order to obtain the necessary hydraulic cross section and (2) the canal excavation and excavated material disposal from the powerhouse to the Santee River destroys a wide corridor of hardwood (cypress) timberland wildlife habitat.

c. Meandering alignment (Plan D). A tailrace alignment from the powerhouse running along the edge of the Santee River flood plain (against the hillside) to the Santee River at Lake Mattassee would pass through the low elevations of Mattassee Run, a natural drainage feature, to minimize the volume of excavation. Several curves (less than three degrees curvature) are required in the alignment to take full advantage of the lower topography. Subsurface investigations for the tailrace canal encountered weak shale rock at higher elevations (up to elevation 7 msl) along the edge of the flood plain. Principal advantages of the meandering route were (1) least volume of excavation and (2) least impact of canal excavation and material disposal on hardwood timberland wildlife habitat in the flood plain. Disadvantages of the meandering route were (1) interior drainage facilities would be required and (2) an increase in the excavation quantity of weak rock.

5. Selected tailrace canal alignment. The Plan D meandering alignment was selected as the most feasible tailrace canal alignment to satisfy the aforementioned engineering criteria. Alignment selection was based on the comparative cost estimates of alignment features with consideration given to environmental impact and subsurface conditions associates with each alternative. The selected alignment represents the lowest total cost tailrace canal plan of the three alternatives considered.

STUDIES FOR OPTIMUM CANAL DIMENSIONS

6. General. Numerous combinations of canal widths and depths would satisfy the hydraulic design criteria of providing 49 feet of net head at the powerhouse when Lake Moultrie is at elevation 74.0 msl and the canal is conveying 24,500 cfs flow to the powerhouse. Hydraulic studies indicated that the most efficient channel section would have a depth of flow of 46 feet (equivalent to an invert at elevation 28 msl for the intake canal and elevation -26 msl for the tailrace canal). In order to focus design studies on the most practicable range of canal invert elevations, the deepest canal inverts (and, therefore, the narrowest bottom widths) for which common excavation costs would apply were determined from subsurface exploration data. An examination of boring logs established the top of firm rock along the proposed canal alignments. Top of firm rock is considered the cost boundary between common excavation unit costs and rock excavation unit cost. It became apparent that a relatively small amount of rock excavation would be tolerable when comparing excavation costs of various canal cross sections. Canal side slopes were assumed to be 1 vertical on 3 horizontal for all alternative cross sections.

7. Intake canal. An examination of top of firm rock data from subsurface investigations set the lower limit of the practicable range of intake canal invert elevations at elevation 50 msl. Required rock excavation volumes increase greatly at invert elevations below 50 msl. The upper limit of practicable canal inverts was determined to be elevation 54 msl based on the trend to a much wider canal and increasing excavation volumes at invert elevations higher than 54 msl. Within the range of elevations 50 msl to 54 msl, the most favorable economic combination of excavation volume and excavation unit costs occurs at about elevation 50 msl. Therefore, an intake canal invert at elevation 50 msl was selected for final design. The selected invert for the intake canal at elevation 50 msl would require hard rock excavation up to 4 feet thick between canal stations 245+00 and 295+00. The top of firm rock along the intake canal centerline is shown in profile on Plate 19.

8. Tailrace canal. The lower limit of practicable invert elevations for the tailrace canal was set at 0.0 msl based on (a) the location and type of rock encountered in the subsurface investigations and (b) the vertical distance between the selected canal invert and the existing thalweg (approximately elevation 5 msl) in the Santee River at the tailrace canal exit. The upper limit of practicable canal inverts was set at elevation 3.5 msl based on increasing canal bottom widths and corresponding larger excavation volumes at invert elevations higher than 3.5 msl. Within the range of elevations 0.0 msl to 3.5 msl, the most favorable economic combination of excavation volume and excavation unit costs occurs at about elevation 0.0 msl. Therefore, a tailrace canal invert at 0.0 msl was selected for final design. Excavation of weak rock would

be required above the selected invert elevation 0.0 msl at several locations in the tailrace canal (see geologic top of rock in section on Plates 21 through 25). Firm rock (hard sandstone) would be encountered above elevation 0.0 msl in the area of the existing SCL railroad, station 419+00. The top of firm rock along the tailrace canal centerline is shown in profile on Plate 20.

INTERIOR DRAINAGE STUDIES

9. Intake canal. Interior drainage for the intake canal as set forth in Design Memorandum 1, Volume 1, of the General Design Memorandum calls for a collector ditch running along the south project boundary. The ditch as specified in the GDM would require two large drop structures (baffle chutes). The largest structure would be located at the Powerhouse and would have a drop of about 48 feet. As a result of changes in the intake canal alignment during the alternative alignment studies discussed in this appendix, other possible interior drainage schemes were investigated. The following intake canal interior drainage plans were considered:

a. Plan A. Basically, Plan A is the same as the GDM plan .n which a collector drainage ditch is placed along the south project boundary, however, the large drop at the powerhouse would be divided into two smaller drops to reduce the size of the drop structures into the workable range for USBR-typ baffle chutes. Plate B-12 shows the intake canal drainage areas. P.an A would intercept all drainage from Area 11 and the drainage from sub-areas 1 and 2 of drainage Area 1. Flow would be conveyed from the existing Lake Moultrie dike to a drop structure (baffled chute) located approximately 2,200 feet southeast of the powerhouse. There it would be dropped from elevation 59 feet msl to elevation 43 ft msl. This drop would place the flow in the first draw (or natural drain) below the powerhouse where it would be conveyed by a ditch to a second drop structure located adjacent to tailrace canal station 409+00. The second drop structure (baffled chute) would drop the flow from elevation 24 ft msl to elevation 6 ft msl. Because of the depth of cut required in this plan, three additional small drops would be required where major inflow points occur along the ditch. Plan A is moderately expensive as it would require a deep cut (up to 20 feet) in order to drain flow from the existing ditch behind Georgia Pacific toward the Powerhouse. The collector ditch invert would have to be set at elevation 67.0 ft msl. The deep cuts would create slope stability problems and a long-term slope maintenance problem. The cuts are deep enough to lower the groundwater table in land adjacent to the project boundary. Chief advantage of this plan is that drainage of intercepted water would be accomplished entirely within project boundaries and flow easements on adjacent lands would not be required.

b. Plan B. This plan would divide the Plan A collector 'itch into two ditches. One ditch would intercept all of the Area 'n-age as in Plan A and convey it in deep cut towards the Plar structures. Also, intercepted by this ditch would be the from sub-area 1 of drainage Area I. A second ditch would flow from sub-area 2 of drainage Area I and drain this flow Lake Moultrie where it would join an existing ditch behind t. gia Pacific plant. The existing drainage ditch behind Georgia F drains towards Crawl Creek. To cause this flow to reverse and to drain into the headwater area of Halfway Swamp, a 10-foot wide (bottom width) ditch having an invert elevation at 67 ft msl would be dug until it daylights approximately 900 feet below State Highway 35. The existing Georgia Pacific drainage ditch would be widened and deepened to these dimensions. The alignment of the new ditch is shown on Plates B-29, B-30 and B-31 in Appendix B.

c. Plan C. Same as Plan B except sub-area 1 as well as sub-area 2 of drainage Area I would be drained in the direction of Halfway Swamp. This plan is superior to Plan B because the interceptor ditch which drains flow towards the Powerhouse would not have to be as deep as in Plan B. Plan C would require additional lands in fee or easement down Halfway Swamp which are outside of the GDM approved project right-of-way limits. Some environmental impact to deepening and widening the existing drainage ditch in Halfway Swamp would ensue with construction of Plan C, however, several similar ditch improvements have been made in Halfway Swamp within the last ten years. Plan C is the least expensive of all plans considered. The depth of cut for the collector ditches in Plan C would be considerably less than for Plans A and B and slope maintenance would be minimized. No additional drop structures to the two major Plan A structures would be required.

d. Plan D. This plan is similar to Plan C but instead of draining flows into the headwater area of Halfway Swamp, a ditch would collect the flow from sub-areas 1 and 2 of drainage Area 1 and this flow would be lifted into the intake canal by a pumping station. Plan D is the most expensive plan because of the large pumping station required. However, Plan D has all the advantages of Plans A and C.

10. Selected intake canal interior drainage plan. Plan C was the selected plan. It was the most cost effective plan and the drainage easements required in Halfway Swamp outside the GDM approved project boundaries can be readily obtained following standard procedures.

11. Tailrace canal. As was the case with the intake canal, changes in the GDM tailrace canal alignment as a result of alignment alternative studies led to restudy of the tailrace canal interior drainage. With the GDM tailrace alignment, it was possible to drain the intercepted run-off of Drainage Area III, see Plate B-12 in Appendix B,

through the existing Mattassee Run drain. GDM plan formulation called for improving the existing Mattassee Run drainage so that it would have a 15-foot bottom width with 1 vertical on 2 horizontal side slopes. The proposed GDM drainage channel was to follow the existing Mattassee Run drainage alignment and use the existing thalweg gradient. As discussed in another portion of this appendix, alignment studies resulted in moving the tailrace alignment closer to the Santee River hillside. This alignment roughly coincides with Mattassee Run and, therefore, precludes the use of the GDM tailrace interior drainage scheme. The following alternative tailrace canal interior drainage plans were considered:

a. Plan A. Plan A would drop the intercepted drainage from Area III, sub-areas 1 through 8, directly into the tailrace canal. Run-off from several adjacent sub-areas would be combined in order to minimize the number of structures required. The drops would be from approximately elevation 24.0 ft msl to elevation 6.0 ft msl and would be accomplished by 5 drop structures. Because of the number of structures, Plan A has the highest first cost of the three plans considered. Annual maintenance of the related canal slopes and structures would be minimal.

b. Plan B. Plan B calls for a single collector ditch paralleling the tailrace canal along the hillside. This ditch would collect flows from Area III, sub-areas 1 through 8, and convey the flows down to tailrace centerline station 595+00, where it would be discharged into the tailrace canal. In order to accomplish this without need of a major drop structure, the drainage ditch's invert would be cut from elevation 22.0 ft msl at the existing culverts under the SCL Railroad to elevation 6.0 ft msl near tailrace canal station 595+00. At this point, intercepted drainage would be able to flow into the tailrace canal without the need of a drop structure. However, because of the deep cut necessary in this plan some of the intercepted flow coming into the drainage ditch would require small drop structures to prevent side channel erosion. More ditch slope maintenance would be required than in Plans A and C. In addition, the depth of cut and the side slopes for the ditch use up a large portion of the area needed for disposal of excavated materials from the tailrace canal. In order to have maintenance access from the lower portions of the project along the right bank of the tailrace canal, a large culvert would have to be constructed for a patrol road from the access road to the levee at the lower end of the project. The principle advantage of Plan B is that no large drop structures would be required.

c. Plan C. Plan C, much like Plan B, calls for a single collector ditch. However, instead of having a deep cut, the proposed ditch invert would be essentially horizontal at elevation 22.0 ft msl. The ditch would follow the same alignment as Plan B and at approximate tailrace canal station 594+00 flow would be dropped by means of a baffle chute from elevation 22 ft msl to elevation 6.0 ft msl. No side channel structures would be required for this plan. Plan C has the least first cost of the three plans. Maintenance of the canal slopes and structure in Plan C would be comparable to Plan A and less than Plan B. Access from the lower end of the project to the right bank tailrace canal levee would be provided by adding a concrete slab over the top of the drop structure.

12. Selected tailrace canal interior drainage plan. Plan C was the selected plan for the tailrace canal interior drainage. It was the most cost-effective plan that would perform the collection and safe discharge of interior drainage flows.

SUMMARY OF TAILRACE ALIGNMENT STUDY
SANTEE COOPER PROJECT

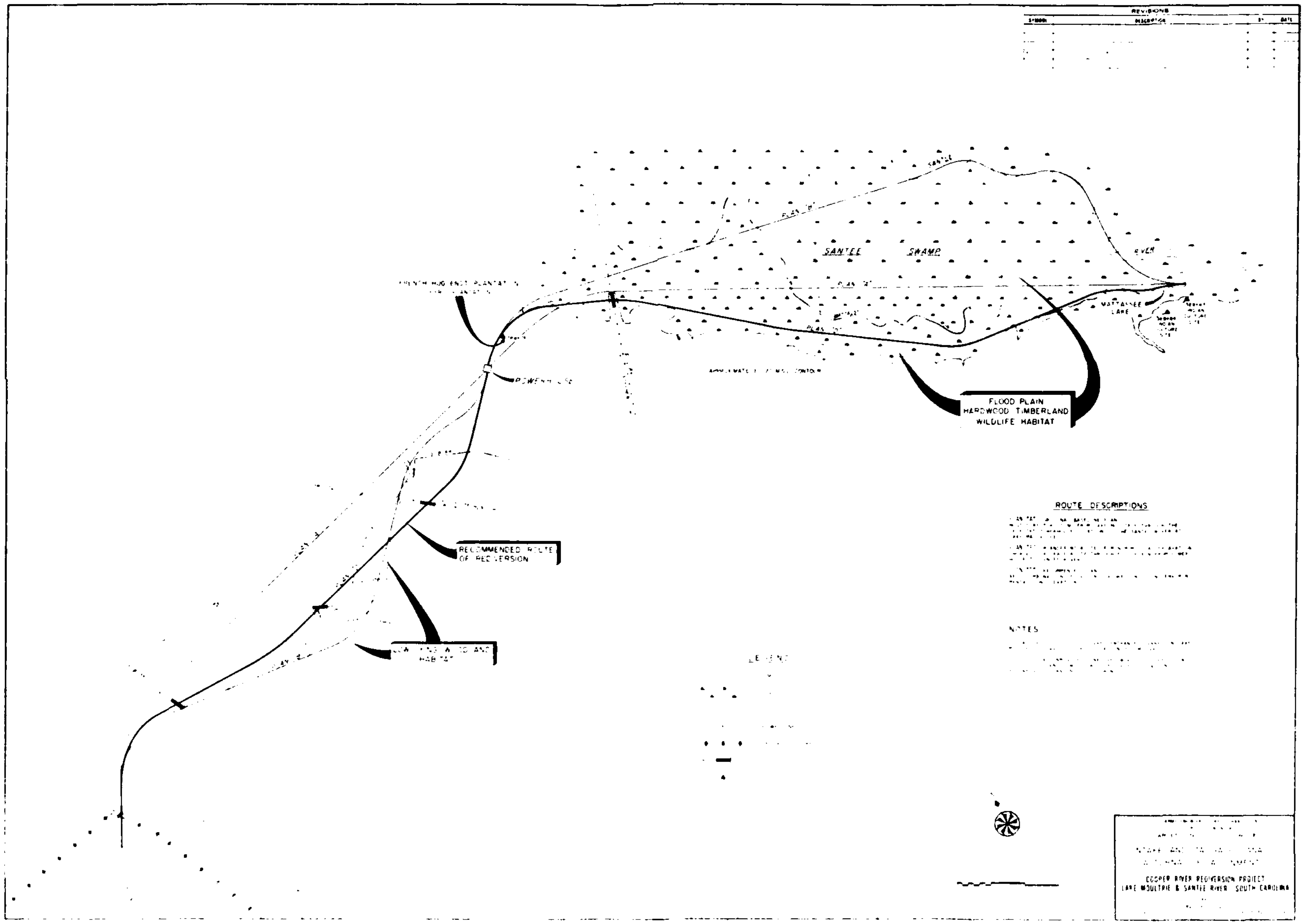
General. The presently proposed discharge canal extends from the powerhouse to the confluence of the Santee River and Mattassee Lake, a total length of 26,800 feet. An alternative proposal changes the alignment of the discharge canal to enter the Santee River at a point approximately 2.33 miles upstream of Mattassee Lake, reducing the canal's length to 17,840 feet. To meet tailwater design criteria, it was found that the water surface elevation at the mouth of the shorter discharge canal must be reduced by two feet at a flow of 25,000 cfs. A cost comparison of the two alternatives was studied.

Method of Study. Steady-state profiles for surveyed cross sections for the reach of the Santee River between Jamestown and Wilson Dam were run using the HEC-2 water surface profiles computer program. It was necessary to adjust the crosssections and Manning's "n" values to simulate gage readings for flows of 25,000, 50,000 and 100,000 cfs. After reasonable simulation was obtained, various channel improvements were tried to determine which was necessary to achieve the necessary stage reduction at the canal entrance. A 200-foot bottom width channel with 3-on-1 side slopes, starting at Jamestown and extending upstream to the canal entrance was found necessary to accomplish a 2-foot stage reduction. The channel invert elevation was assumed to be on-grade, due to transition problems and large amounts of rock excavation required for a deeper channel. Next, excavation quantities were computed for the channel improvement. The difference between excavation quantities for the two tailrace canals was determined using the length ratio of the canals. It was then assumed most of the excavation would be unclassified, costing $0.90 per cubic yard. The results are as follows:

	Quantity	Cost
200-foot channel improvement, Santee River	3,067,808 cy	$ 2,761,027
Shorter discharge canal	5,059,104 cy	$ 4,553,194
Total cost of shortened canal	8,126,912 cy	$ 7,314,221
Presently proposed canal	7,600,000 cy	$ 6,840,000
Difference	526,912 cy	$ 474,221

Conclusion. The shorter tailrace canal entails greater total excavation quanitites and subsequently higher costs than does the original alignment. It also has possible adverse environmental effects due to channelization of the Santee River. The proposal for canal realignment is therefore not feasible at this time.

EXHIBIT NO. 1


REVISIONS
SANTEE SWAMP
RIVER
MATTASSEE LAKE
FLOOD PLAIN
HARDWOOD TIMBERLAND
WILDLIFE HABITAT
POWERHOUSE
RECOMMENDED ROUTE
OF REDIVERSION
ROUTE DESCRIPTIONS
NOTES
COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX D

SLOPE AND LEVEE STABILITY ANALYSIS

U. S. ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

APPENDIX D

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COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX D

SLOPE AND LEVEE STABILITY ANALYSIS

INTRODUCTION

1. General. Stability analyses were conducted on the cut slopes and levee embankments recommended in the Cooper River Rediversion Project - General Design Memorandum (dated January 72). All cut slopes were analyzed by the wedge method along an assumed failure plane. The slip circle method was employed to analyze the levee slopes. A time-sharing computer system aided analyses of the slopes. The Kansas City wedge (KCWEG) program was used to analyze all the cut slopes. A modified WES slip circle program, entitled SAVA104 was used to analyze the embankment slopes. Numerous wedges and circular failure planes were computer analyzed for each loading condition, then the most critical stability condition was manually checked. By making simplifying assumptions regarding properties of in situ and embankment materials and groundwater conditions, the number of cross sections requiring analysis was kept to a minimum. The soil profiles and design data for each manual analysis are shown on Plates 14 through 36. The computations and graphical solutions (to determine the minimum safety factor) are also presented on the plates referenced above. The boring logs are furnished in Appendix A, Geology and Soils. All laboratory test results are also contained in Appendix A, including both undisturbed and remolded tests. The graphical analyses, strength selection and selection of loading conditions were performed in accordance with EM 1110-2-1902, dated April 1970.

2. Factors of safety. The following values were adopted as the minimum required Factors of Safety:

Loading Condition	Minimum Required Factor of Safety Cut	Minimum Required Factor of Safety Embankment
Intake Canal		
End-of-Construction Case	1.30	1.30
Steady Seepage Case	1.10	1.50
Partial Pool Case	----	1.10
Sudden Drawdown Case	----	----
Earthquake Loading	1.00	1.00

Loading Condition	Minimum Required Factor of Safety Cut	Minimum Required Factor of Safety Embankment
Tailrace Canal		
End-of-Construction Case	1.30	1.30
Steady Seepage Case	1.00	1.10
Partial Pool Case	----	1.10
Sudden Drawdown Case	1.20	1.10
Earthquake Loading	1.00	1.00

These safety factors were considered adequate since all analyses were based on test results of the weakest in situ materials found during the drilling program. In addition, the consequence of failure of the canal or levee slopes was considered minimal since failure would cause no loss of life or pool. Factors of safety obtained in the stability analyses are compared to minimum required factors of safety in Tables D-2 and D-4.

INTAKE CANAL-CUT SLOPES

3. Typical Sections. Cut slopes were analyzed for stability at four representative locations along the intake canal. The typical sections were as follows: (1) Station 140+00 (Plates 14, 15, 16) located lakeward of the existing Lake Moultrie dike, (2) Station 194+30 (Plates 17, 18) adjacent to S.C. State Highway 35, (3) Station 246+00 (Plates 19, 20, 21) 1000 feet south of S.C. State Highway 45 and (4) Station 272+75 (Plates 22, 23) 1500 feet north of Hwy. 45 and about 4000 feet south of U.S. Highway 52. These sections were used to represent the canal cut slope stability from Lake Moultrie (Station 135+34) to approximately Station 339+00 near the Powerhouse. Results of the analyses govern each reach represented by the typical sections.

4. General Soil Types and Geologic Conditions. In general, there are three predominant soil types within the influence of the intake canal excavation. The top 10 (±) feet of soil is a clayey, tan, fine to medium grain sand (SC) suitable for embankment fill. A very soft, "low blow count", gray, fat clay (CH-MH) approximately 12 (±) feet in thickness underlies this sand in several areas. The soft, fat clay occurs at and below the canal invert (Elevation 50 msl). The last predominant soil type is a clayey sand beneath the gray, fat clay having generally the same characteristics as the upper clayey sand (SC). Groundwater levels in the intake canal vary from elevation 70 msl near Lake Moultrie (Station 140+00) to about elevation 50 msl near the Powerhouse (Station 339+00). Hard limestone rock was encountered as high as elevation 54 msl between Stations 245+00 and 295+00.

5. Engineering Properties of Soil. The soil samples were tested by SAD Laboratories. The purpose of the testing program was to identify physical properties of representative soil types. The tests for physical

properties were moisture content, Atterburg limits, and grain size analyses. Undisturbed samples were extracted from the field for undisturbed (quick, rapid, slow), unconfined compression and consolidation tests. A summary of results of the intake canal undisturbed and remolded testing is shown in Table D1. Actual lab strengths from each typical cross-section were used to select the design strengths for conducting the stability analyses. These strengths are presented graphically on Plates D-1 through D-3. In some instances, the laboratory reported two friction angles (maximum and ultimate) for the direct shear test. According to EM 1110-2-1906, both angles should be recorded when a brittle material reveals a significant decrease in shear stress with increasing strain (after a 'peak' failure). When this condition occurred, the ultimate angle was always selected as the design strength. Laboratory test data sheets are presented in Appendix A, Geology and Soils.

6. Methods to Determine Top of Rock. Information for determining the assumed "top of rock" was obtained from borings. The holes were "split-spoon sampled" to drive refusal and some were continued by coring the rock. In augered borings auger refusal was assumed to be "top of rock" when rock fragments similar to known geologic formations in the area were recovered on the auger bit.

7. Engineering Properties of Rock. The soils are primarily underlain by light gray limestone and dark gray shale. In some reaches of the intake canal the shale and the limestone surfaces appear to be badly decomposed or even deteriorated for one to two feet. In general, both rock types are moderately hard in a fresh state.

8. Settlement Calculations. Computations were made to predict settlement of the weakest material within the intake canal (project) limits. The weakest layer, an 11-foot-thick gray CH-MH soil located within the canal excavation in the Lake Moultrie dike area will be surcharged by a 13-foot high levee. The calculations indicated that total anticipated settlement of the canal banks due to levee surcharge should not exceed 4 inches. Settlement calculations are presented in Exhibit 1, Settlement Calculations.

9. Slope Design Criteria. The design assumptions and design criteria established for conducting the stability analyses are presented as follows. No tension cracks were assumed since existing surface drainage appeared to be adequate throughout the area and because the slopes are expected to be covered with dense vegetation once the project is completed. Failure planes were assumed at numerous elevations in the cut slope. However, as evidenced from field observation and test results, the critical failure always occurred in the gray, soft fat clay that is 'sandwiched' between two stronger soils. One soft clay sample, like those tested, was photographed during a 'wet-dry' cycle test. It appears that this soil cannot tolerate exposure to wetting and drying during construction and therefore should be excavated under water. The top of

TABLE NO. D1

UNDISTURBED AND REMOLDED STRENGTHS

INTAKE - CANAL

Hole No.		γ_m	γ_s	γ_b	Uncomf. Comp. K/SF	Q TEST C K/SF	Q TEST Ø	Q TEST tanØ	R TEST C K/SF	R TEST Ø	R TEST tanØ	S TEST C K/SF	S TEST Ø	S TEST tanØ
1. UNDISTURBED														
	2	110	113	51	.70	-	-	-	-	-	-	-	-	-
IT-10	1				.48	0.70	4°	.07	1.4	7°	.123	0	22.5°	.414
	1	112	116	52	-	-	-	-	-	-	-	-	24.0°	.445
IT-15	2				.63	1.0	0	0	1.2	6°	.105	0	16.5°	.296
	2	116	118	56	.60	.6	1.5°	.026	.80	11°	.194	0	25°	.466
IT-21	1				.50	-	-	-	-	-	-	-	-	-
	2	98	102	40	.27	.5	0	0	.30	13.5°	.240	0	14°	.249
IT-27	1				.62	-	-	-	-	-	-	-	-	-
CS-14		123	124	62	-	-	-	-	-	-	-	0	38°	.781
CS-15		123	124	62	-	.5	9°	.158	.54	14.5°	.259	-	-	-
IT-2B-1A		119	123	61	-	2.0	3°	.05	.80	13.5°	.240	0	28°	.532
IT-2B-1B		124	127	65	-	-	-	-	1.8	25°	.466	-	-	-
2. DISTURBED														
Comp A @ OMC		122	131	68	.63	2.5	29°	.553	5.4	26.4°	.500	0	33.7°	.669
Comp A @ -2%		-	-	-	.79	1.39	31.3°	.609	-	-	-	-	-	-
Comp A @ +2%		-	-	-	.47	.64	33.7°	.667	5.20	20.7°	.377	-	-	-
Comp B @ OMC		126	131	68	3.51	1.6	28°	.53	.60	15.4°	.275	0	34.8°	.694
Comp B @ -2%		-	-	-	6.02	1.0	32.3°	.63	-	-	-	-	-	-
Comp B @ +2%		-	-	-	2.05	1.8	20.3°	.37	.60	11.7°	.207	-	-	-

cut (orginal ground) in the intake canal ranges from elevation 74.0' msl to 86.0' msl. The canal invert, bottom width and water surface were hydraulically established to maintain a specified power production head (elevation 74 msl). Borings indicate that the apparent ground water table ranges from elevation 65.0' msl to 75.0' msl over most of the intake canal length. A seismic coefficient of 0.15 (15% of the vertical load) is recommended for this Zone 3 area in EM 1110-2-1902 (See Plate D-4). The levee and excavated material deposited on either side of the canal were considered to cause surcharge loading. However, in most cases this weight did not affect the stability of the cut slopes because of the wide berm provided between the canal and levees for earthquake attrition. One minor stability consideration was access. In all cases, the berms and slopes are easily assessible to an all-terrain vehicle and to maintenance equipment.

10. Loading Conditions. The intake canal cuts were analyzed under two normal loading conditions, end-of-construction and steady seepage, and for earthquake loading. After extensive hydrologic study, it was determined that sudden drawdown would not be a realistic design condition since the canal water level would not fluctuate rapidly more than one or two feet over the life of the project. Seepage forces would be minimal for the steady seepage and partial pool conditions since ground water would be at the same level as the canal water surface. The end-of-construction case was analyzed for the normal loading condition only, since the probability of strong earthquake occurring during construction is low. The steady seepage case was investigated for normal and earthquake loading. Partial pool case was not investigated because it would be a duplication of the steady seepage analysis.

11. Results. The intake canal cut slopes were determined to be stable under normal loading at slope values of 1v on 3.5h near Lake Moultrie (Station 135+40 to Station 150+00) and 1v on 3h for the remainder of the intake canal. However, slopes of 1v on 8h and 1v on 6h would be necessary to withstand a severe earthquake (seismic coefficient = 0.15). Therefore, stability berms were added at elevation 78 msl to allow the design slopes to fail under severe earthquake without destroying the levees along the canal. The most critical loading condition for stable intake canal cut slopes was steady seepage under earthquake loading. This loading condition may not be exactly applicable since the proposed canal water surface would be one to five feet higher than the adjacent ground water level. However, the purpose of the steady seepage analyses was not to investigate effects of seepage forces on the slopes but to determine the long term slope stability under normal and earthquake loading. A summary of the factors of safety obtained in the analyses are listed in Table No. D2.

TABLE NO. D2

STABILITY ANALYSIS SUMMARY
INTAKE CANAL
COOPER RIVER REDIVERSION PROJECT

Location	Type of Analysis	Case	Type of Test	Slope	Safety Factor By Hand / Required F. S.	Remarks
				CANAL SLOPES		
140+00	Wedge	SS	S	1 on 3.5h	1.15 / 1.10	Normal Loading
140+00	Wedge	SS	S	1 on 3.5h	1.00 / 1.00	Earthquake @ ψ=0.05
140+00	Wedge	SS	S	1 on 8h	2.00 / 1.10	Normal Loading
140+00	Wedge	SS	S	1 on 8h	0.95 / 1.00	Earthquake @ ψ=0.15
140+00	Wedge	EOC	Q	1 on 3.5h	5.80 / 1.30	Normal Loading
194+30	Wedge	SS	S	1 on 3h	1.85 / 1.10	Normal Loading
194+30	Wedge	SS	S	1 on 3h	1.14 / 1.00	Earthquake @ ψ=0.15
194+30	Wedge	EOC	Q	1 on 3h	2.10 / 1.30	Normal Loading
246+00	Wedge	SS	S	1 on 3h	1.15 / 1.10	Normal Loading
246+00	Wedge	SS	S	1 on 3h	0.80 / 1.00	Earthquake @ ψ=0.15
246+00	Wedge	SS	S	1 on 6h	1.67 / 1.10	Normal Loading
246+00	Wedge	SS	S	1 on 6h	1.00 / 1.00	Earthquake @ ψ=0.15
246+00	Wedge	EOC	S	1 on 3h	4.18 / 1.30	Normal Loading
272+75	Wedge	SS	S	1 on 3h	1.62 / 1.10	Normal Loading
272+75	Wedge	SS	S	1 on 3h	1.10 / 1.00	Earthquake @ ψ=0.15
272+75	Wedge	EOC	Q	1 on 3h	4.65 / 1.30	Normal Loading
				LEVEE SLOPES		
335+00	Slip Circle	SS	S	1 on 3h	1.72 / 1.50	Normal Loading
335+00	Slip Circle	SS	S	1 on 3h	1.00 / 1.00	Earthquake @ ψ=0.15
335+00	Slip Circle	P.P.	$\frac{R+S}{2}$	1 on 4h	1.99 / 1.10	Normal Loading
335+00	Slip Circle	P.P.	$\frac{R+S}{2}$	1 on 4h	1.25 / 1.00	Earthquake @ ψ=0.15
335+00	Slip Circle	EOC	Q	1 on 3h	1.62 / 1.30	Normal Loading - Downstream
335+00	Slip Circle	EOC	Q	1 on 4h	1.82 / 1.30	Normal Loading - Upstream

12. Typical Sections. An investigation of stability of the levee embankments along the intake canal was concentrated on one section. This section is representative of the most severe loading conditions and the maximum expected levee height (28 feet). This section is labeled Station 335+00 (see Plates D-24 through D-27) and is representative of partially inundated levee embankments from 250 feet above of S.C. State Highway 64 (Station 329+00) to approximately 750 feet below the same road (Station 339+00). Levee embankments less than 10 feet high along the intake canal would be completely above the canal waters, would only be susceptible to wetting by surface runoff and are expected to cause no stability problem. Any portion of the levees with heights between 10 feet and 15 feet may become partially saturated at the base. However, a berm will be maintained in the embankment at elevation 78 msl that will allow only the embankment below the berm to be inundated. Consequently, no analyses were considered necessary for this condition. Lastly, embankments with heights greater than 15 feet would be partially inundated at the base and subjected to continuous seepage. Stability of embankments with heights greater than 15 feet are represented by the analysis of the above section.

13. General Soil Types and Investigations. Generally, select excavation will be used to construct the levees. This select material is expected to be directly placed in the levee from canal excavation, or if necessary stockpiled until needed. The select fill material is expected to be obtained from the top 10 feet of canal excavation and to consist of tan, medium to fine grained, clayey sand (SC). The select material is readily available in sufficient quantities to construct the levee and the ground water table is at a depth such that the select soil should require no drying before replacement.

14. Engineering Properties of Soil. Bag samples of the clayey sand in the upper 10 feet of soil strata were obtained from various locations along the alignment of the intake canal. These samples were combined to make two composite mixtures for testing by NED laboratories. The purpose of the testing program was to classify the soil and to provide ranges of physical properties of the material to be used as embankment fill. Testing included: (1) moisture contents, (2) Atterburg limits, (3) grain size analyses and (4) standard Proctor compaction tests. After the above parameters were reviewed, strength tests were performed on remolded composite samples of the proposed embankment material. The Q and R triaxial tests, and S direct shear tests were run at various moisture contents (minus 2 percent to plus 2 percent of optimum moisture content) in order to determine the range of shear strengths of the proposed embankment materials within the range of probable placement moisture contents. A summary of the remolded (disturbed) strength test results is shown in Table No. D1. Actual lab remolded strengths for the composite clayey sand material were used to select the

embankment design strengths for determining levee embankment stability. The lab strengths are plotted graphically on Plates D-5 through D-7. Strengths for the embankment foundation materials were selected from undisturbed test results of a soil obtained from nearby borings (CS-15 and 17).

15. Embankment Settlement. Consolidation tests were run on the remolded soil composites in order to calculate anticipated levee settlement. The levees will be a controlled fill, placed in compacted thin lifts. The surcharge on the levee will be contributed by its own weight and the weight of adjacent disposal area material. The calculations indicate that settlement of the dike is expected to range from 3" to 4" (maximum). Settlement computations are presented in Exhibit 1, Settlement Computations.

16. Embankment Design Criteria. Certain design assumptions were established for conducting stability analyses on the levee embankment slopes. The levees are planned to be constructed in controlled thin lifts, therefore, no tension crack effects were considered since surface drainage should be adequate to prevent ponding. The levee height was set at elevation 86.0' msl from Lake Moultrie to the Powerhouse by hydrologic studies. The water surface would saturate the embankment only in the reach between Station 290+00 and the Powerhouse (Station 362+62). In the reach through higher ground, the water table ranged from elevation 65.0' msl to 75.0' msl. However, between S.C. State Highway 64 and the Powerhouse, the water table remained quite close to original ground. It should be noted that original ground ranges from 54.0' msl to about 64.0' msl in this reach. Seepage of water from the canal through the levee was calculated in accordance with EM 1110-2-1901. This seepage line is shown on Plates 24 through 27. A seismic coefficient of 0.15 was used for all analyses involving an earthquake loading condition since the project is located in Zone 3 (See Plate D-4). The crest width of the levees was established at 20 feet to accommodate stability, equipment access and future maintenance.

17. Loading Conditions. The embankment slopes were analyzed for the end-of-construction, steady seepage (long-term) and partial pool cases. Stability analyses of steady seepage and partial pool cases were conducted for both normal and earthquake loading. The end-of-construction case was analyzed for normal loading only, since the possibility of a strong earthquake occurring during construction is remote. Hydrologic studies showed that rapid drawdown would not occur, therefore, the levee embankments were not analyzed for rapid drawdown loading.

18. Results. In the reach of maximum levee height (Station 339+00 to Station 362+00) between S. C. State Highway 64 and the proposed Powerhouse site, 1v on 4h slopes were required for stability on the canal side of the levees. The most critical loading condition for the canal-side levee slope was partial pool at elevation 70.3 msl under earthquake loading. The possibility of a severe earthquake

occurring during a drought condition (pool level lower than elevation 71 msl) during the life of the project is considered quite remote. However, slopes were established at 1v on 4h to remain stable should this combination of events occur. A 30-foot wide berm was added at elevation 78 msl for partial pool stability. A slope of 1v on 3h was determined to be stable for the land side of the levees near the Powerhouse, and for both inside and landside (outside) levee slopes from Lake Moultrie (Station 146+00) to Station 335+00. The steady seepage case governed the slope value of the landside levee slope between Station 330+00 and Station 340+00 Even though seepage would not occur through the levees along the remainder of the intake canal, the steady seepage slope value (1v on 3h) was selected for the landside slope. Factors of safety obtained in the analyses are summarized in Table D2.

TAILRACE CANAL - CUT SLOPES

19 Typical Sections. Stability analyses were conducted at three representatives sections along the tailrace canal. These three sections represent the following reaches of the canal: (1) Station 370+00 - immediately downstream of the Powerhouse, (2) Station 390+00 - downstream from the Powerhouse to the SCL Railroad and (3) Station 500+00 - the remainder of the canal with a levee on the river side of the canal and a high bluff on the land side. The cross-section at Station 370+00 was developed by Philadelphia District and will be presented in the Powerhouse Foundation Design Report. The section at Station 390+00 is merely a composite of sections at Stations 370+00 and 500+00 which better represents the geographical features at that location. The section at Station 500+00 (Plates D-28 through D-31) was selected for stability analysis as the most representative of subsurface conditions and loading conditions in the tailrace canal. Factors of safety resulting from these analyses would represent the cut slope stability of the entire tailrace canal except for the first 1500 feet of canal downstream from the Powerhouse. The design slopes developed by Philadelphia District were applied to the initial 1500 feet of canal downstream from the Powerhouse.

20 General Soil Types and Geologic Conditions. There are two predominant soil types within the excavation limits of the tailrace canal. The upper soil is a brown, lean clay (CL) ranging from 10 feet to 15 feet thick. A very hard, cemented sand (SM) underlies the clay and extends to rock. Rock in the tailrace canal is predominantly a weak shale except for layers of hard sandstone near the SCL railroad crossing (Station 419+00). Groundwater levels in the Santee River flood plain vary in depth from ground surface (artesian flow) to approximately 6 feet below ground surface. The groundwater level in the hillside is higher in elevation than in the flood plain. Groundwater levels as high as elevation 48 msl have been recorded in the hillside near the Powerhouse.

21. Engineering Properties of Soil. The soil samples obtained from drilling the site were tested by NED Laboratories. The purpose for testing was to classify the soils and to determine their shear strengths. Typical classification tests performed were moisture contents, grain size analyses, Atterburg limits, and standard compaction tests. Quick, rapid, and slow test results were obtained to determine the undisturbed shear strengths of the soils under several loading conditions. A summary of the laboratory test data is shown in Table D3. Lab strengths are graphically presented on Plates D-8 through D-10. Values were selected from these plots to represent strengths of the lean clay. The strengths of the cemented sand were selected from test results of borings (CS-14, CS-15 and CS-17) in the intake canal having somewhat similar soil characteristics. However, the intake canal soils were uncemented clayey sands; therefore, the design strengths used were conservatively low.

22. Determination of Top of Rock. 'Top of Rock' was determined for purposes of the stability analyses from the boring data using 'split-spoon' sampler refusal, 4"x5½" rock core and fishtail auger refusal The 'top of rock' elevation in split-spoon borings was assumed to be refusal to split-spoon penetration, i.e. blow count in excess of 100 blows per foot. Complex interbedding of shales and cemented sands throughout the tailrace canal made selection of 'top of rock' elevations highly judgmental.

23. Engineering Properties of Rock. In the tailrace canal area, rock is predominantly a dark gray shale interbedded with lightly cemented sands and a calcareously cemented sandstone. There appeared to be no definite layering within the strata and no potentially weak planes for failure in comparison to the overlying soils Thin layers of limestone and fine grained siltstone are present in some areas. All types of rock except the sandstone are weak and highly weathered and can be excavated by dragline without blasting.

24. Settlement Calculations. Calculations were made on material remolded at +2% OMC to anticipate settlement resulting from a saturated soil condition in the tailrace canal. The computations included a surcharge load contributed by a 23-foot high, densely compacted dike. The calculations indicated that total anticipated settlement of the canal banks due to levee surcharge should not exceed 7 inches. These computations are shown in Exhibit 1, Settlement Calculations.

25. Slope Design Criteria. No tension cracks were assumed. Original ground ranged from 17' to 23' msl. The invert and bottom width of the canal were set at 0.0' msl and 285 ft., respectively. Hydrologic investigations determined that the tailrace water surface would fluctuate between elevations 3 and 7 msl, twice daily. The drilling logs indicate that the groundwater table in the Santee River flood plain ranges from elevation 17.0 to 23.0 msl. The groundwater is found very close to ground surface at several locations. The 50 year design flood elevation

TABLE NO. D3

UNDISTURBED AND REMOLDED STRENGTHS

TAILRACE CANAL

1. UNDISTURBED

Hole No.	γ_m	γ_s	γ_b	Uncomf. Comp. K/SF	Q TEST C K/SF	Q TEST Ø	Q TEST tanØ	R TEST C K/SF	R TEST Ø	R TEST tanØ	S TEST C K/SF	S TEST Ø	S TEST tanØ
T-18	120	120	58	1.6	.8	0	0	.70	9°	.156	0	20.9°	.381
T-26	120	124	62	2.20	2.7	9°	.156	1.12	15.6°	.279	0	28.2°	.535
T-30	120	122	60	2.7	1.90	13.2°	.234	.44	14°	.25	0	23°	.425
T-32-1	121	122	60		.44	3.5°	.06	.52	11.7	.21	0	27.1°	.513
T-32-2	94	96	34	.82	.90	0	0	.38	12.7	.22	0	26.1°	.490
2. UNDISTURBED													
OMC CS#1	110	117	55	1.8	1.4 3.0	12.7 5.6	23 .10	.66	13.9	.25	0	28.3°	.54
-2% OMC CS#1	108	117	55	-	3.2	12.3	.22	-	-	-	-	-	-
+2% OMC CS#4	112	117	55	-	2.2	4.2	.07	.86	10.8	.19	-	-	-
OMC CS#2	107	115	53	1.4	2.0	7.1	.13	.34	14.8	.26	0	26.8	.50
-2% OMC CS#2	106	115	53	-	2.2	12.8	.23	-	-	-	-	-	-
+2% OMC CS#2	108	115	53	-	1.8	6.7	.12	.06	17.1	.31	-	-	-
OMC T-27B	125	132	70	1.6	.96	30.8	.60	1.2	14.5°	.26	0	36.7	.75
OMC -2% T-27B	123	133	71	-	2.0	25.5	.48	-	-	-	-	-	-
OMC +2% T-27B	127	132	70	-	1.0	28.2	.54	1.9	17°	.31	-	-	-

was determined to be 45 (msl) and a flood of this magnitude could last long enough to saturate the foundation ground and levee embankments. Criteria set forth by EM 1110-2-1902 suggests a seismic coefficient of 0.15 (See Plate D-4) for earthquake loading. The levees and disposed excavated material were considered to cause surcharge loading. However, this surcharge weight did not affect the stability of the cut slopes because of the large berm provided at elevation 26 msl running the entire canal length on both sides. Berms and slopes were established so as to be readily accessible to all-terrain vehicles and maintenance equipment.

26. Loading Conditions. The cut slopes were analyzed under several normal loading conditions: end-of-construction, steady seepage, and rapid drawdown and for earthquake loading. Due to the frequent water level fluctuation (twice daily) of about 15 feet, partial pool was not considered a realistic case and was not analyzed. Seepage forces would be minimal for the steady seepage condition since the groundwater level in the flood plain is within the range of daily tailwater fluctuations. The end-of-construction and steady seepage cases were analyzed for normal loading only, while the rapid drawdown case was analyzed for normal loading and earthquake loading. It is most likely that the tailrace canal would be in some stage of drawdown during a seismic event.

27. Results. Rapid drawdown was determined to be the critical loading case and, therefore, was analyzed for both the normal and the earthquake loading conditions. Under all normal loading conditions, a cut slope of 1v on 3h was sufficiently stable for the entire tailrace canal. However, it would be necessary to cut the slopes back to 1v on 6h in order to withstand a severe earthquake (seismic coefficient = 0.15). The iv on 3h cut slopes would be stable during low magnitude earthquake events. In order to avoid costly excavation, stability berms were added at elevation 26 msl of sufficient width so that neither earthquake failure nor severe tailwater erosion would destroy the levees. The primary purpose of the stability berms is to provide for attrition of the cut slopes due to tailwater velocity scour. The tailrace canal cut slopes were analyzed for steady seepage loading even though this loading condition may not apply since the groundwater level in the flood plain is within the range of daily tailwater level fluctuations. A summary of the factors of safety obtained in the analyses are listed in Table No. D4 along with required factors of safety. The critical stability case was rapid drawdown under earthquake loading.

TABLE NO. D4

STABILITY ANALYSIS SUMMARY
TAILRACE CANAL
COOPER RIVER REDIVERSION PROJECT

Location	Type of Analysis	Case	Type of Test	Slope	Safety Factor By Hand / Required F. S.	Remarks
			CANAL SLOPES			
370+00	- - -	- - -	- - -	- - -	- - -	Stability Analysis Conducted by Philadelphia District
390+00	- - -	- - -	- - -	- - -	- - -	Composite of NAP Sta. 370+00 and SAC Sta 500+00
500+00	Wedge	EOC	Q	1 on 3h	$\frac{1.35}{1.30}$	Canal - Normal Loading
500+00	Wedge	SS	S	1 on 3h	$\frac{1.03}{1.00}$	Canal - Normal Loading
500+00	Wedge	SDD	R	1 on 3h	$\frac{1.35}{1.20}$	Canal - Normal Loading
500+00	Wedge	SDD	R	1 on 3h	$\frac{1.00}{1.00}$	Canal - Earthquake Loading $\psi=0.06$
500+00	Wedge	SDD	R	1 on 6h	$\frac{1.10}{1.00}$	Canal - Earthquake Loading $\psi=0.15$
			LEVEE SLOPES			
500+00	Slip Circle	EOC	Q	1 on 3h	$\frac{1.93}{1.30}$	Downstream Levee - Normal Loading
500+00	Slip Circle	SS	S	1 on 3h	$\frac{1.41}{1.10}$	Downstream Levee - 50-Year Flood, Normal Loading
500+00	Slip Circle	SS	S	1 on 3h	$\frac{1.06}{1.00}$	Downstream Levee - Earthquake Loading $\psi=0.15$
500+00	Slip Circle	PP	S	1 on 3h	$\frac{1.24}{1.10}$	Upstream Levee - Flood Pool 330.4 Elevation, Normal Loading
500+00	Slip Circle	SDD	$\frac{R+S}{2}$	1 on 4h	$\frac{2.33}{1.10}$	Upstream Levee - 50-Year Flood, 30' Berm-Normal Loading Before Drawdown
500+00	Slip Circle	SDD	$\frac{R+S}{2}$	1 on 4h	$\frac{1.10}{1.10}$	Upstream Levee - 59-Year Flood, 30' Berm-Normal Loading After Drawdown

TAILRACE CANAL - LEVEE EMBANKMENTS

28. Typical Sections. Stability analyses of the levee embankments along the tailrace canal were conducted for one typical cross-section. This section represents a maximum fill height of 23 feet and will be referred to as Station 500+00 (See Plates D-32 thru D-36 for stability computations). The embankment will be above the groundwater table normally but may become saturated by Santee River flooding.

29. General Soil Types and Investigations. Generally, the more suitable material from canal excavation will be used to build the levee embankments. The predominant embankment fill material is expected to be fine grained, brown lean clay (CL) obtained from the top 10 to 15 feet of tailrace canal excavation. Based on low plastic limits, moderately high moisture contents and a large percentage of clay fines, it is expected that the clay fill material will need to be dried before placement. Stronger, more granular fill material will be available from the bottom of the canal excavation.

30. Engineering Properties of Soil. Bag samples were obtained from the upper 10 to 15 feet of soil at several locations along the tailrace canal alignment. These samples were mixed together to form composite samples. The composite samples were tested for strength and classification by New England Division Laboratories. Testing included: (1) moisture contents, (2) Atterburg limits, (3) grain size analyses and (4) standard Proctor compaction tests. Several quick, rapid and slow strength tests were run at various moisture contents on remolded specimens from the composite soil samples. The purposes of this testing were to determine the physical properties of the embankment fill material for stability analyses and to provide ranges of quality for control of construction of the embankment fill. Foundation strengths were selected from test results (See para. 21) of undisturbed boring samples. A summary of the remolded test results is presented in Table D3. Lab strengths are plotted graphically on Plates D-11, D-12 and D-13. Design strengths were selected as shown on these plots.

31. Embankment Settlement. Consolidation tests were run on the three composite soil samples in order to determine the most compressible mixture. The results from the most compressible mixture were then used to calculate anticipated levee settlement. This embankment will be a controlled fill, placed in thin compacted lifts. The only surcharge on the levee will be its own weight. The calculations indicate (See Exhibit 1) that settlement should not exceed 8" (maximum).

32. Embankment Design Criteria. Certain design criteria were established for the embankment slopes of the tailrace levee embankments. No tension

crack effects were considered since the levees are planned to be constructed in controlled thin lifts and, surface runoff is not expected to pond on the embankment. The levee crest on the right descending side of the canal was set at elevation 35 msl (10 year Santee River flood) or original ground (whichever is higher). The riverside (left descending) levee crest was established at elevation 45.0 msl from the Powerhouse to the SCL Railroad (station 419+00). Then the crest would slope uniformly from elevation 45.0 to 35.0 msl at the end of the levee (station 595+00). It should be noted that a 50-year Santee River flood would overtop elevations 45 msl. Under normal operating conditions the levees would not be subjected to any water loading, only surface runoff. The riverside levee is designed to contain seasonal Santee River flooding and even major floods approaching a 50-year frequency. After an extended period of flooding, the embankments were assumed to become completely saturated by water on both sides of the levees. A seismic coefficient of 0.15 (See Plate D-4) was applied to the project area. A levee crest width of 20 feet was established to accomodate all-terrain vehicles and maintenance equipment.

33. Loading Conditions. The levee embankment slopes were analyzed for several normal loading conditions: end-of-construction, steady seepage, partial pool and rapid drawdown, and for earthquake loading. Stability analyses were conducted on the above cases at various tailwater and Santee River flood elevations. The rapid drawdown and partial pool cases were not analyzed for earthquake loading because the probability of a severe earthquake occuring immediately after a 50-year Santee River flood is low. Earthquake loading was not analyzed for the end of construction case because the probability of an earthquake occuring at the end of construction is low.

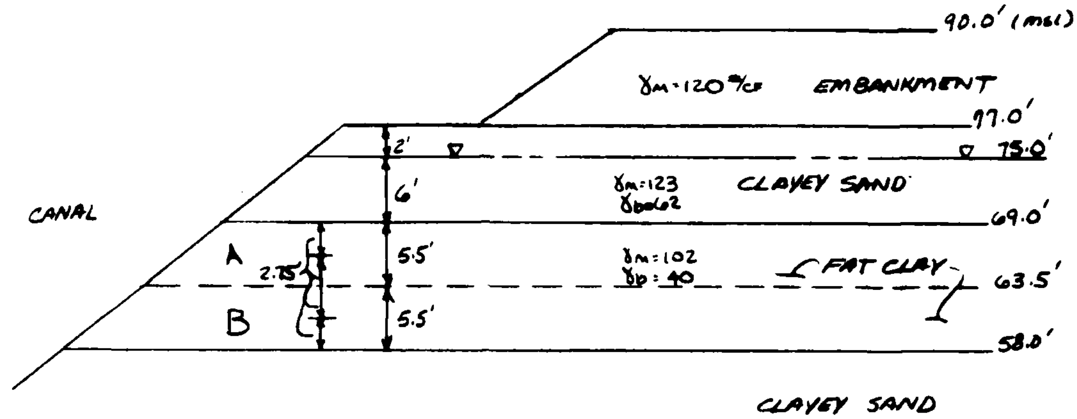
34. Results. A slope value of lv on 4h was found stable against potential embankment failure for both slopes of the riverside levee. The landside levee slope of lv on 3h was stable for all loading conditions. Factors of safety obtained in the analyses are summarized in Table No. D4. **Berms were added at elevation 26 for stability and accessibility** to all-terrain vehicles and maintenance equipment. The critical stability case was rapid drawdown from the 50-year flood level under normal loading.

EXHIBIT NO. 1

SETTLEMENT CALCULATIONS

FOUNDATION SETTLEMENT

Section taken just
Inside of existing Lake
Moultrie dike. (station 140+00)



① AT CENTER OF LAYERS $\quad \sigma_{0A}' = 123(2) + 62(6) + 2.75(40) \approx 730\#/sf$

$\sigma_{0B}' = 123(2) + 62(6) + 5.5(40) + 2.75(40) \approx 950\#/sf$

② Use b = 100 ft for width of a continuous foundation load (surcharge)

$\Delta\sigma$ @ surface = 13'(120) + 25% (impact) ≈ 1950 #/sf

③ Leonards pp. 559 $\quad$ Influence factors ⇒ for A → $\frac{13.5}{100} = 0.14$ ∴ Use 0.95q

B → $\frac{19}{100} = 0.19$ ∴ Use 0.95q

④ $\Delta\sigma_A = \Delta\sigma_B = 0.95(q) = 0.95(1950) = 1850$

⑤ $\Delta H_T = \sum H_i \frac{C_c}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$ $\quad H_i = 5.5$; $C_c = \Delta e/\log$ cycle $= 0.106$, $e_0 = 0.899$, σ_0' = see ①

⑥ $\Delta H_A = 5.5 \frac{0.106}{1+0.899} \log \frac{730+1850}{730} = 0.17$

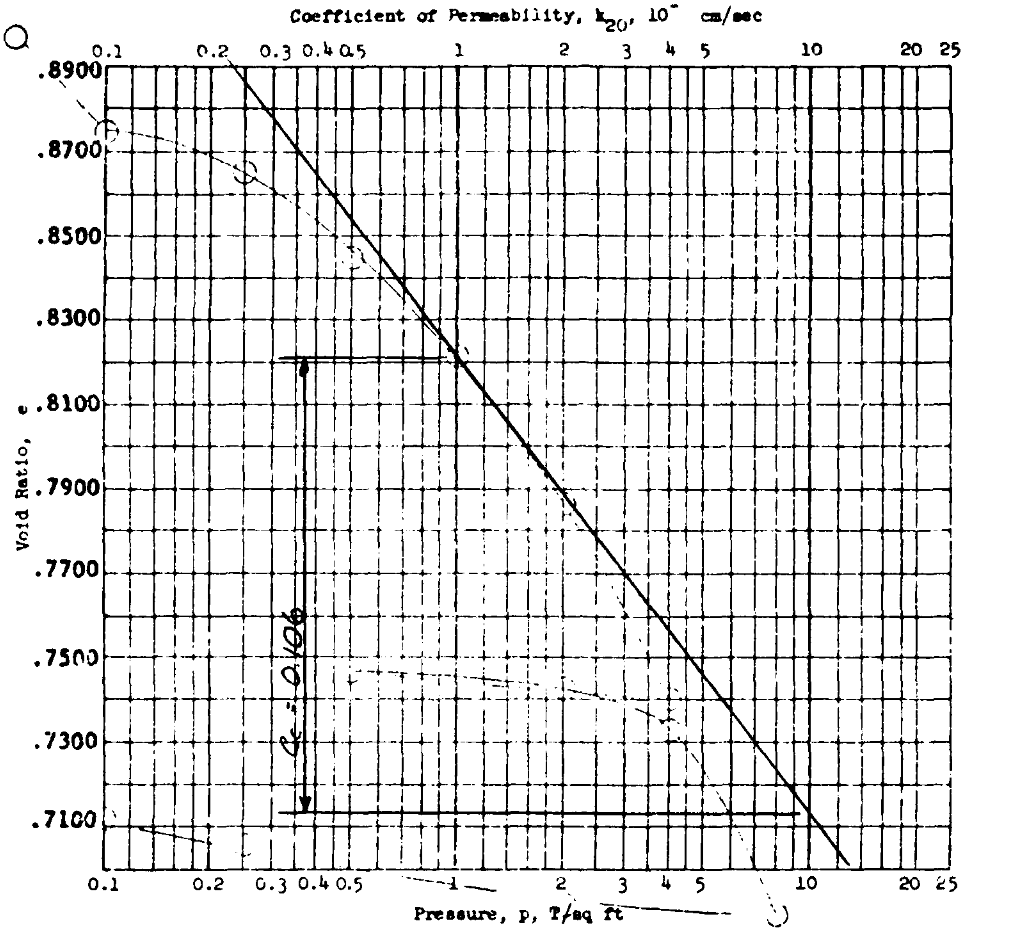
$\Delta H_B = 5.5 \frac{0.106}{1+0.899} \log \frac{950+1850}{950} = 0.14$

$\Delta H_T = 0.31$ ft or 4" Settlement

⑦ Total settlement of CH (fat Clay) as a result of spoil disposal (surcharge) on banks of canal near Lake Moultrie is expected to be 4".

WORK ORDER NO. SANCA-75-32
REQ. NO. 9283

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY,
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30061

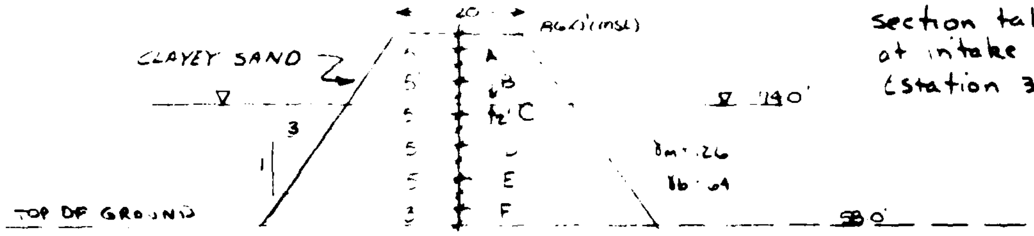


Type of Specimen Undisturbed		Before Test		After Test	
Diam 3.98 in.	Ht 1.26 in.	Water Content, w_o	31.9 %	w_f	26.0 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.899	e_f	0.712
Precousol. Pressure, p_c T/sq ft		Saturation, S_o	97.3 %	S_f	100.0 %
Compression Index, C_c		Dry Density, γ_d	90.0 lb/ft³		
Classification Gray sandy fat clay (CH)		k_{20} at e_o = × 10⁻ cm/sec			
LL 67	G_s 2.74	Project Charleston District COOPER RIVER REDIVERSION			
PL 19	D_{10} < .001 mm	Lab. No. 235/304			
Remarks 1. See lab classification data on ENG Form 2087 reported previously.		Area Intake Canal			
		Boring No. IT-27		Sample No. 2	
2. Specimen flooded throughout test.		Depth, El 14.5' - 17.0'		Date 11 Nov 1975	
		CONSOLIDATION TEST REPORT			

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE

BY GSM DATE 3/31/76 SUBJECT Intake Canal
CHKD. BY DATE Cooper River Rediversion Project
Settlement Calculations
SHEET NO. 3 OF 6
JOB NO. 8818430412000

Embankment Settlement



section taken at intake Canal (station 335+00)

① σ_0 AT CENTER OF LAYERS

σ_{0A} = 2.5(.126) = 315 #/sf
σ_{0B} = 7.5(.126) = 945
σ_{0C} = 12(.126) + 0.5(.064) = 1545
σ_{0D} = 12(.126) + 5.5(.064) = 865
σ_{0E} = 12(.126) + 10.5(.064) = 2185
σ_{0F} = 12(.126) + 14.5(.064) = 2440

② Use B = 20' for width of a continuous foundation load (surcharge)
$\Delta\sigma_{surface}$ = ? + impact = 750 psf (assumed)

③ Leonards p 559 ⇒ Influence factor

A = $\frac{2.5}{20}$ = 0.125 → Use 0.95q $\Delta\sigma_A'$ = 0.95(750) = 713 #/sf
B = $\frac{7.5}{20}$ = 0.375 → Use 0.9q $\Delta\sigma_B$ = 0.9(750) = 675
C = $\frac{12.5}{20}$ = 0.625 → Use 0.7q $\Delta\sigma_C$ = 0.7(750) = 525
D = $\frac{17.5}{20}$ = 0.875 → Use 0.6q $\Delta\sigma_D$ = 0.6(750) = 450
E = $\frac{22.5}{20}$ = 1.125 → Use 0.5q $\Delta\sigma_E$ = 0.5(750) = 375
F = $\frac{26.5}{20}$ = 1.325 → Use 0.45q $\Delta\sigma_F$ = 0.45(750) = 338

④ $\Delta H_T = \Sigma H_i \frac{C_c}{1+e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$ H_i = 5', e_0 = 0.493, C_c = Δe/log cycle = 0.053 σ_0 = see ①

⑤ $\Delta H_A = 5 \frac{0.053}{1+0.493} \log \frac{315+713}{315} = 0.09'$

$\Delta H_B = 5 \frac{0.053}{1+0.493} \log \frac{945+675}{945} = 0.04'$

$\Delta H_C = 5 \frac{0.053}{1+0.493} \log \frac{1545+525}{1545} = 0.02$

$\Delta H_D = 5 \frac{0.053}{1+0.493} \log \frac{865+450}{865} = 0.02$

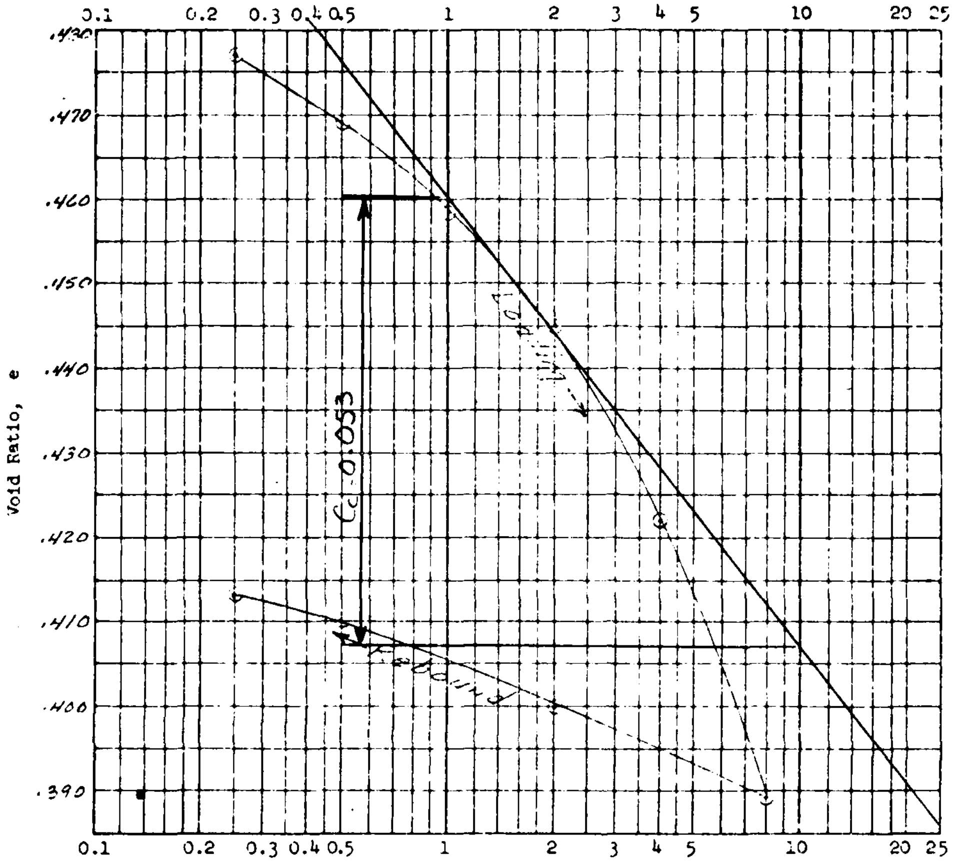
$\Delta H_E = 5 \frac{0.053}{1+0.493} \log \frac{2185+375}{2185} = 0.01'$

$\Delta H_F = 3 \frac{0.053}{1+0.493} \log \frac{2440+338}{2440} = 0.01$

ΔH_T = 0.20' or 3" Settlement

⑥ Total Settlement of the proposed clayey sand dike is expected to be 3".

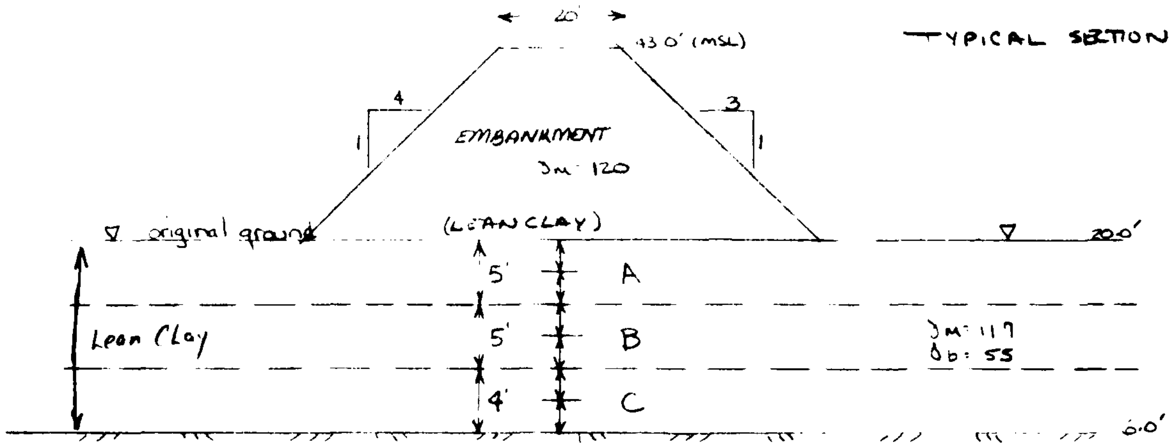
Coefficient of Permeability, k_{20}, 10^{-} cm/sec



Pressure, p, T/sq ft

Type of Specimen *Remolded*[1]		Before Test		After Test	
Diam *4.44* in.	Ht *1.0* in.	Water Content, w_o	*15.7* %	w_f	*17.1* %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	*0.473*	e_f	*0.413*
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	*83.9* %	S_f	*100* %
Compression Index, C_c *0.11*		Dry Density, γ_d	*109.9* lb/ft^3		
Classification *[illegible] Fine SAND(SC)*		k_{20} at e_o = × 10^{-} cm/sec			
LL *28*	G_s *2.63*	Project *Cooper River Rediversion*			
PL *16*	D_{10}	*St. Stephen, S. Carolina*			
Remarks *(1)- Samples remolded to approx. moisture content of 15.7% (O.M.C. +2.0) and dry density of [illegible] (95% max. density)*		Area *INTAKE CHANNEL*			
(2)- Combined following samples		Boring No. *IT-7, 13, 21, 27*		Sample No. *Composite "B"*[2]	
IT-7; S-3 IT-13; S-2 IT-21; S-3 IT-27 S-2		Depth *3.0 - 15.0'*		Date *December 1975*	
		CONSOLIDATION TEST REPORT			

FOUNDATION SETTLEMENT



① AT CENTER OF LAYERS $\sigma'_{oA} = 55(2.5) = 138$ #/SF
$\sigma'_{oB} = 55(7.5) = 413$ #/SF
$\sigma'_{oC} = 55(12) = 660$ #/SF

② Use b=100' for width of a continuous foundation load (surcharge)
$\Delta\sigma_{surface} = 23(120) + 25\%$ impact = 3450 #/SF

③ Leonards pp 559 Influence factor A → $\frac{2.5}{100} = 0.03$ Use q ∴ $\Delta\sigma_A = 3450$ #/SF
B → $\frac{7.5}{100} = 0.08$ Use $0.95q$ ∴ $\Delta\sigma_B = 3450(.95) = 3280$
C → $\frac{12}{100} = 0.12$ Use $0.9q$ ∴ $\Delta\sigma_C = 3450(.9) = 3100$

④ $\Delta H_T = \sum H_i \frac{C_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}$ H_i = increment height ; C_c = Δe/log cycle ; e_o = 0.511 ; σ'_o see ①

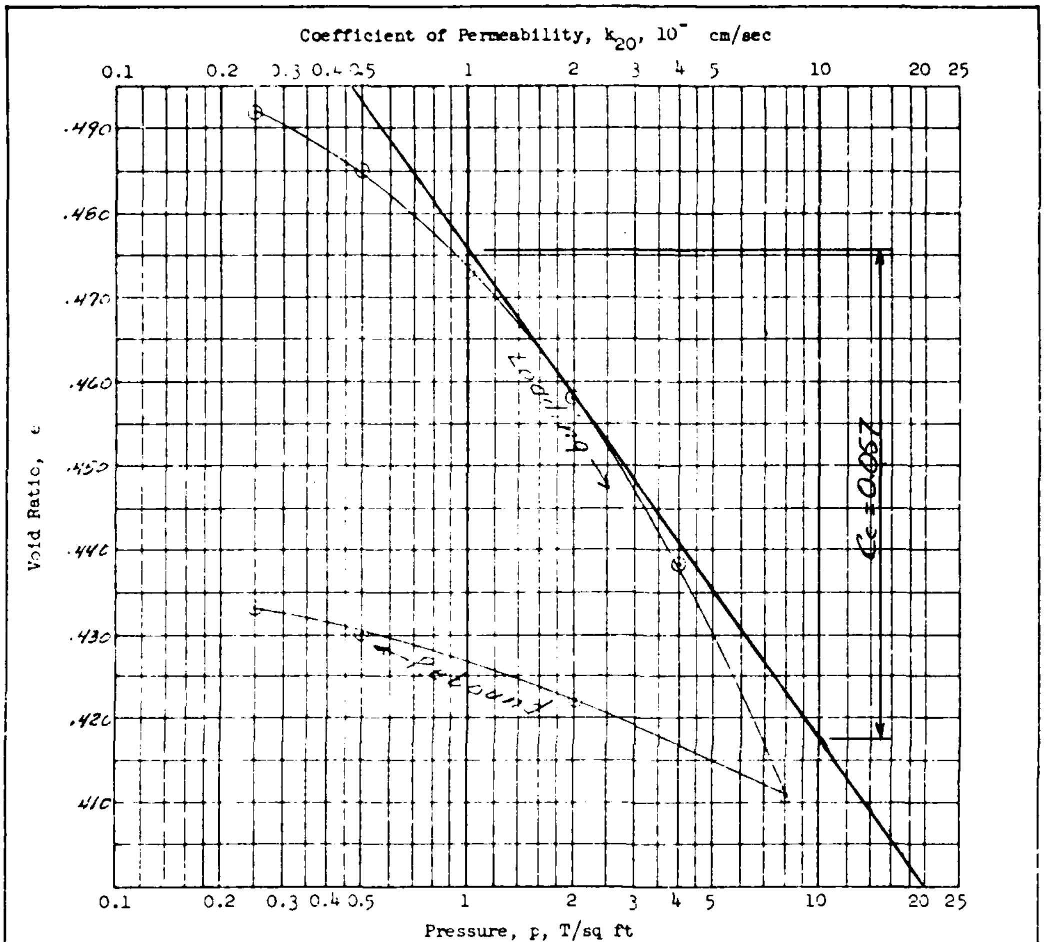
⑤ $\Delta H_A = 5 \cdot \frac{0.057}{1+0.511} \log \frac{138+3450}{138} = 0.27$

$\Delta H_B = 5 \cdot \frac{0.057}{1+0.511} \log \frac{413+3280}{413} = 0.18$

$\Delta H_C = 4 \cdot \frac{0.057}{1+0.511} \log \frac{660+3100}{660} = 0.11$

$\Delta H_T = 0.56$ or 7" Settlement

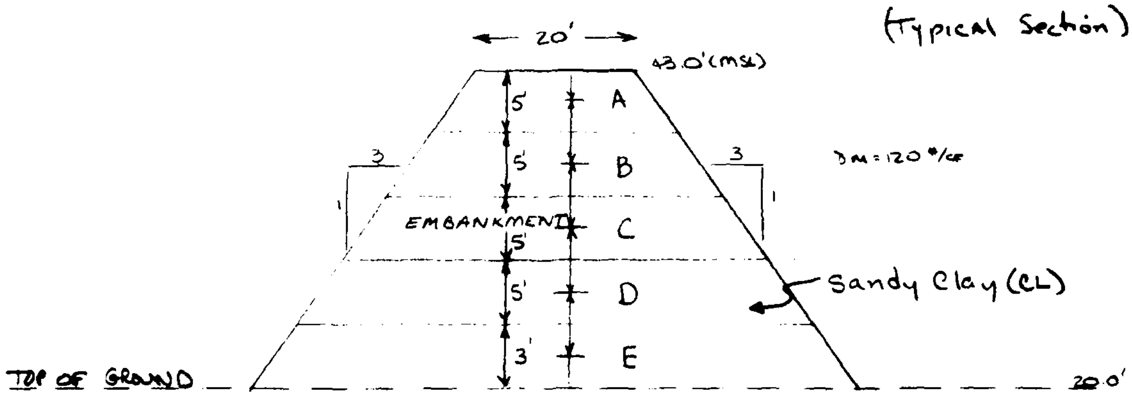
⑥ Total Settlement of Lean clay foundation soils) as a result of spoil & dike surcharge on the canal banks is expected to be 7".



Type of Specimen *Remolded*		Before Test		After Test	
Diam *4.45* in.	Ht *1.0* in.	Water Content, w_o	*14.0* %	w_f	*14.6* %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	*0.511*	e_f	*0.433*
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	*73.7* %	S_f	*80.3* %
Compression Index, C_c *0.08*		Dry Density, γ_d	*111.0* lb/ft^3		
Classification *clayey-silty fine SAND (SC-SM)*		k_{20} at e_o = × 10^{-} cm/sec			
LL *23*	G_s *2.69*	Project *Cooper River Rediversion*			
PL *17*	D_{10}	*St. Stephen, S. Carolina*			
Remarks *Samples remolded @ approx.*		Area *TAILRACE CANAL*			
moisture content of 14.2%		Boring No. *T-27*		Sample No. *B-1*	
(O.M.C. + 2%) and dry density		Depth *0.0' - 9.0'*		Date *Dec. 1975*	
of 111.2 Pcf. (95% max. density)		**CONSOLIDATION TEST REPORT**			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

Embankment Settlement



① AT CENTER OF LAYERS

$\sigma'_{oA} = 2.5(120) = 300$ #/SF
$\sigma'_{oB} = 7.5(120) = 900$
$\sigma'_{oC} = 12.5(120) = 1500$
$\sigma'_{oD} = 17.5(120) = 2100$
$\sigma'_{oE} = 21.5(120) = 2580$

② Use $b = 20'$ for the width of a continuous foundation load (surcharge)
$\Delta\sigma_{SURFACE} = 0 + \text{impact} \cdot 750$ #/SF

③ Leonards pp 559 @ $b = 20'$ Influence factor

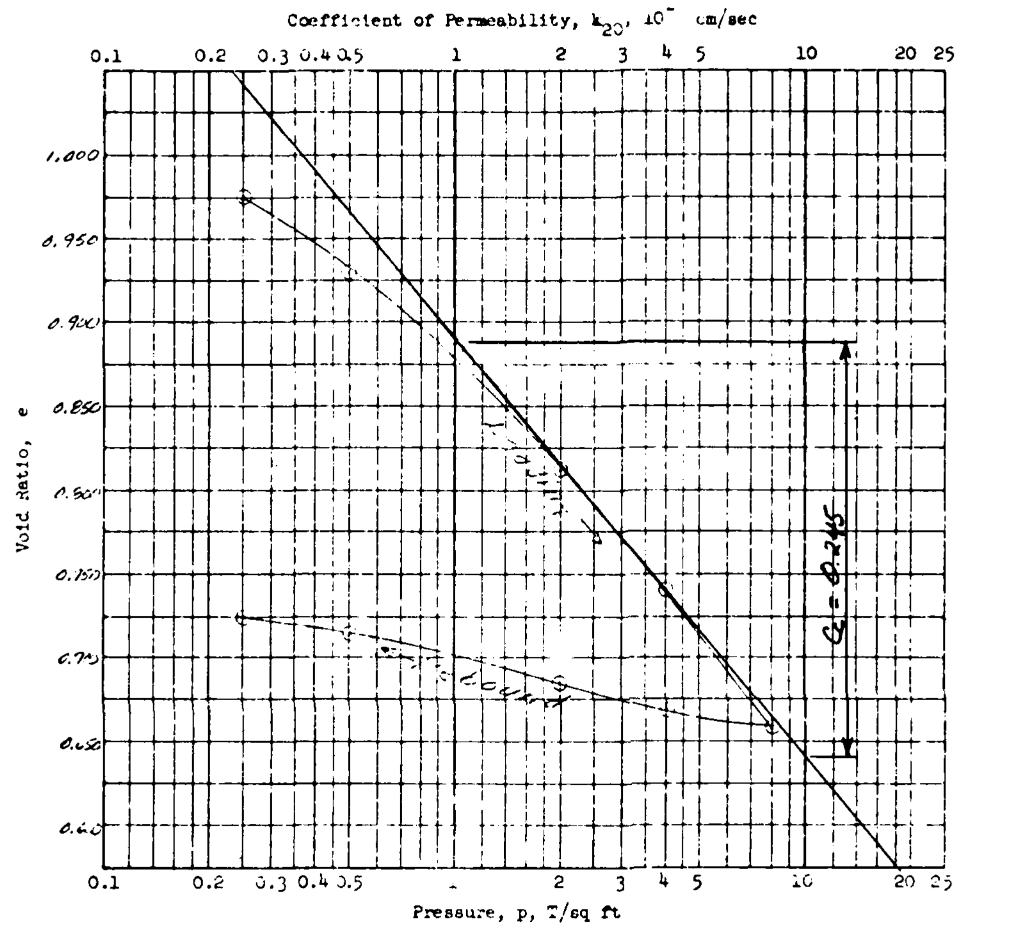
A → $\frac{2.5}{20} = 0.13$ Use 0.9q ∴ $\Delta\sigma_A = 750(0.9) = 675$ #/SF
B → $\frac{7.5}{20} = 0.38$ Use 0.85q ∴ $\Delta\sigma_B = 750(0.85) = 638$
C → $\frac{12.5}{20} = 0.63$ Use 0.7q ∴ $\Delta\sigma_C = 750(0.7) = 525$
D → $\frac{17.5}{20} = 0.88$ Use 0.6q ∴ $\Delta\sigma_D = 750(0.6) = 450$
E → $\frac{21.5}{20} = 1.08$ Use 0.5q ∴ $\Delta\sigma_E = 750(0.5) = 375$

④ $\Delta H_T = \sum H_i \frac{C_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}$ H_i: increment height; C_c: Δe/log cycle $= 0.245$; $e_o = 0.996$

⑤
$$\Delta H_A = 5 \cdot \frac{0.245}{1+0.996} \log \frac{300+675}{300} = 0.31$$
$$\Delta H_B = 5 \cdot \frac{0.245}{1+0.996} \log \frac{900+638}{900} = 0.14$$
$$\Delta H_C = 5 \cdot \frac{0.245}{1+0.996} \log \frac{1500+525}{1500} = 0.08$$
$$\Delta H_D = 5 \cdot \frac{0.245}{1+0.996} \log \frac{2100+450}{2100} = 0.05$$
$$\Delta H_E = 3 \cdot \frac{0.245}{1+0.996} \log \frac{2580+375}{2580} = 0.02$$

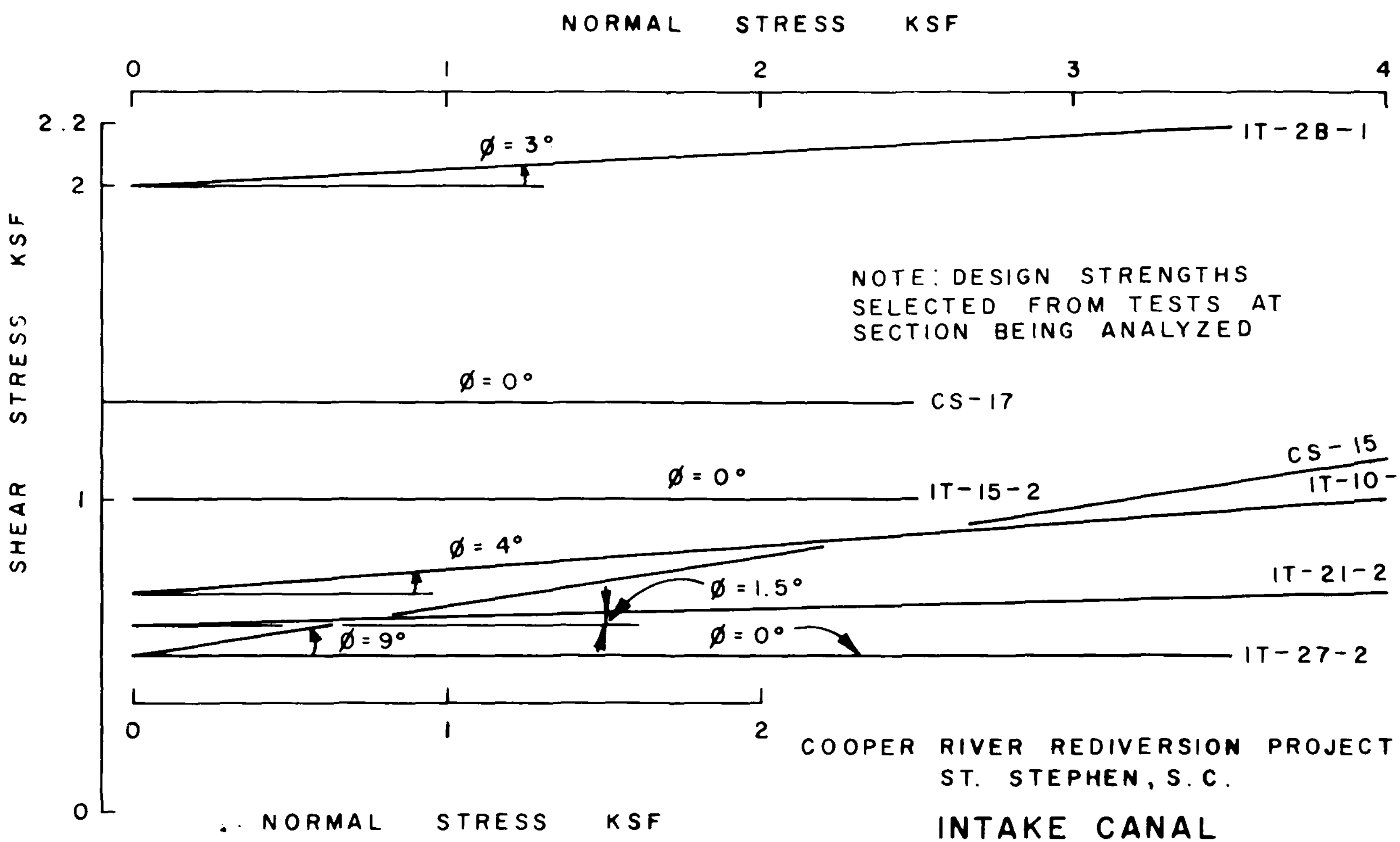
$\Delta H_T = .60$ or 7½" settlement

⑥ Total settlement of dike is expected to be 7½" ∴ overbuild by 1' to compensate for this settlement + ground settlement



Type of Specimen *Remolded*	Before Test		After Test	
Diam *4.44* in. Ht *1.0* in.	Water Content, w_o	*29.8* %	w_f	*30.2* %
Overburden Pressure, p_o T/sq ft	Void Ratio, e_o	*0.996*	e_f	*0.725*
Preconsol. Pressure, p_c T/sq ft	Saturation, S_o	*80.6* %	S_f	*100* %
Compression Index, C_c *0.24*	Dry Density, γ_d	*84.1* lb/ft^3		
Classification *Fine sandy Clay (CH)*	k_{20} at e_o = × 10^{-} cm/sec			
LL *57* G_s *2.69*	Project *Cooper River Rediversion*			
PL *27* D_{10}	*St Stephen, S. Carolina*			
Remarks *Sample molded @ approx. moisture content of 29% (OMC+2%) and dry density of 84 pcf (95% maximum density)*	Area *TAILRACE CANAL*			
	Boring No. *T-11 & T-17 combined*		Sample No. *CS #2*	
	Depth El *0.0 - 6.0'*		Date *January 1976*	
	CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE

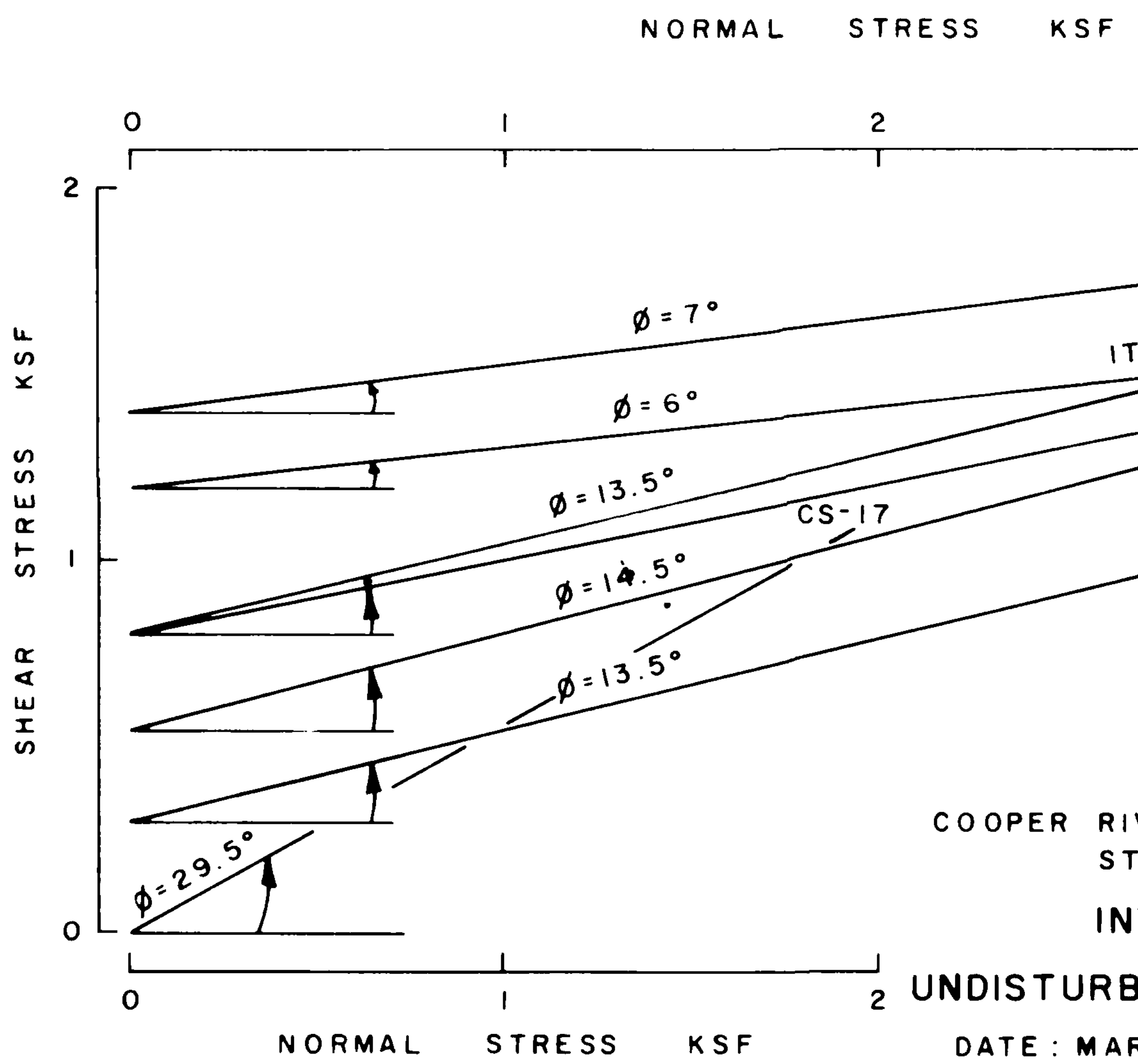


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S. C.

INTAKE CANAL

UNDISTURBED 'Q' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-1



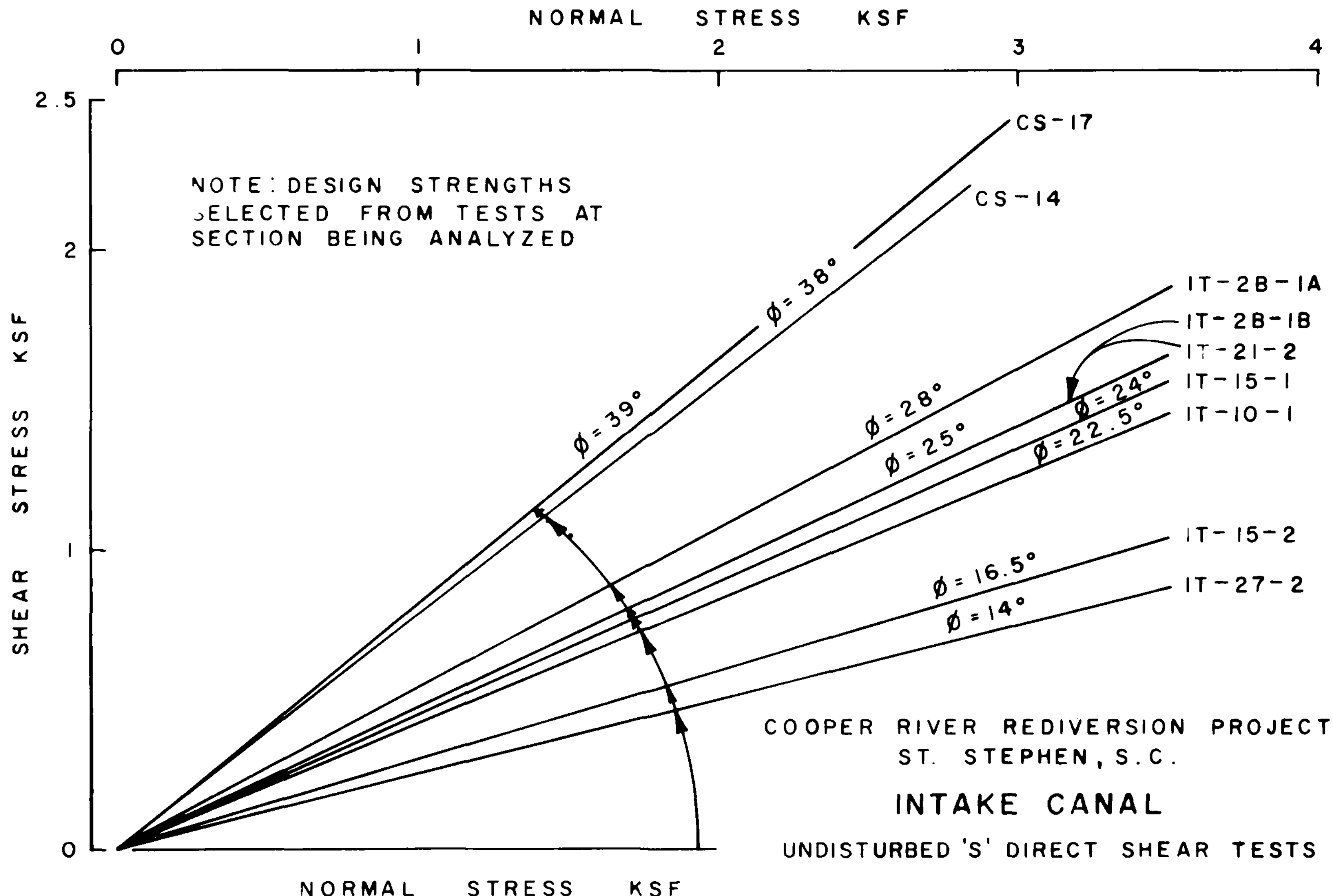
NOTE: DESIGN STRENGTHS SELECTED FROM TESTS AT SECTION BEING ANALYZED

COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

INTAKE CANAL

UNDISTURBED 'R' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-2

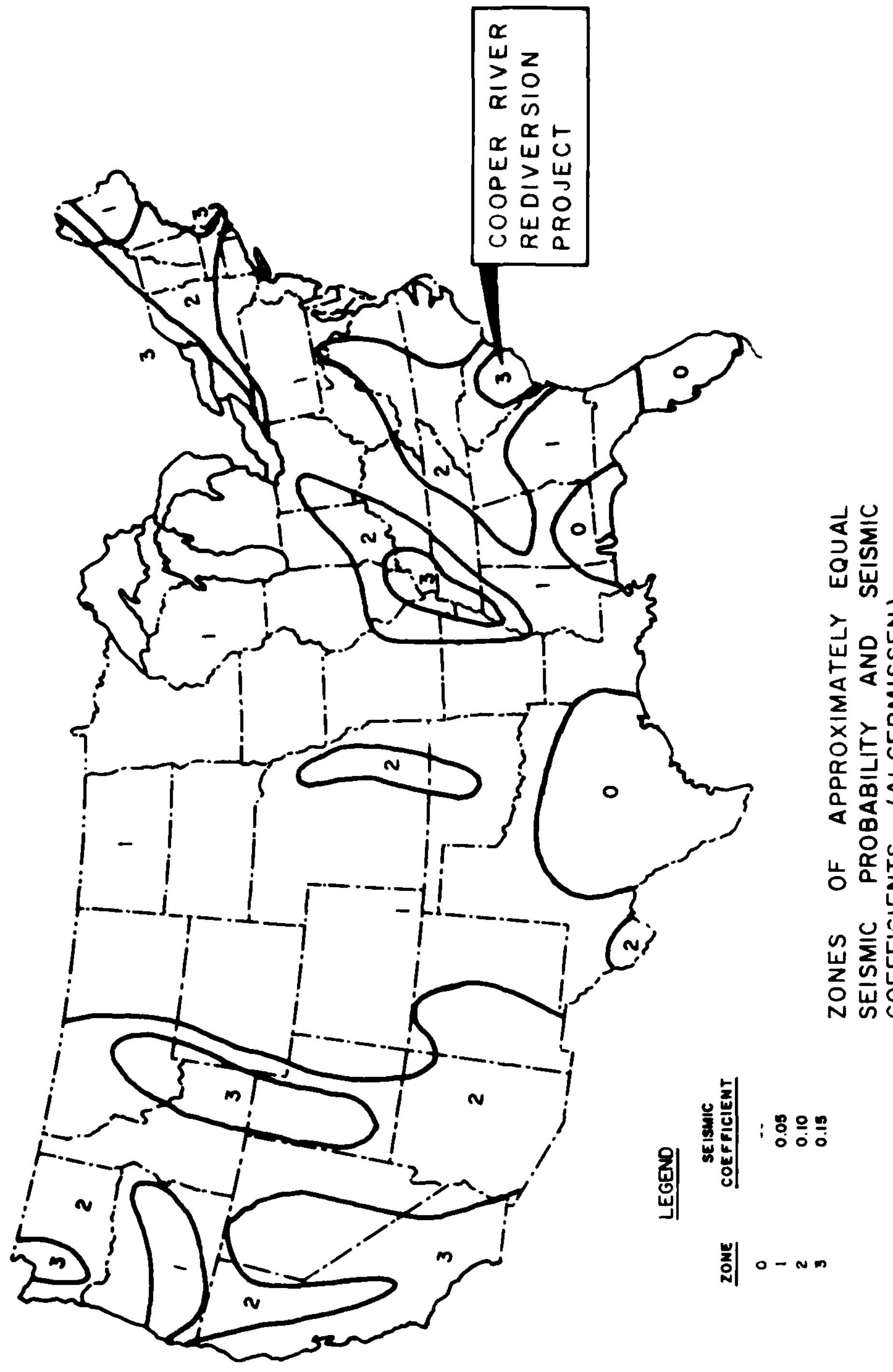


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

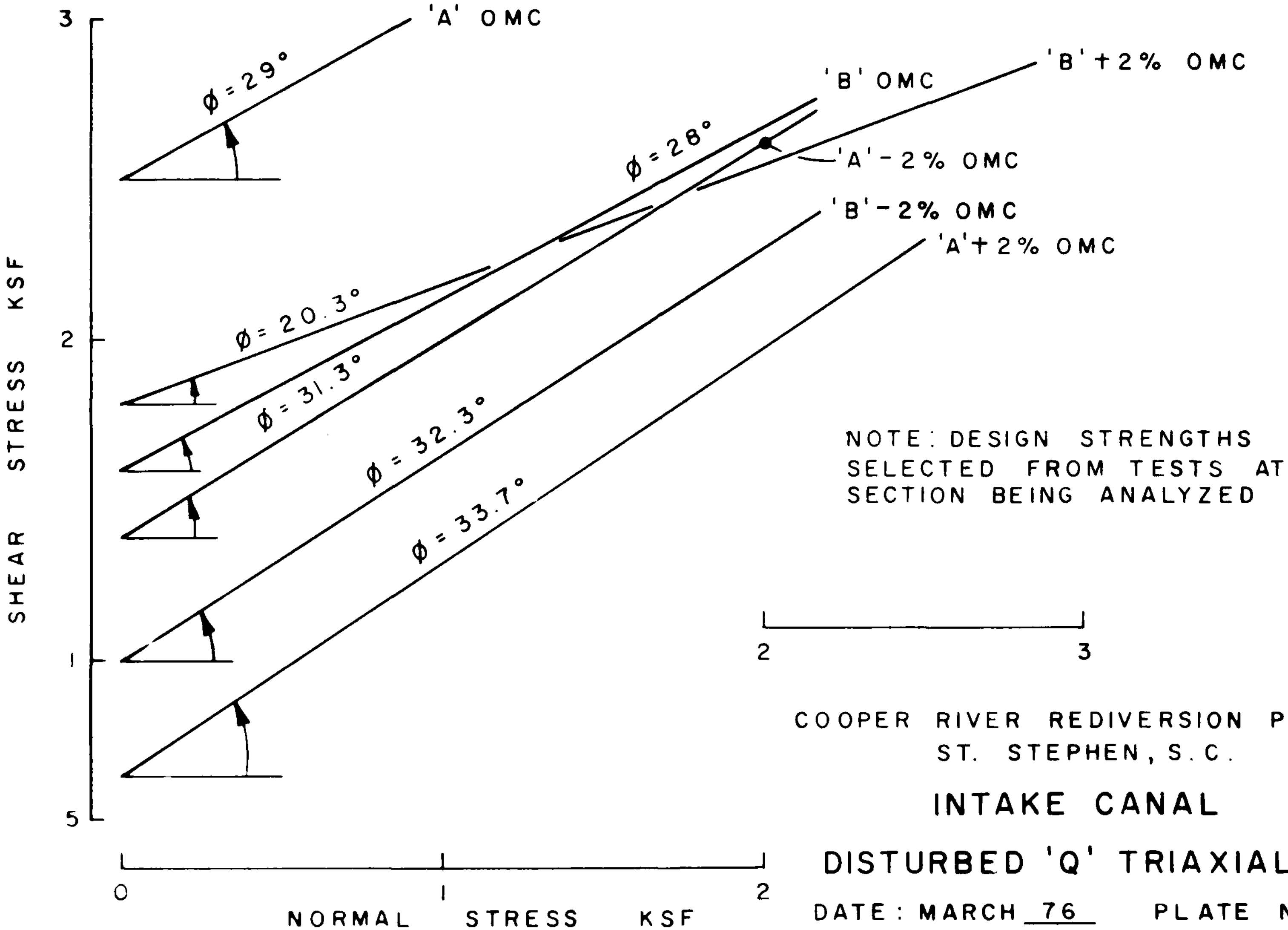
INTAKE CANAL

UNDISTURBED 'S' DIRECT SHEAR TESTS

DATE: MARCH 76 PLATE NO. D-3



SEISMIC RISK MAP
(FROM FIGURE 6 IN EM
1110-2-1902, APRIL 1970)



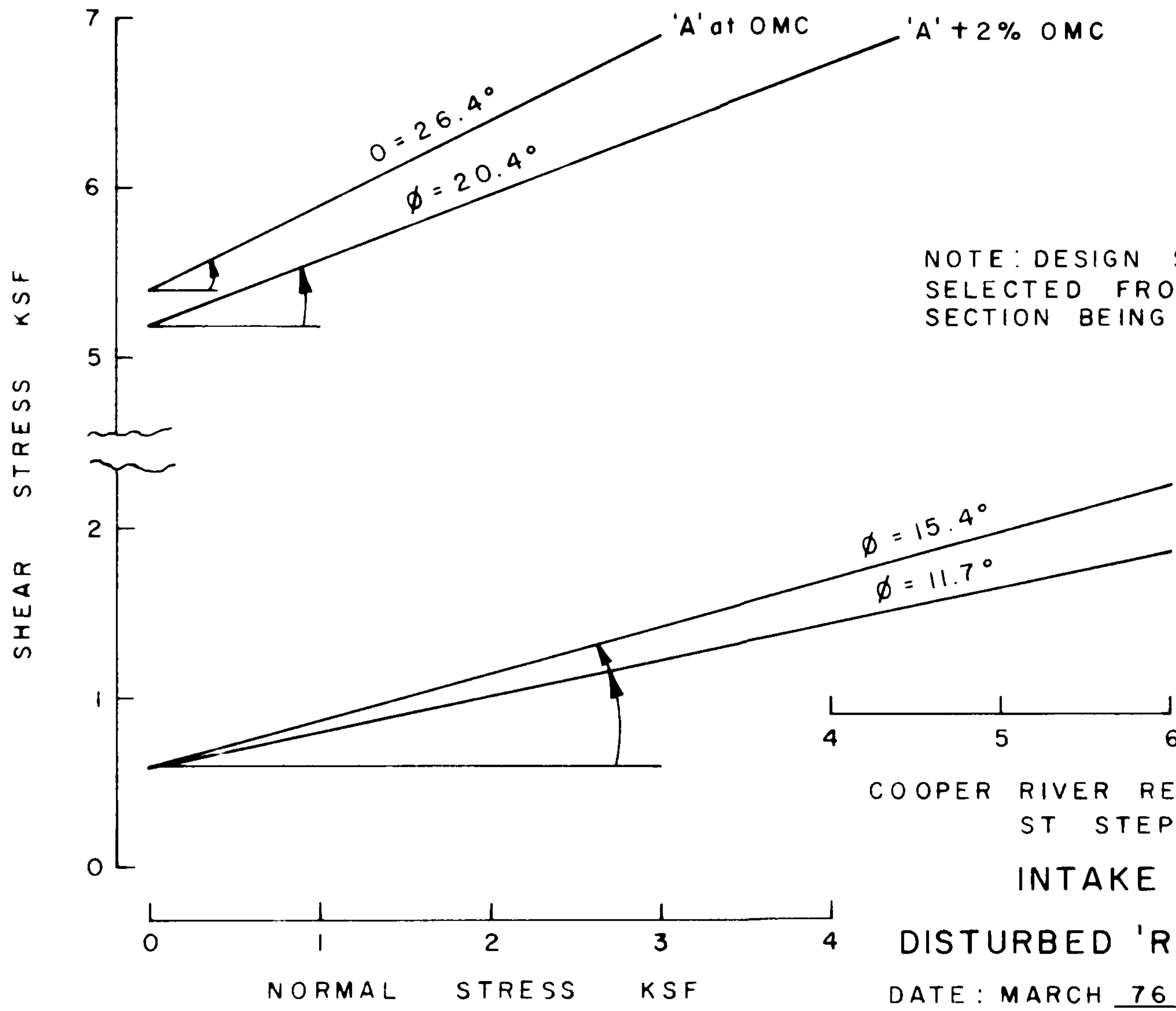
NOTE: DESIGN STRENGTHS SELECTED FROM TESTS AT SECTION BEING ANALYZED

COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S. C.

INTAKE CANAL

DISTURBED 'Q' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-5



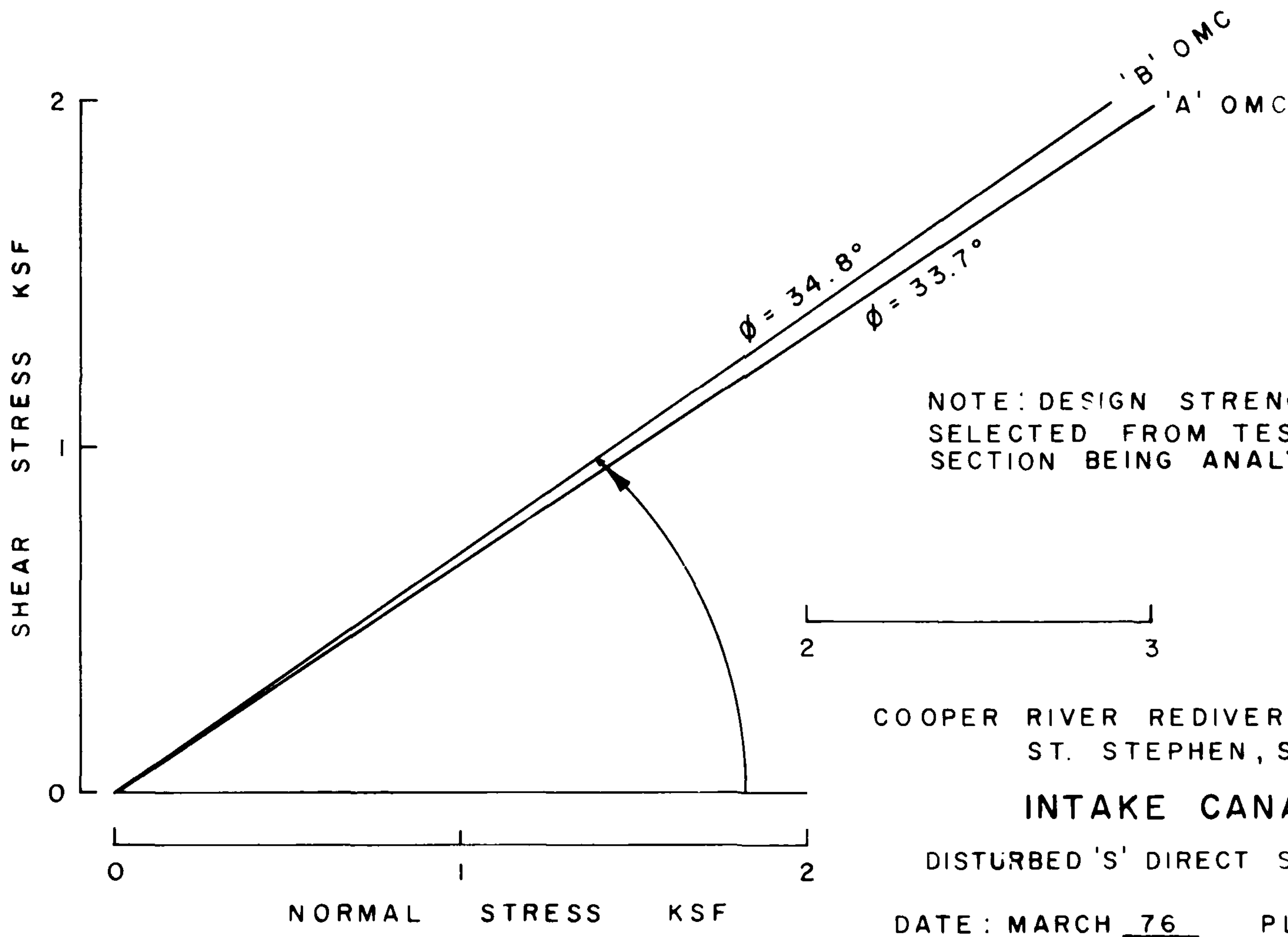
NOTE: DESIGN STRENGTHS SELECTED FROM TESTS AT SECTION BEING ANALYZED

COOPER RIVER REDIVERSION PROJECT
ST STEPHEN, S.C.

INTAKE CANAL

DISTURBED 'R' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-6



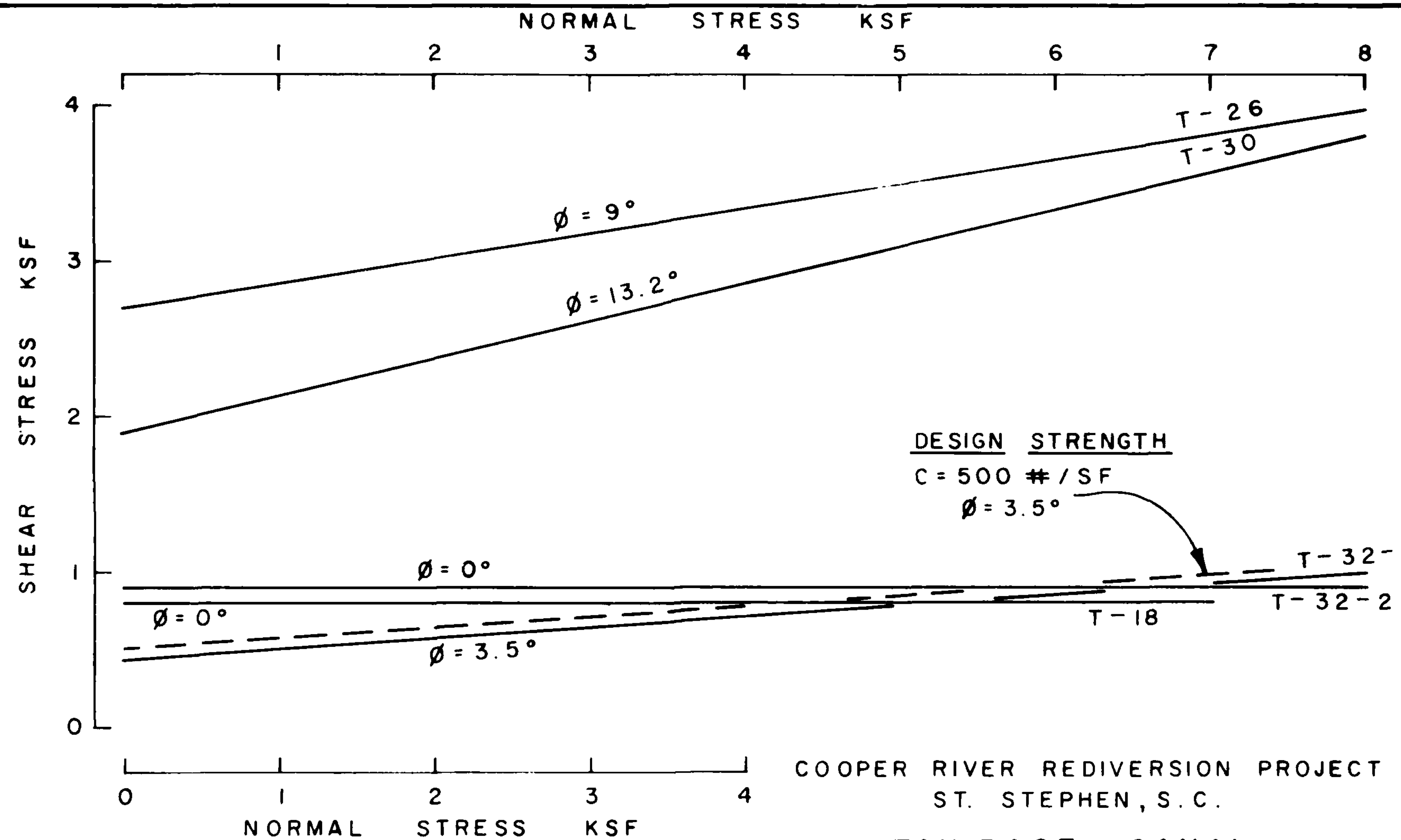
NOTE: DESIGN STRENGTHS SELECTED FROM TESTS AT SECTION BEING ANALYZED

COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

INTAKE CANAL

DISTURBED 'S' DIRECT SHEAR TESTS

DATE: MARCH 76   PLATE NO. D-7

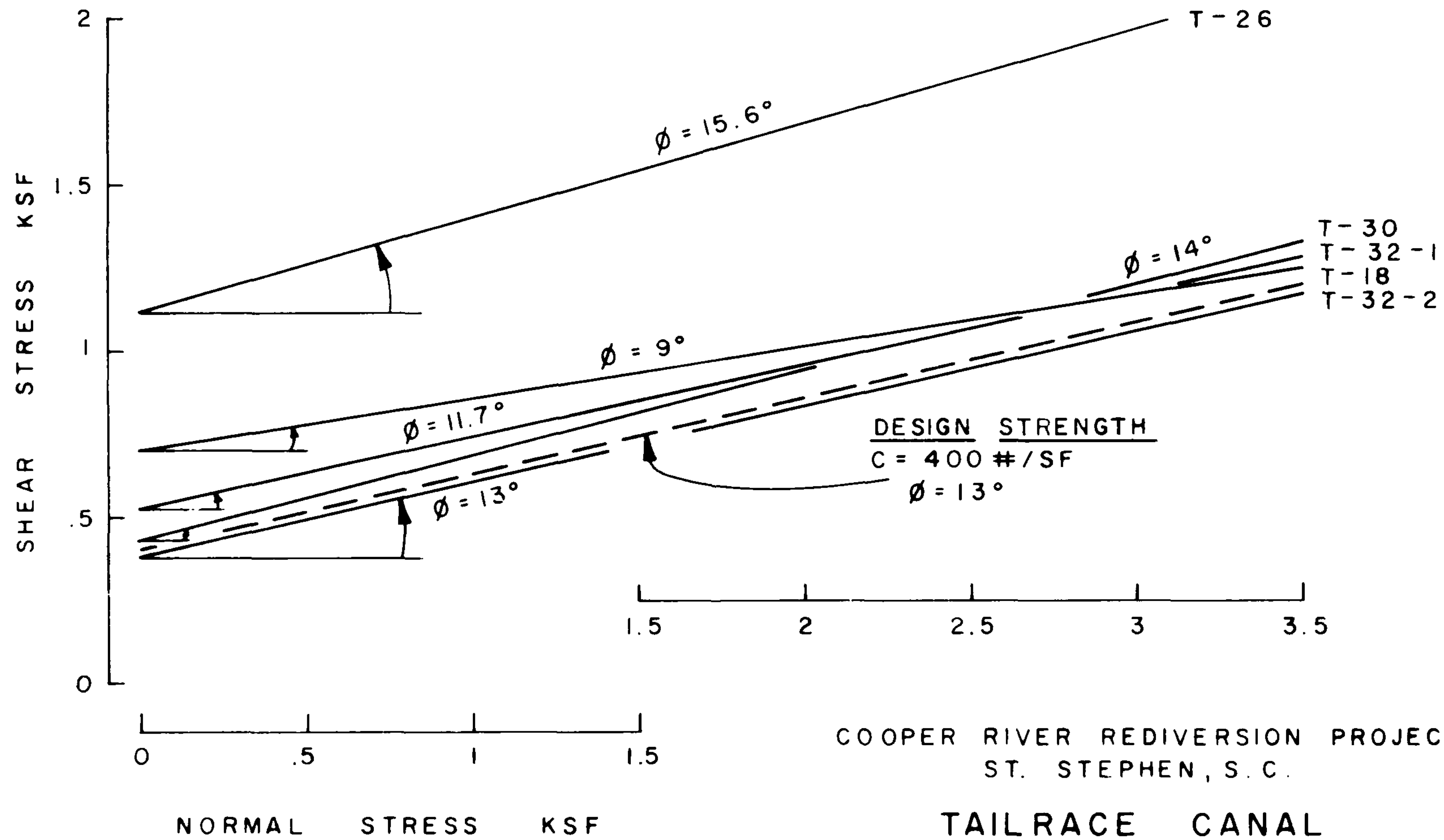


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

UNDISTURBED 'Q' TRIAXIAL TESTS

DATE : MARCH 76 PLATE NO. D-8

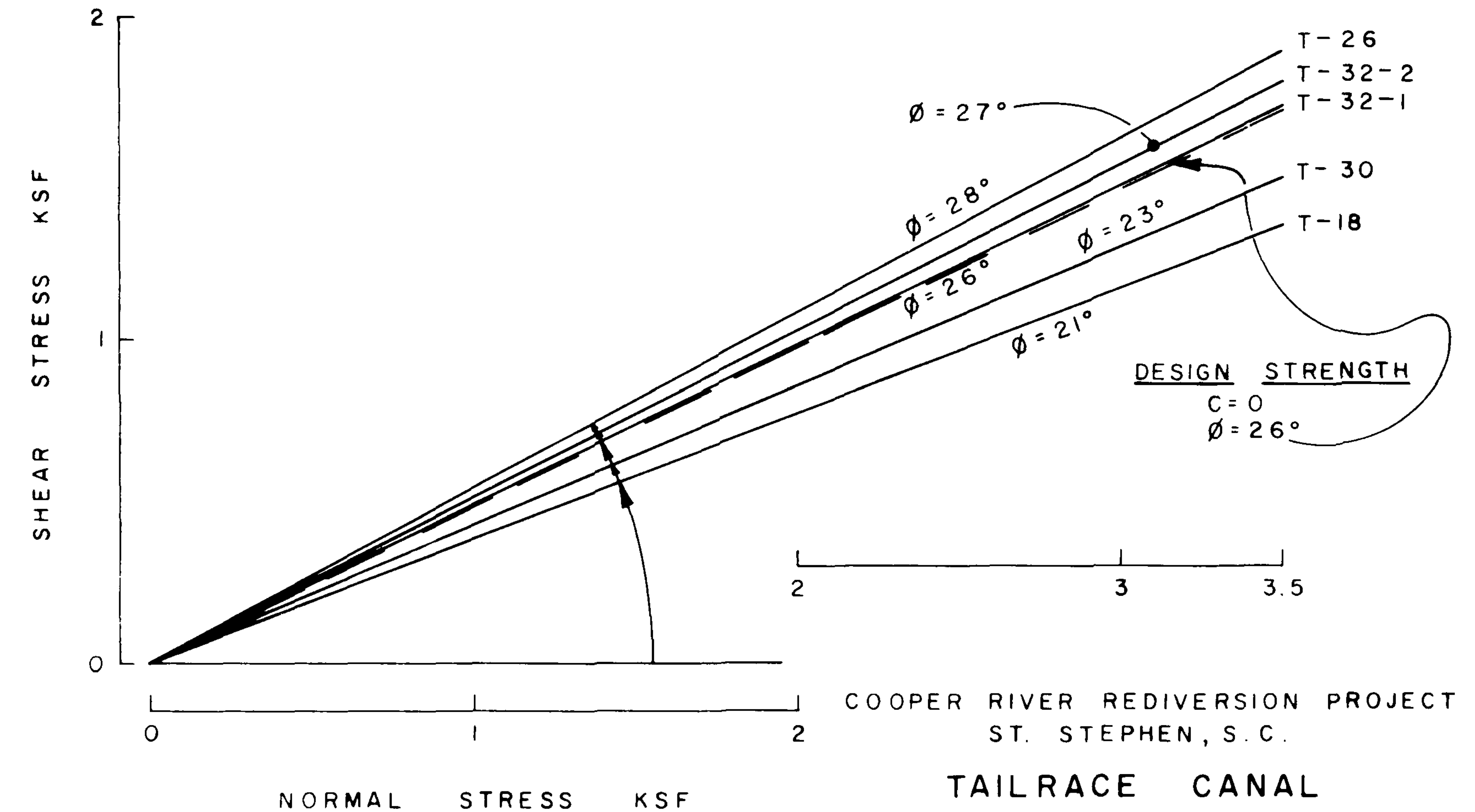


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

UNDISTURBED 'R' TRIAXIAL TESTS

DATE: MARCH 76   PLATE NO. D-9

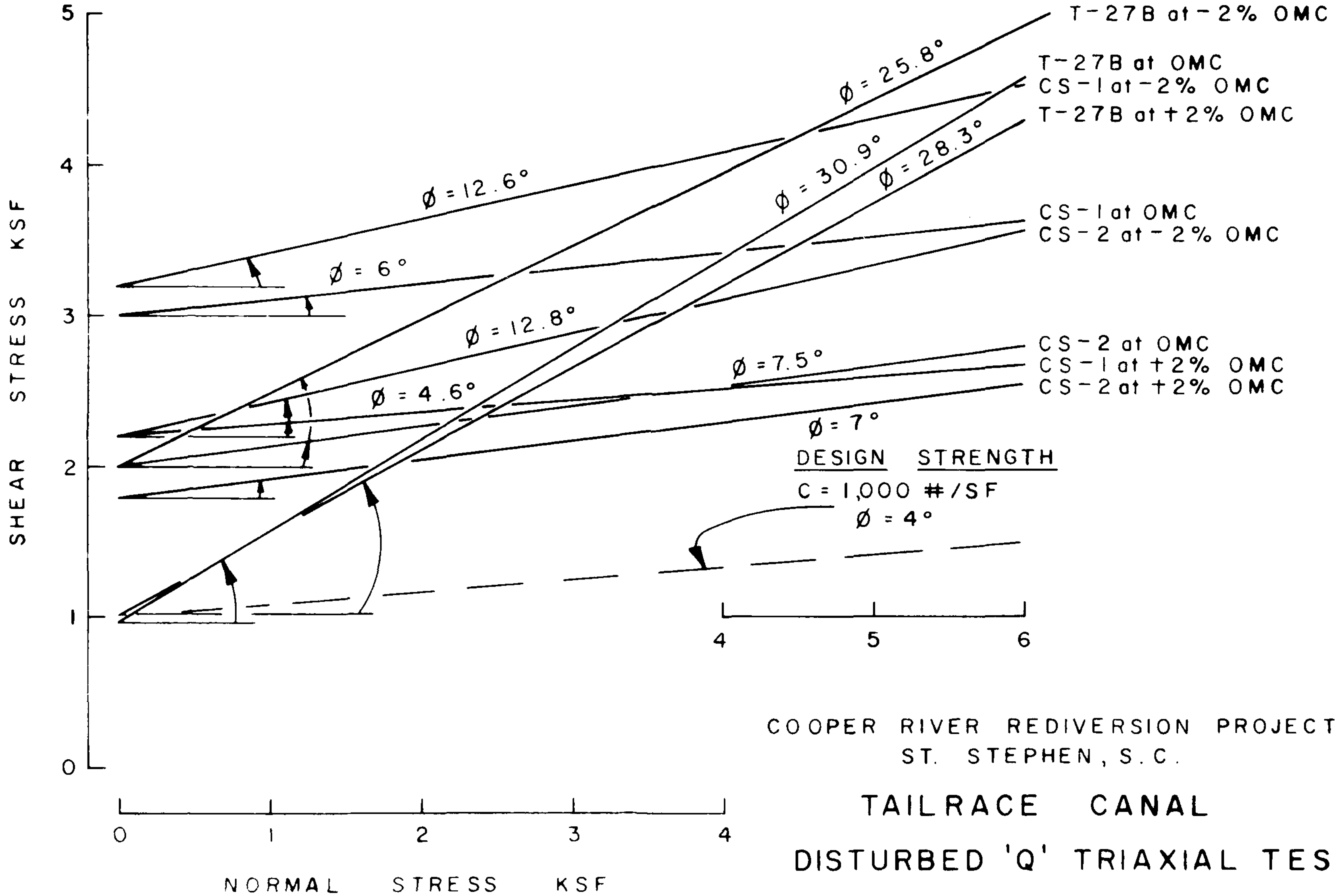


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

UNDISTURBED 'S' DIRECT SHEAR TESTS

DATE : MARCH 76 PLATE NO. D-10

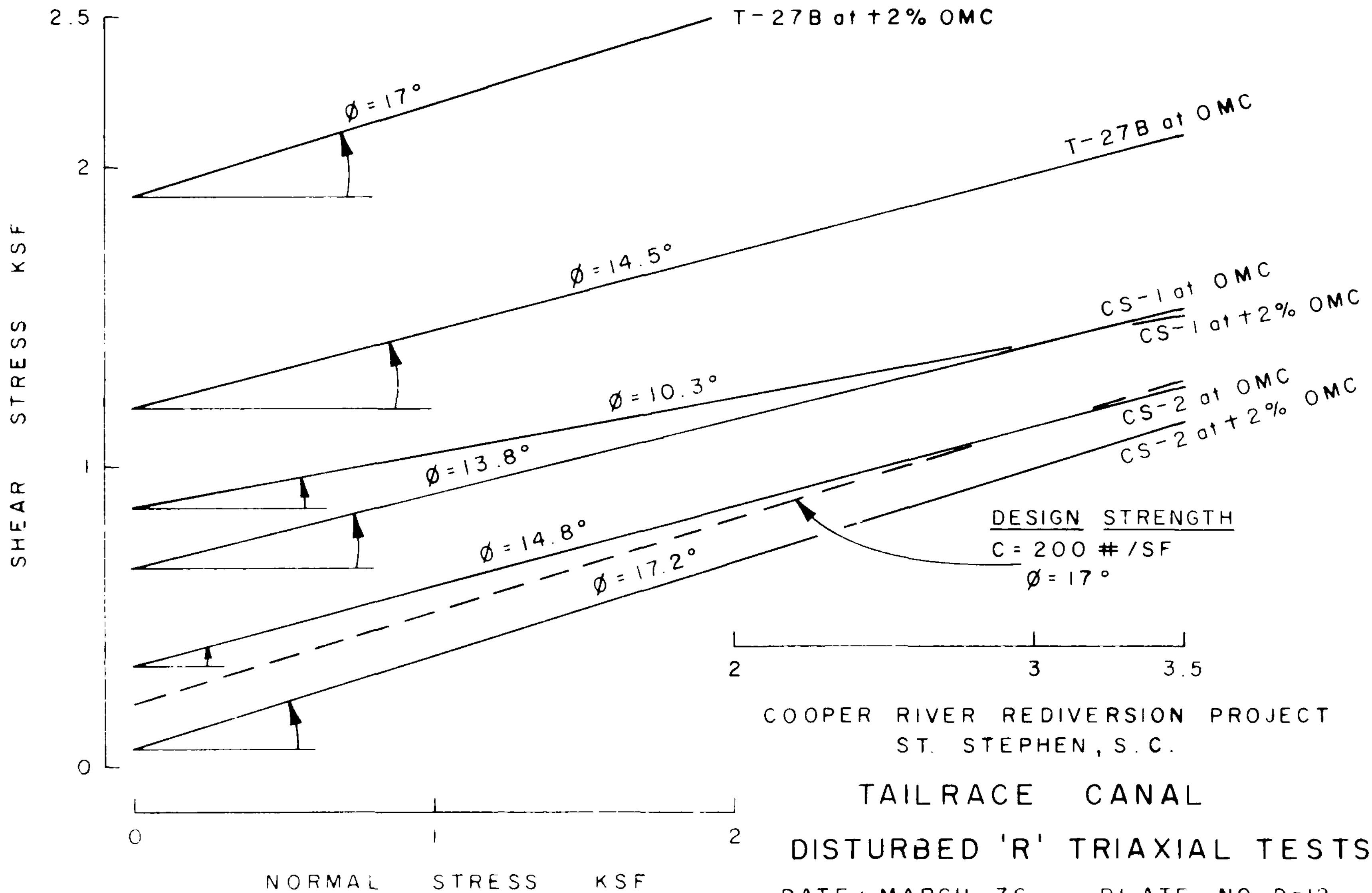


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

DISTURBED 'Q' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-11



COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

DISTURBED 'R' TRIAXIAL TESTS

DATE: MARCH 76 PLATE NO. D-12

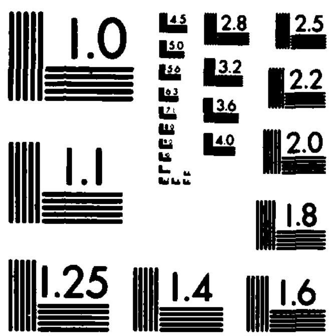
AD-A149 941 COOPER RIVER REDIVERSION PROJECT LAKE MOULTRIE AND SANTEE RIVER SOUTH CAROLINA INTAKE AND TAILRACE CANALS (U) CORPS OF ENGINEERS CHARLESTON SC CHARLESTON DISTRICT JUN 76 5/5

UNCLASSIFIED F/G 13/2 NL

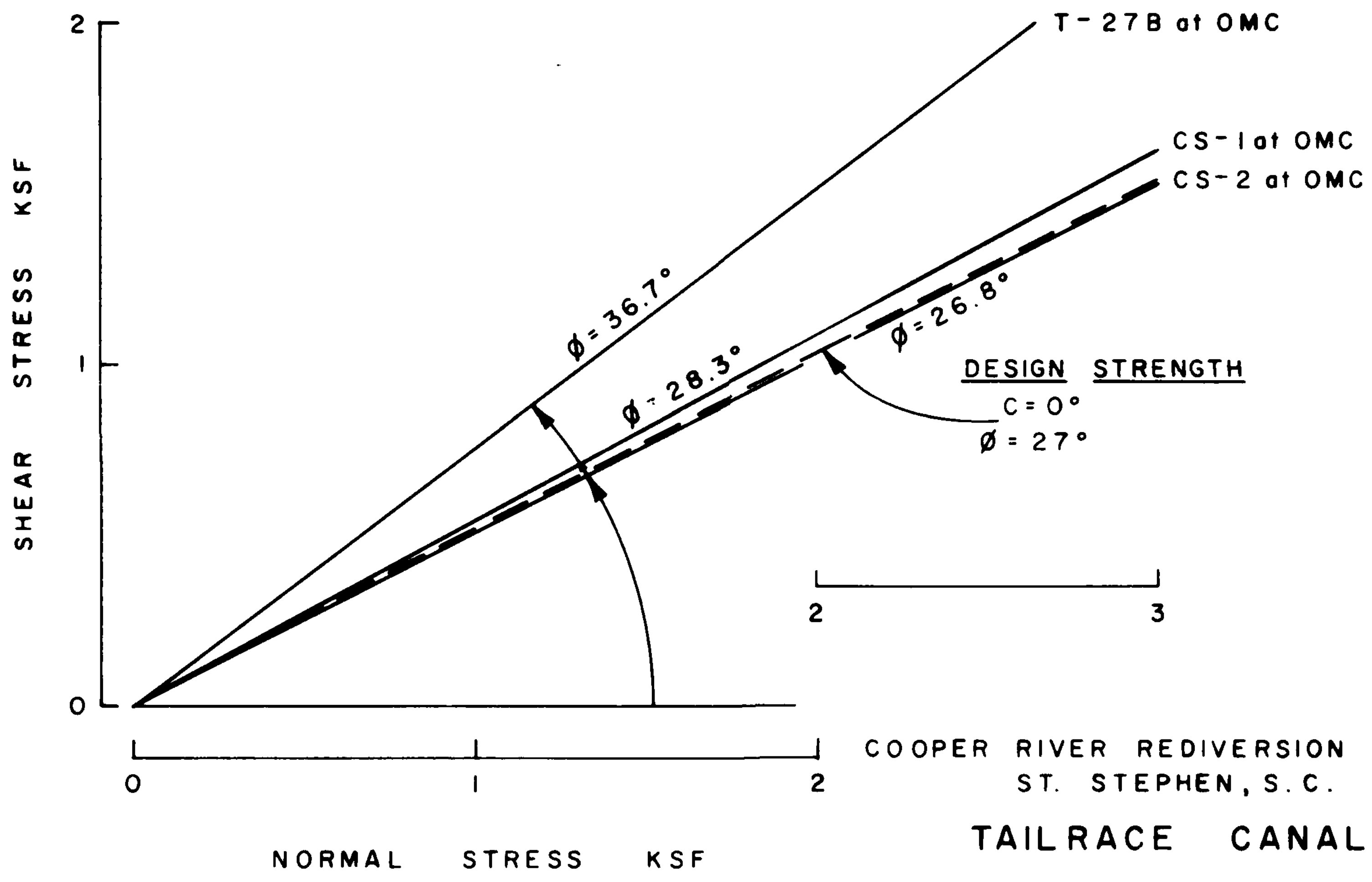
END

FILMED

DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

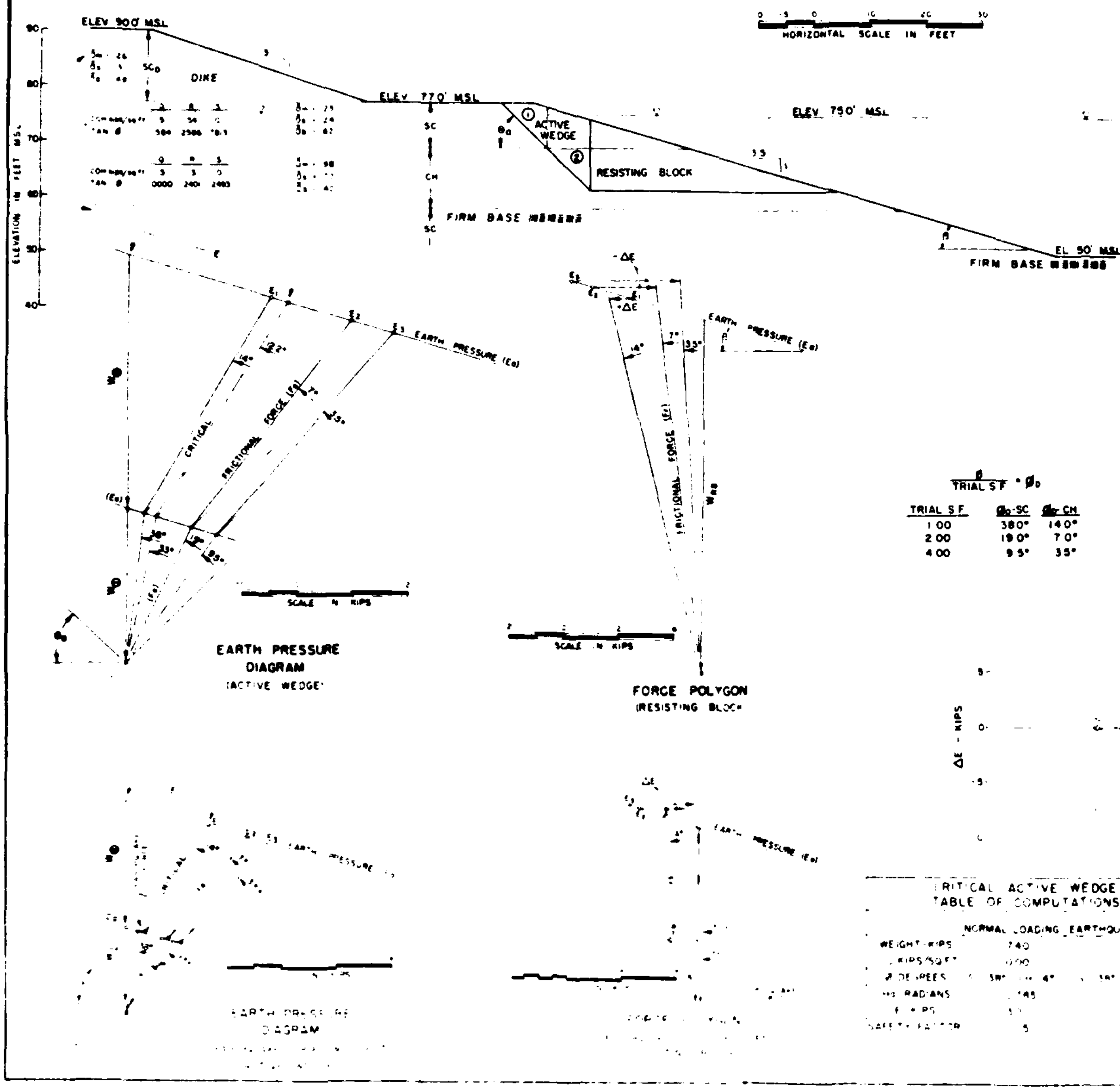


COOPER RIVER REDIVERSION PROJECT
ST. STEPHEN, S.C.

TAILRACE CANAL

DISTURBED 'S' DIRECT SHEAR TESTS

DATE: MARCH 76   PLATE NO. D-13



EARTH PRESSURE DIAGRAM
(ACTIVE WEDGE)

FORCE POLYGON
(RESISTING BLOCK)

DESIGN DATA

1. CASE: STEADY SEEPAGE
2. METHOD: WEDGE
3. SLOPE: 1 ON 3.5 H
4. HEIGHT = 27'
5. BOTTOM OF CANAL ELEV. = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV. = 75.0' MSL
7. NORMAL GWT ELEV. = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

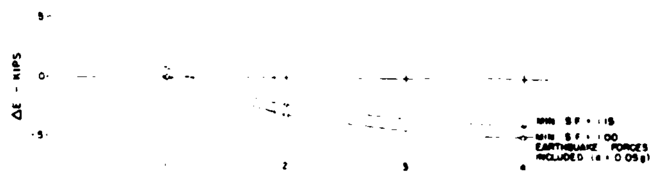
NOTES

1. ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLOT BELOW).
2. Ea ACTS PARALLEL TO SLOPE OF CUT.

$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$

TRIAL S.F.	ϕ_D-SC	ϕ_D-CH
1.00	38.0°	14.0°
2.00	19.0°	7.0°
4.00	9.5°	3.5°

CHANGE FROM 'S' TO $\frac{R+S}{2}$ WILL NOT INFLUENCE THIS ANALYSIS

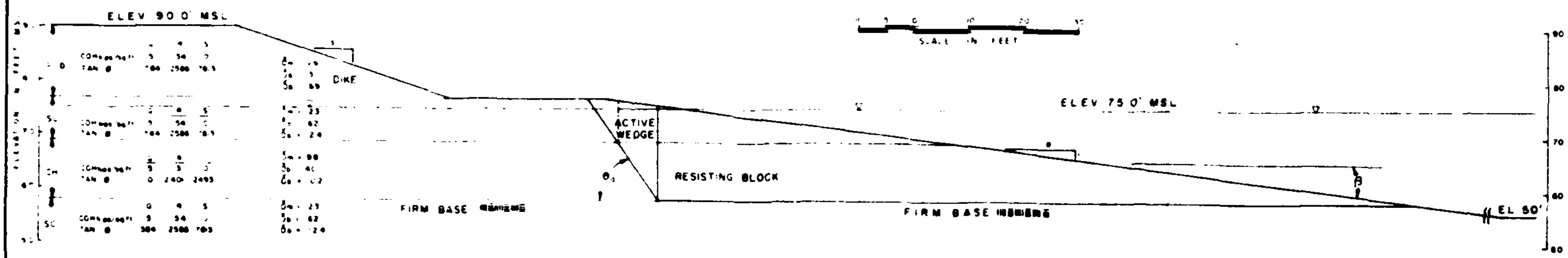


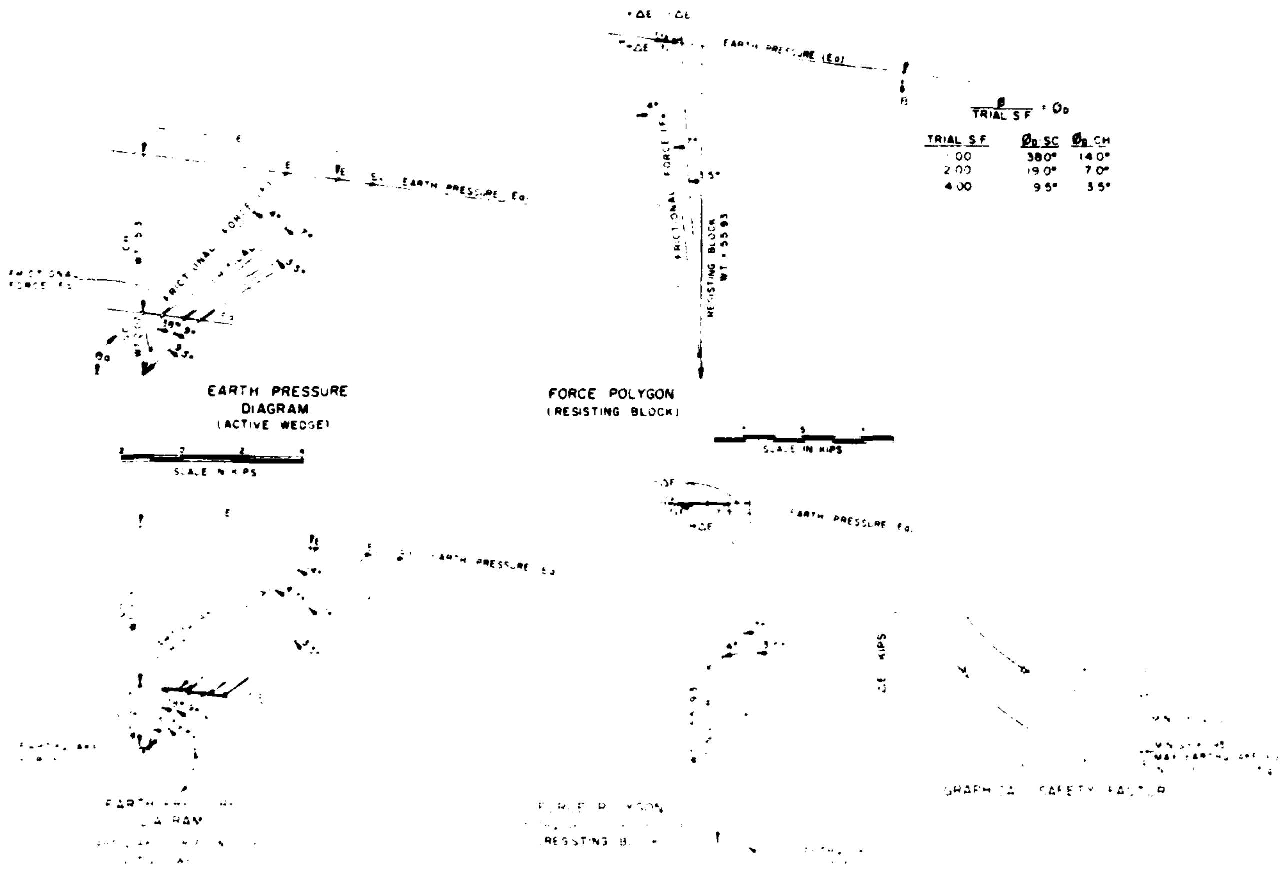
GRAPHICAL SAFETY FACTOR

CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

	NORMAL LOADING	EARTHQUAKE LOADING
WEIGHT-KIPS	740	740
[illegible] KIPS/SQ FT	[illegible]	[illegible]
ϕ DEGREES	[illegible]	[illegible]
[illegible] RADIANS	[illegible]	[illegible]
E KIPS	[illegible]	[illegible]
SAFETY FACTOR	[illegible]	[illegible]

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.
SLOPE STABILITY ANALYSIS
INTAKE CANAL
C/L STATION 140 + 00
CASE: STEADY SEEPAGE
SLOPE: 1 ON 3.5H
METHOD: WEDGE
DATE: APRIL 76
PLATE B-14





CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

	NORMAL LOADING	MAXIMUM EARTHQUAKE LOADING
WEIGHT-KIPS	74	74
C-KIPS/SQ.FT.	0.0	0.0
Ø-DEGREES	SC-38°, CH-14°	SC-38°, CH-14°
θa-RADIANS	0.969	0.969
E-KIPS	6.50	5.75
SAFETY FACTOR	2.0	0.95

DESIGN DATA

1. CASE: STEADY SEEPAGE
2. METHOD: WEDGE
3. SLOPE: 1 ON 8H
4. HEIGHT = 27'
5. BOTTOM OF CANAL ELEV. = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV. = 75.0' MSL
7. NORMAL GWT ELEV. = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

NOTES

1. ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLOT ON PLATE D-14)
2. Ea ACTS PARALLEL TO SLOPE OF CUT

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

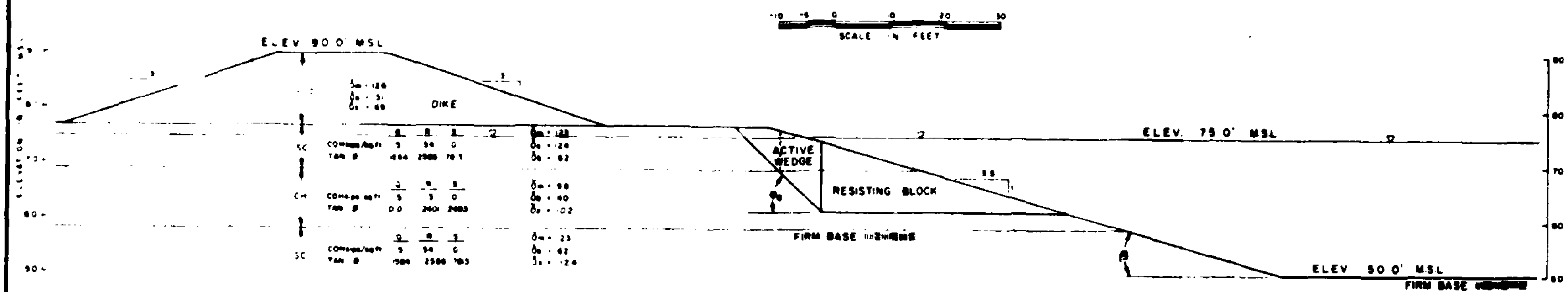
SLOPE STABILITY ANALYSIS

INTAKE CANAL

C/L STATION 140 + 00

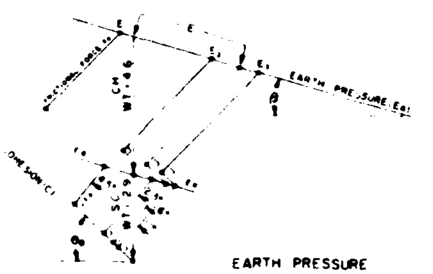
CASE: STEADY SEEPAGE
SLOPE: 1 ON 8H
METHOD: WEDGE
DATE: APRIL 76

PLATE D-15

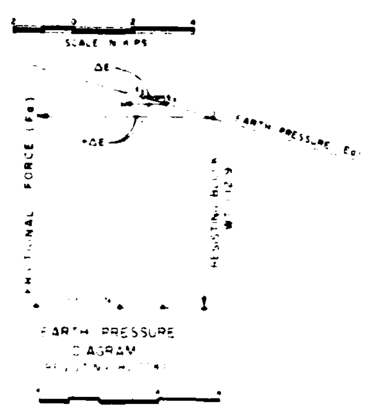


$$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$$

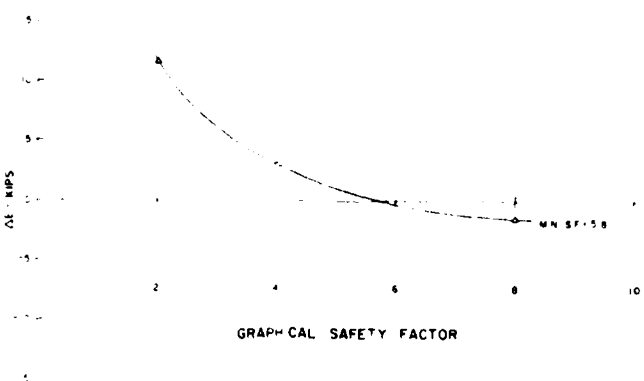
TRIAL S.F.	ϕ_D SC	ϕ_D CH
1.00	4.5°	0°
2.00	2.3°	0°
4.00	1.1°	0°



EARTH PRESSURE
DIAGRAM
(ACTIVE WEDGE)



EARTH PRESSURE
DIAGRAM



GRAPHICAL SAFETY FACTOR

CRITICAL ACTIVE WEDGE TABLE OF COMPUTATIONS		
	NORMAL LOADING	EARTHQUAKE LOADING
WEIGHT-KIPS	7.5	SEE NOTE NO. 2
C_{SC} KIPS/SQ FT	1/1 = 0.96	• • •
C_{CH} "	1/1 = 0.96	• • •
C_{eq} "	= 3.86	• • •
ϕ DEGREES	SC = 4.5°, CH = 0°	• • •
θ_0 - RADIANS	767	• • •
E-KIPS	3.7	• • •
SAFETY FACTOR	5.8	• • •

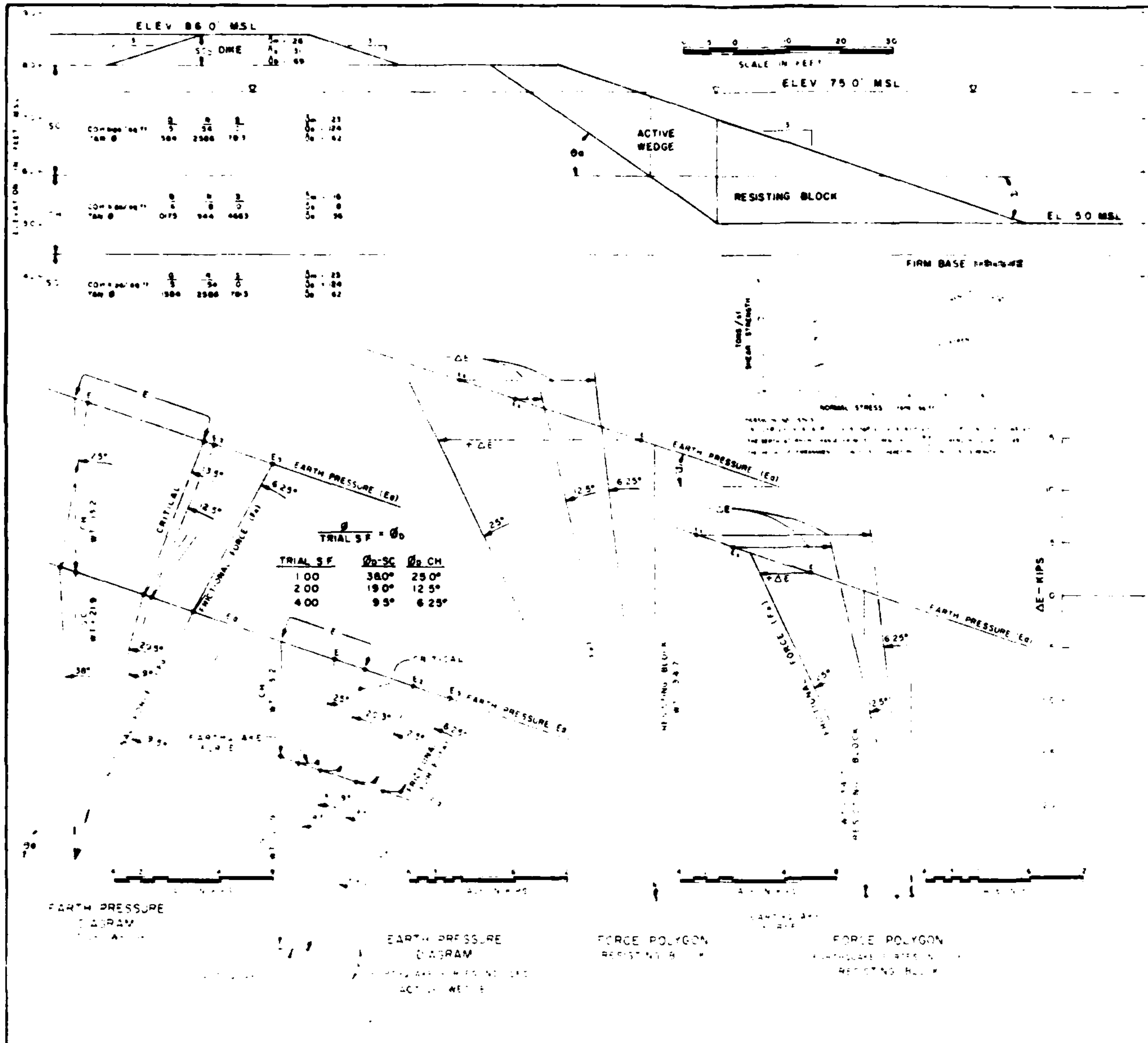
DESIGN DATA

1 CASE: END OF CONSTRUCTION
2 METHOD WEDGE
3 SLOPE 1 ON 3.5 H
4 HEIGHT = 27'
5 BOTTOM OF CANAL ELEV = 50.0' MSL
6 NORMAL CANAL SURFACE ELEV = 75.0' MSL
7 NORMAL GWT ELEV = 75.0' MSL
8 ASSUME NO TENSION CRACKS
9 ASSUME E_a ACTS PARALLEL TO SLOPE OF CUT
10 ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

NOTES

1 ONLY Q STRENGTHS WERE USED FOR STRENGTH CALCULATIONS
2 NO EARTHQUAKE FORCES CALCULATED DUE TO SHORT TERM CONDITION

COOPER RIVER REDIVERSION PROJECT ST STEPHEN, S.C.
SLOPE STABILITY ANALYSIS
INTAKE CANAL
C/L STATION 140 + 00
CASE END OF CONSTRUCTION
SLOPE 1 ON 3.5 H
METHOD WEDGE
DATE APRIL 76 PLATE D 16



$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$

TRIAL S.F.	ϕ_D-SC	ϕ_D CH
1.00	38.0°	25.0°
2.00	19.0°	12.5°
4.00	9.5°	6.25°

DESIGN DATA

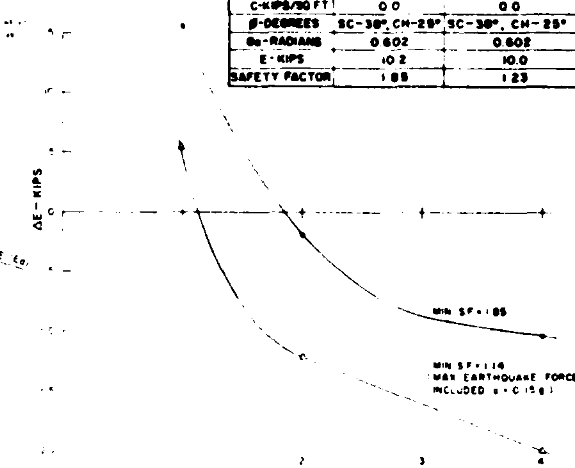
1. CASE STEADY SEEPAGE
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT = 30'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

NOTES

1. ONLY S STRENGTHS WERE USED FOR STRENGTH CALCULATION (SEE PLOT AT LEFT)
2. Ea ACTS PARALLEL TO SLOPE OF CUT

CRITICAL ACTIVE WEDGE TABLE OF COMPUTATIONS

	NORMAL LOADING	MAXIMUM EARTHQUAKE LOADING
WEIGHT-KIPS	35.1	35.1
C-KIPS/SQ FT	0.0	0.0
ϕ-DEGREES	SC-38°, CH-25°	SC-38°, CH-25°
θ_D-RADIANS	0.602	0.602
E-KIPS	10.2	10.0
SAFETY FACTOR	1.85	1.23



GRAPHICAL SAFETY FACTOR

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

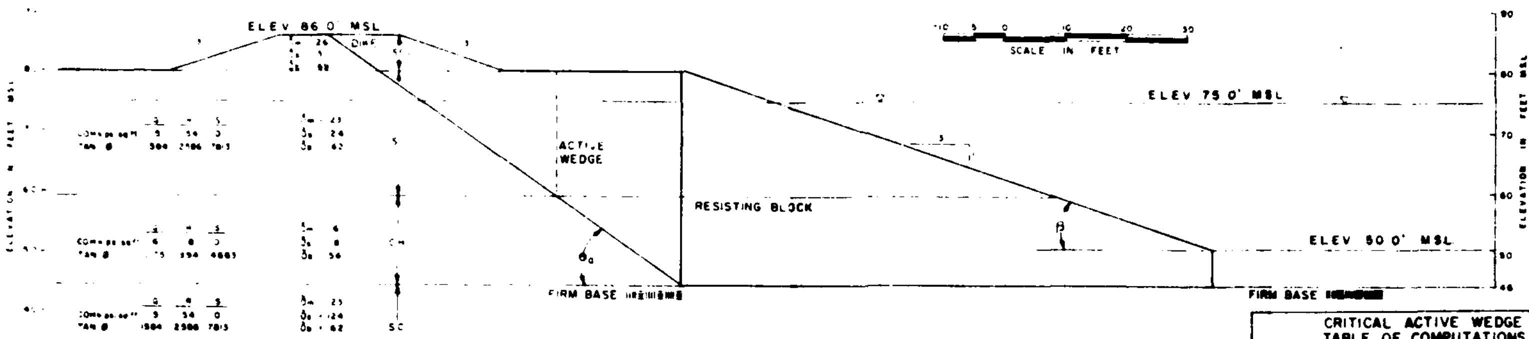
SLOPE STABILITY ANALYSIS

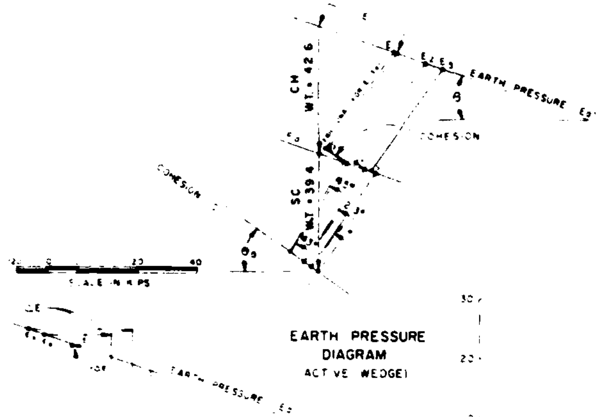
INTAKE CANAL

C/L STATION 194 + 30

CASE STEADY SEEPAGE
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 76

PLATE B 17





EARTH PRESSURE DIAGRAM (ACTIVE WEDGE)

$$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$$

TRIAL S.F.	ϕ_D SC	ϕ_D CH
1.00	4.5°	0.5°
2.00	2.3°	0.25°
4.00	1.1°	0.12°

CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

	NORMAL LOADING	EARTHQUAKE LOADING
WEIGHT-KIPS	82.0	SEE NOTE NO 3
C_D - KIPS/SQFT	= .111	" " "
C_{CH} "	= .74	" " "
C_{SC} "	= .253	" " "
ϕ DEGREES	SC-4.5°, CH-0.5°	" " "
θ_a RADIANS	.611	" " "
E-KIPS	27.5	" " "
SAFETY FACTOR	2.1	" " "

DESIGN DATA

1. CASE: END OF CONSTRUCTION
2. METHOD: WEDGE
3. SLOPE: 1 ON 3H
4. HEIGHT = 30'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME E_a ACTS PARALLEL TO SLOPE OF CUT
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

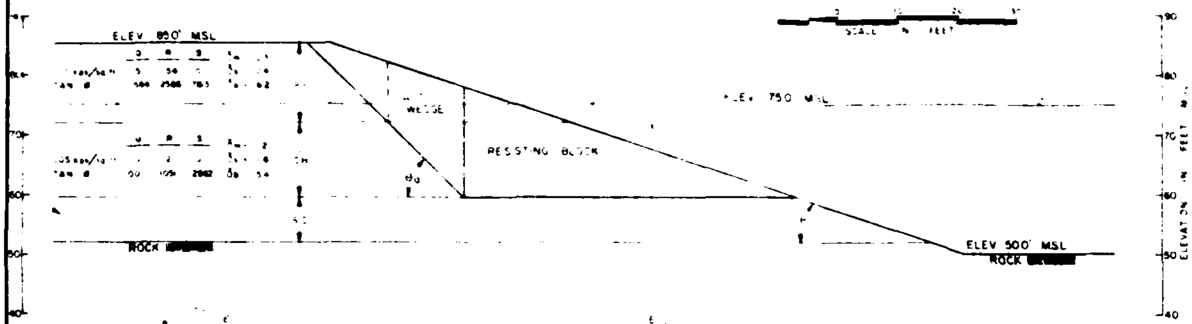
NOTES

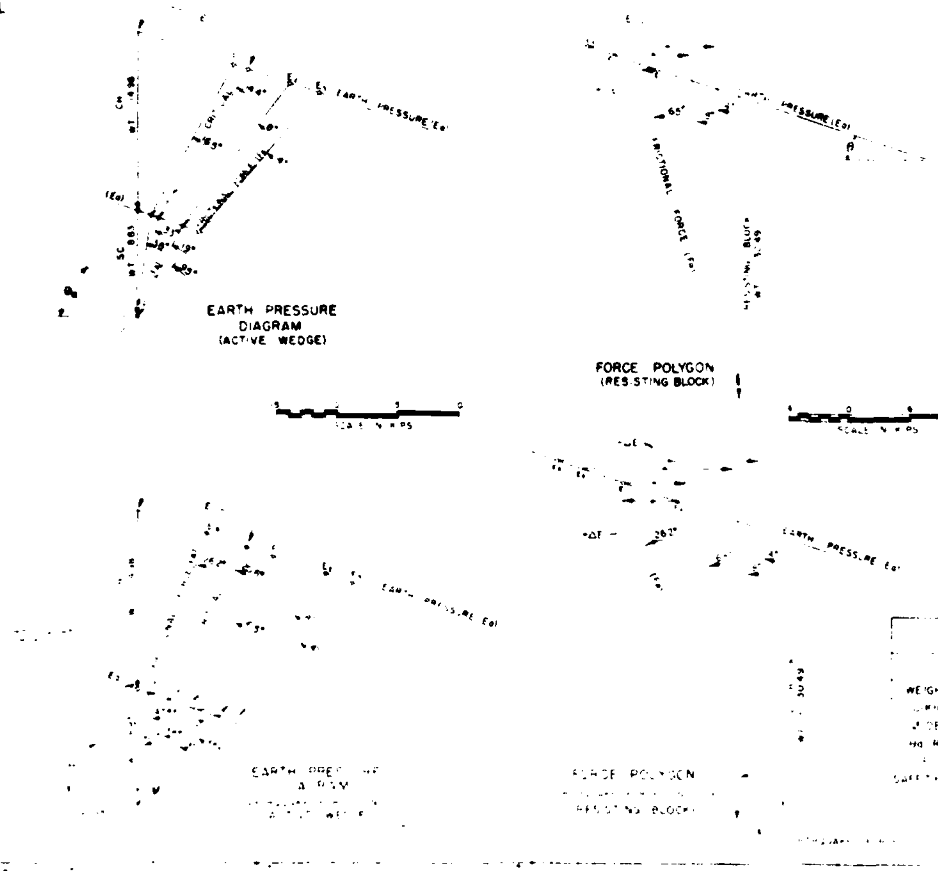
1. ONLY 'Q' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS
2. NO PASSIVE EARTH PRESSURE CONSIDERED
3. NO EARTHQUAKE FORCES CALCULATED DUE TO SHORT TERM CONDITION

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

SLOPE STABILITY ANALYSIS
INTAKE CANAL
C/L STATION 194 + 30
CASE: END OF CONSTRUCTION
SLOPE: 1 ON 3H
METHOD: WEDGE
DATE: APRIL 76

PLATE D 18





EARTH PRESSURE
DIAGRAM
(ACTIVE WEDGE)

FORCE POLYGON
(RESISTING BLOCK)

$\text{TRIAL S.F.} = \frac{\tan \emptyset}{\tan \emptyset_D}$

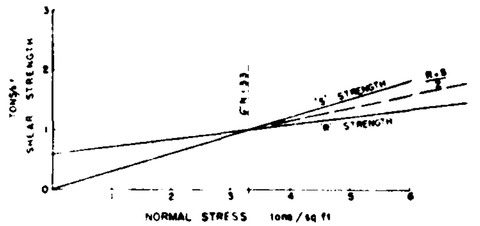
TRIAL S.F.	$\emptyset_D$ SC	$\emptyset_D$ CH
1.00	36.0°	16.5°
2.00	19.0°	8.0°
4.00	9.5°	4.0°

DESIGN DATA

1. CASE STEADY SEEPAGE
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT = 35'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

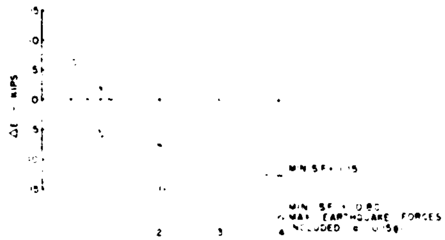
NOTES

1. ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLOT BELOW)
2. Ea ACTS PARALLEL TO SLOPE OF CUT.



DEPTH OF INFLUENCE

THE DEPTH AT WHICH CHANGE FROM 'S' STRENGTH TO $\frac{R+S}{2}$ STRENGTH OCCURS BELOW LIMESTONE



GRAPHICAL SAFETY FACTOR

CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

	NORMAL LOADING	MAXIMUM EARTHQUAKE LOADING
WEIGHT KIPS	[illegible]	[illegible]
KIPS/SQ FT	[illegible]	[illegible]
DEGREES	[illegible]	[illegible]
RADIANS	[illegible]	[illegible]
KIPS	[illegible]	[illegible]
SAFETY FACTOR	[illegible]	[illegible]

COOPER RIVER REDIVERSION PROJECT ST STEPHEN, S.C

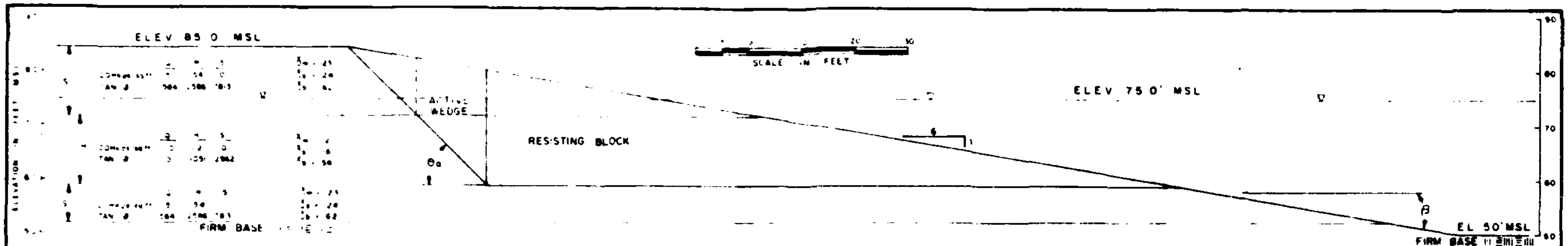
SLOPE STABILITY ANALYSIS

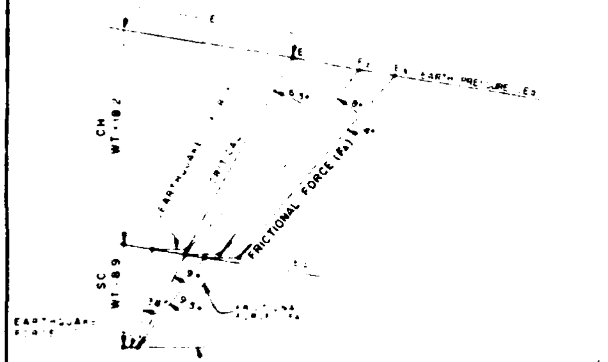
INTAKE CANAL

C/L STATION 246 + 00

CASE STEADY SEEPAGE
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 76

PLATE D 19





EARTH PRESSURE DIAGRAM
EARTHQUAKE FORCE INCLUDED
(ACTIVE WEDGE)

SCALE IN KIPS

FORCE POLYGON
EARTHQUAKE FORCES INCLUDED
(RESISTING BLOCK)

SCALE IN KIPS

$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$

TRIAL S.F.	ϕ_D-SC	ϕ_D-CH
1.00	38.0°	16.5°
2.00	19.0°	8.0°
4.00	9.5°	4.0°

CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

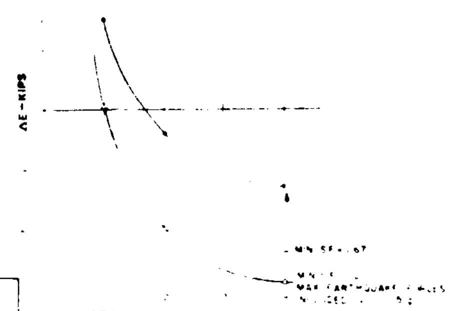
	NORMAL LOADING	MAXIMUM EARTHQUAKE LOADING
WEIGHT-KIPS	[illegible]	[illegible]
KIPS/SQFT	[illegible]	[illegible]
Ø DEGREES	[illegible] 6.5°	[illegible] 6.5°
[illegible]	[illegible]	[illegible]
[illegible]	4.2	5.[illegible]
SAFETY FACTOR	[illegible]	[illegible]

DESIGN DATA

1. CASE: STEADY SEEPAGE
2. METHOD: WEDGE
3. SLOPE: 1 ON 6 H
4. HEIGHT = 35'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV. = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

NOTES

1. ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATION (SEE PLATE D-19)
2. Ea ACTS PARALLEL TO SLOPE OF CUT



GRAPHICAL SAFETY FACTOR

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN S.C.

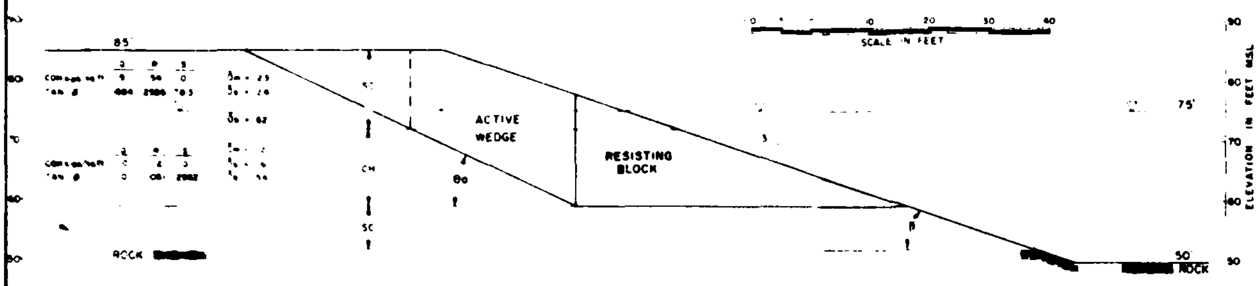
SLOPE STABILITY ANALYSIS

INTAKE CANAL

C/L STATION 246 + 00

CASE STEADY SEEPAGE
SLOPE 1 ON 6H
METHOD WEDGE
DATE APRIL 78

PLATE D 20

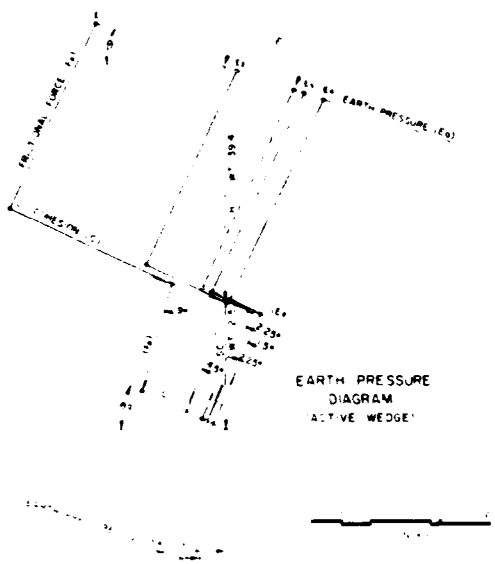


DESIGN DATA

1. CASE END OF CONSTRUCTION
2. METHOD WEDGE
3. SLOPE 1 ON 3 H
4. HEIGHT = 35'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME NO EARTHQUAKE FORCES
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

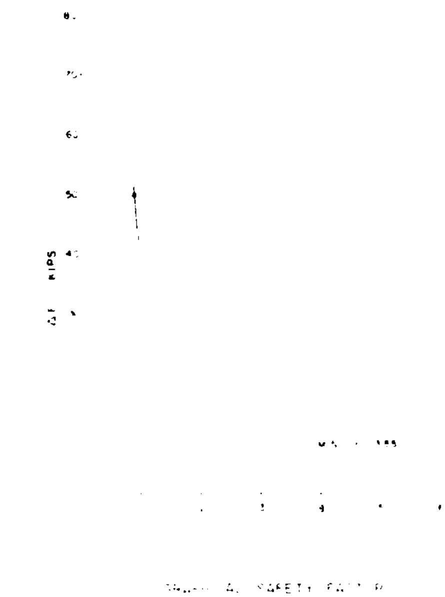
NOTES

1. ONLY 'Q' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS
2. Ea ACTS PARALLEL TO SLOPE OF CUT.



$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$

TRIAL S.F.	ϕ_D SC	ϕ_D CH
1.0	9.0°	0.0°
2.0	4.5°	0.0°
4.0	2.25°	0.0°
6.0	1.5°	0.0°



CRITICAL ACTIVE WEDGE TABLE OF COMPUTATIONS		
	NORMAL LOADING	EARTHQUAKE LOADING
		SEE DESIGN DATA - NO. 9
WEIGHT-KIPS	50.90	
C_{SC} - KIPS/SQFT	[illegible] = 4.40	
C_{CH} - "	[illegible] = 8.60	
C_{SB} - "	[illegible] = 15.80	
ϕ - DEGREES	SC - 9°, CH - 0°	
θ_D - RADIANS	0.436	
E - KIPS	12.00	
SAFETY FACTOR	3.55	

COOPER RIVER REDIVERSION PROJECT ST STEPHEN, S.C.

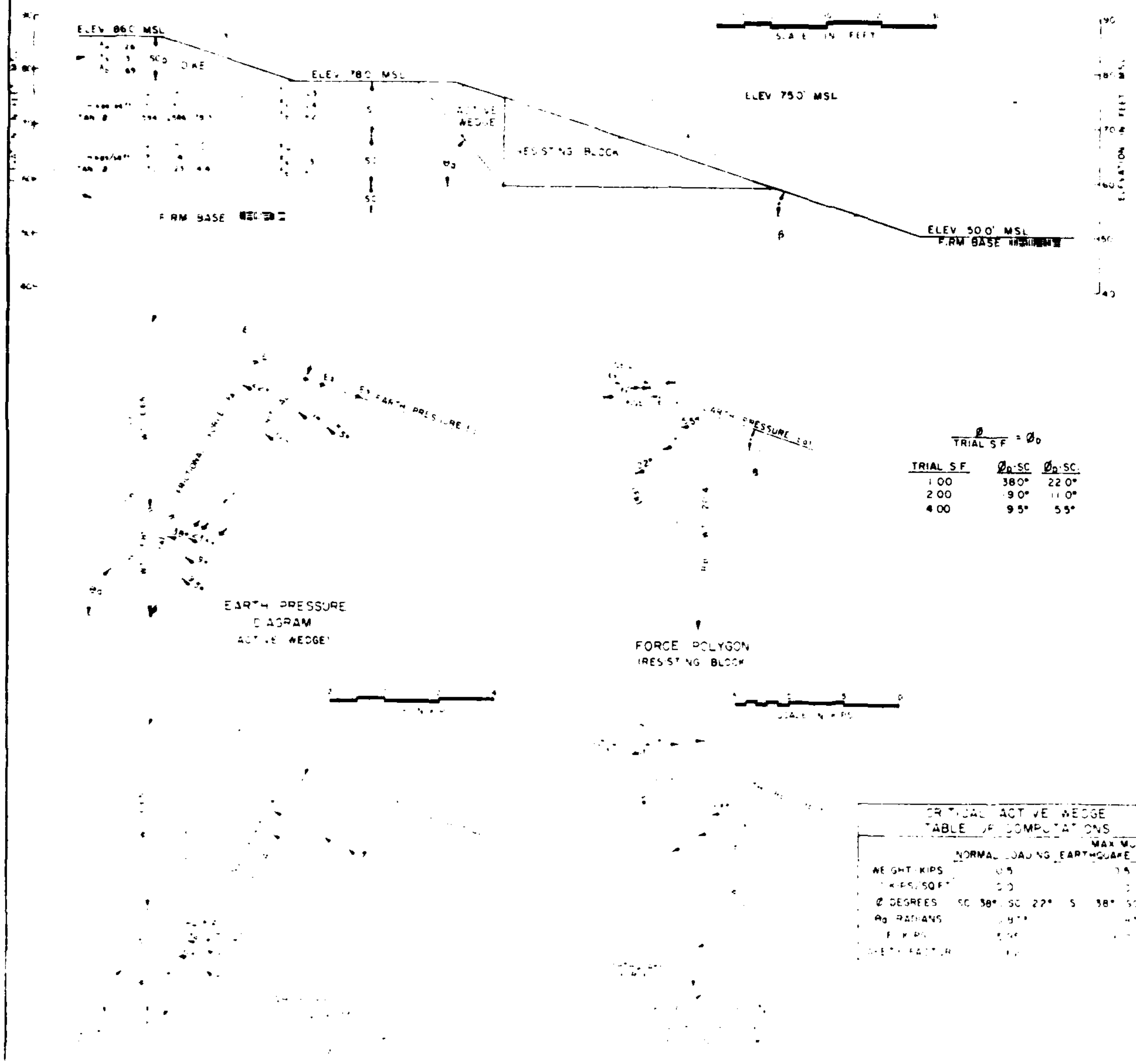
SLOPE STABILITY ANALYSIS

INTAKE CANAL

C/L STATION 246 + 00

CASE END OF CONSTRUCTION
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 76

PLATE D 21



$$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$$

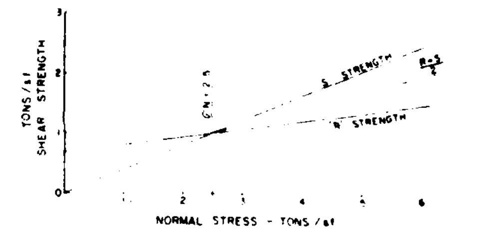
TRIAL S.F.	ϕ_D S	ϕ_D SC
1.00	38.0°	22.0°
2.00	19.0°	11.0°
4.00	9.5°	5.5°

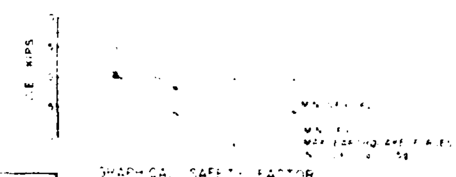
DESIGN DATA

1. CASE STEADY SEEPAGE
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT = 28'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

NOTES

1. ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLOT BELOW)
2. E_a ACTS PARALLEL TO SLOPE OF CUT





CRITICAL ACTIVE WEDGE
TABLE OF COMPUTATIONS

	NORMAL LOADING	MAXIMUM EARTHQUAKE LOADING
WEIGHT KIPS	[illegible]	[illegible]
KIPS/SQ FT	[illegible]	[illegible]
ϕ DEGREES	S 38° SC 22°	S 38° SC 22°
θ_D RADIANS	[illegible]	[illegible]
E_a KIPS	[illegible]	[illegible]
SAFETY FACTOR	[illegible]	[illegible]

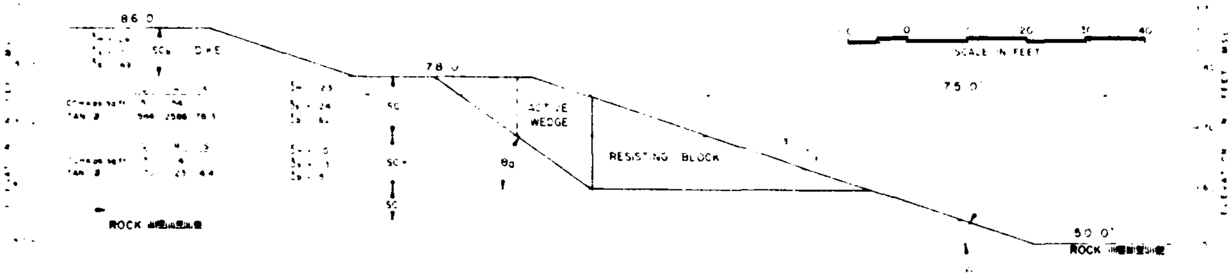
COOPER RIVER REDIVERSION PROJECT ST. STEPHEN S.C.

SLOPE STABILITY ANALYSIS

INTAKE CANAL

C/L STATION 272 + 75

CASE STEADY SEEPAGE
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 75 PLATE D-22



DESIGN DATA

1. CASE END OF CONSTRUCTION
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT = 28'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 75.0' MSL
8. ASSUME NO TENSION CRACKS
9. NO EARTHQUAKE LOAD WAS CALCULATED DUE TO SHORT TERM CONDITION
10. ASSUME STRAIGHT (OR CONTINUOUS) FAILURE LINE FOR ACTIVE WEDGE

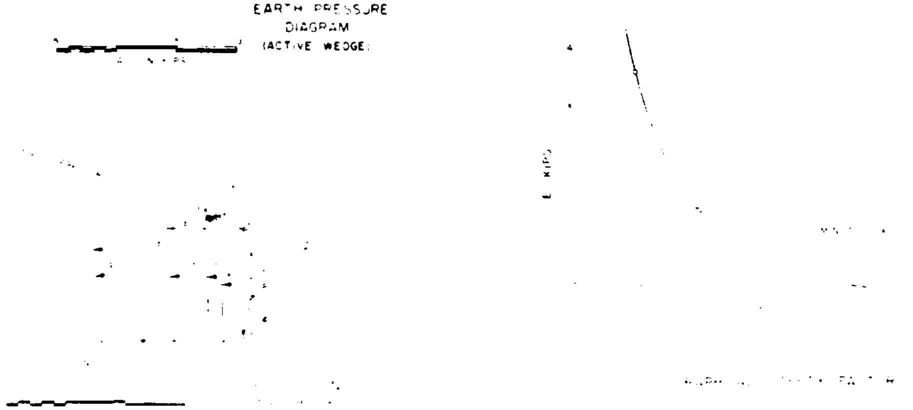
NOTES

1. ONLY Q STRENGTHS WERE USED FOR STRENGTH CALCULATIONS
2. θ_0 ACTS PARALLEL TO SLOPE OF CUT

$$\frac{\phi}{\text{TRIAL S.F.}} = \phi_D$$

TRIAL S.F.	ϕ_D-SC	ϕ_D SC+
1.0	9.0°	4.0°
2.0	4.5°	2.0°
4.0	2.3°	0°
7.0	1.3°	0.6°

EARTH PRESSURE DIAGRAM (ACTIVE WEDGE)



CRITICAL ACTIVE WEDGE TABLE OF COMPUTATIONS	NORMAL LOADING	EARTHQUAKE LOADING
WEIGHT-KIPS	173	SEE DESIGN DATA NO. 9
C_{SC}-KIPS/SQFT	1.83	"
C_{SC+} "	2.34	"
C_{RB} "	5.16	"
ϕ-DEGREES	SC-9°, SC+-4°	"
θ_0-RADIANS	0.611	"
E-KIPS	6.2	"
SAFETY FACTOR	4.65	"

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN S.C.

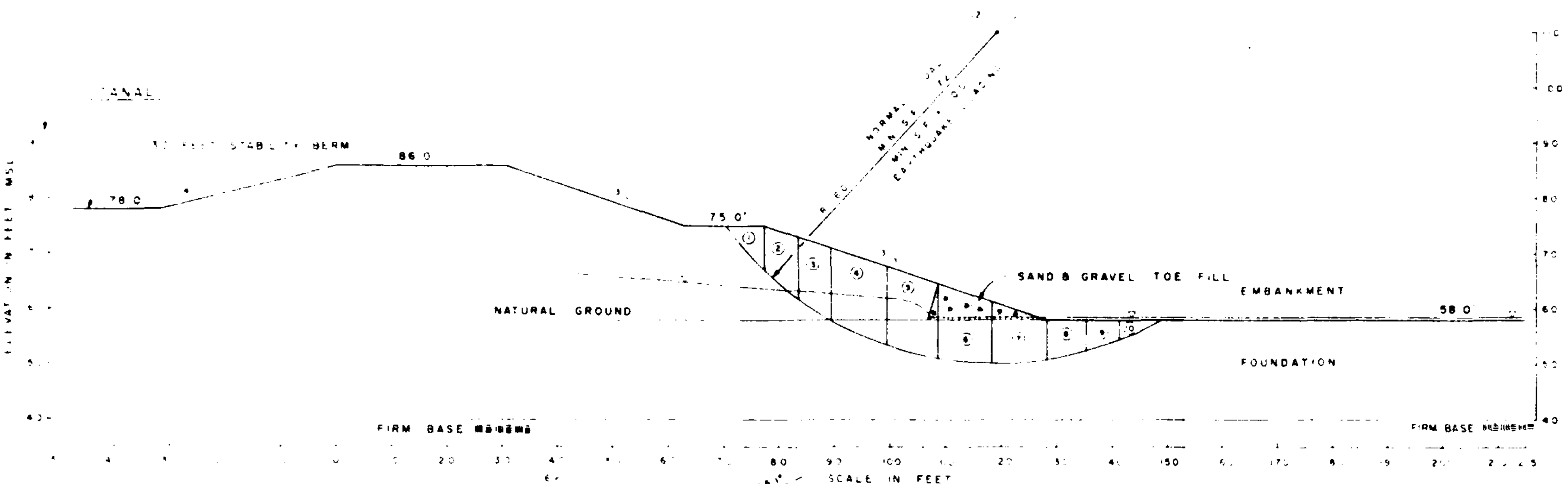
SLOPE STABILITY ANALYSIS

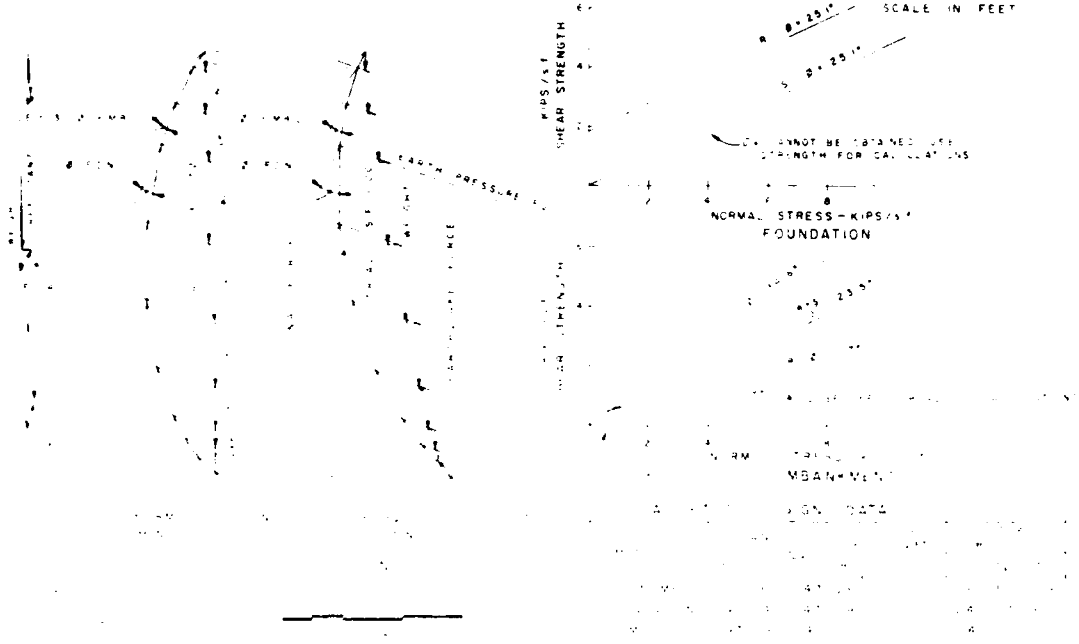
INTAKE CANAL

C/L STATION 272 + 75

CASE END OF CONSTRUCTION
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 75

PLATE D 23





DESIGN DATA

1. CASE STEADY SEEPAGE
2. METHOD FINITE SLICE (SLIP CIRCLE)
3. SLOPE 1 ON 3H
4. HEIGHT = 28'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. NORMAL CANAL SURFACE ELEV = 75.0' MSL
7. NORMAL GWT ELEV = 58' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION (a = 0.15 g)

NOTES

REFER TO GRAPHS FOR STRENGTH SELECTION
... PARALLEL TO SLOPE

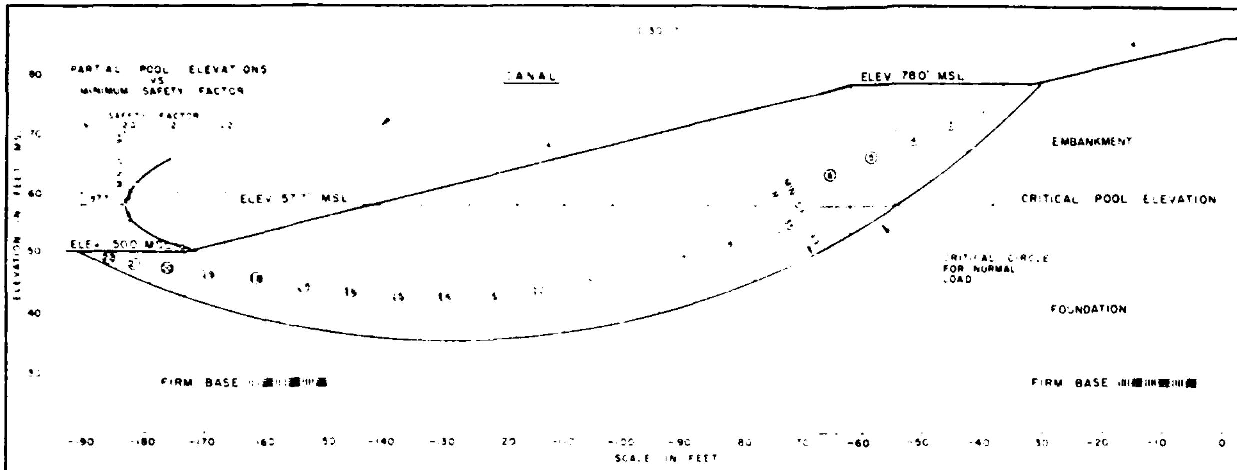
COOPER RIVER REDIVERSION PROJECT ST. STEPHEN S.C.

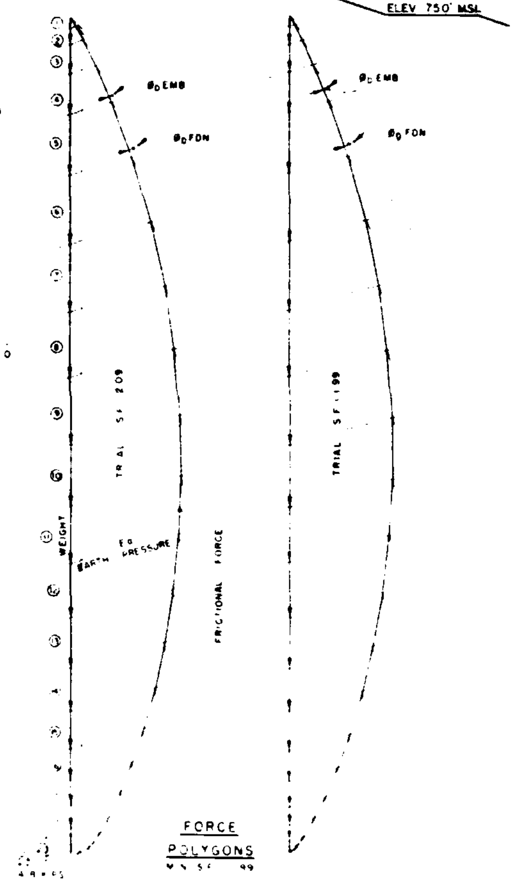
SLOPE STABILITY ANALYSIS

INTAKE CANAL

C.L STATION 335 + 00

CASE STEADY SEEPAGE
SLOPE 1 ON 3H
METHOD SLIP CIRCLE
DATE APRIL





DESIGN DATA

1. CASE PARTIAL POOL
2. METHOD FINITE SLICE (SLIP CIRCLE)
3. SLOPE 1 ON 4H
4. HEIGHT = 36'
5. BOTTOM OF CANAL ELEV = 50.0' MSL
6. CRITICAL POOL ELEV = 57.7' MSL
7. NORMAL GWT ELEV = 58.0' MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION (SEE PLATE D-26 FOR EARTHQUAKE FORCE CALCULATIONS)

ϕ EMB = 23.5°
ϕ FDN = 25.1°
$\phi_D = \frac{\phi}{\text{TRIAL SF}}$

NOTES

REFER TO GRAPHS FOR STRENGTH [illegible] TION IN PLATE D-24

[illegible] ACTS PARALLEL TO SLOPE

ADOPTED DESIGN DATA

MATERIAL	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
EMBANKMENT	[illegible]	[illegible]	[illegible]	[illegible]	0.6	[illegible]	[illegible]	[illegible]	[illegible]
FOUNDATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4	[illegible]	65
FIRM BASE	[illegible]	[illegible]	47	25	[illegible]	[illegible]	4	[illegible]	65

COOPER RIVER REDIVERSION PROJECT ST STEPHEN S C

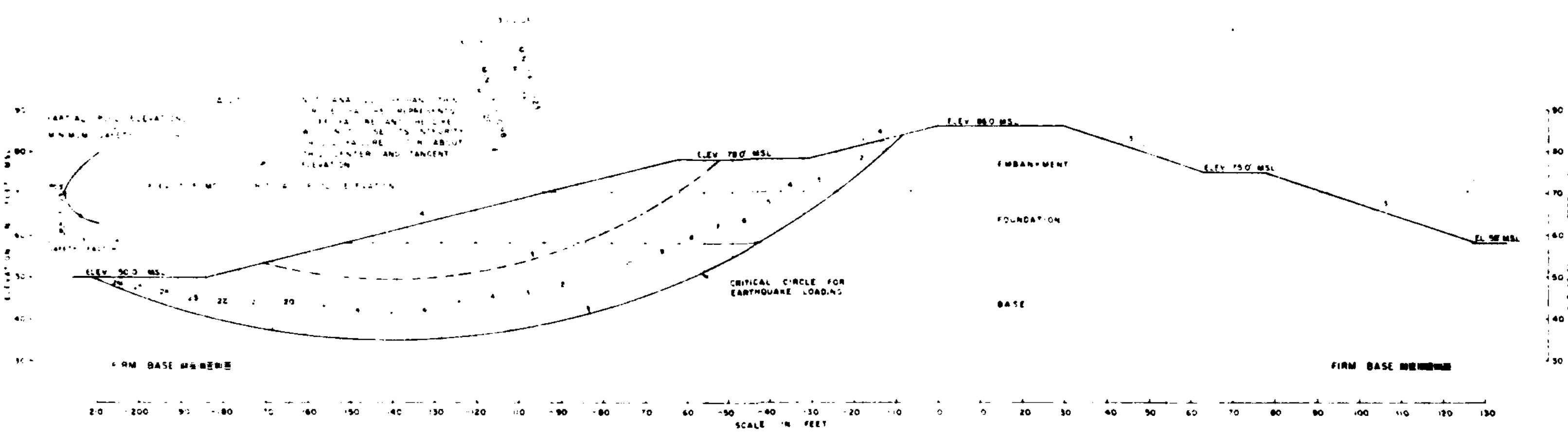
SLOPE STABILITY ANALYSIS

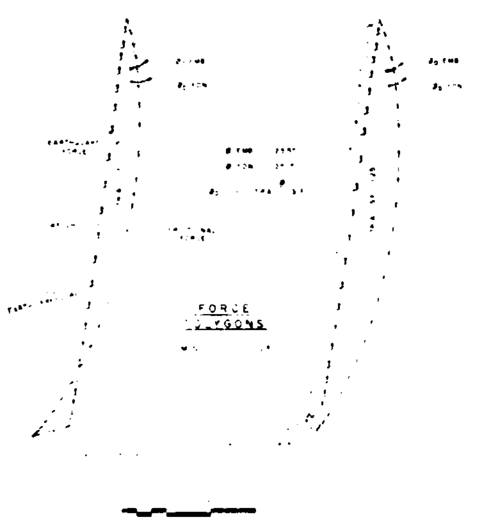
INTAKE CANAL

C/L STATION 335 + 00

CASE PARTIAL POOL
SLOPE 1 ON 4H
METHOD SLIP CIRCLE
DATE APRIL 76

PLATE [illegible] 25





DESIGN DATA

1. CASE PARTIAL POOL
2. METHOD FINITE SLICE (SLIP CIRCLE)
3. SLOPE 1 ON 4H
4. HEIGHT 36'
5. BOTTOM OF CANAL ELEV 50' MSL
6. CRITICAL POOL ELEV 70.3 MSL
7. NORMAL GWT ELEV 70.3 MSL
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION

NOTES

1. REFER TO GRAPHS FOR STRENGTH SELECTION ON PLATE D 24
2. [illegible] PARALLEL TO SLOPE

ADOPTED DESIGN DATA									
MATERIAL	TAN Ø			COHESION KIPS/SQ FT			UNIT WT		
	Q	R	S	Q	R	S	γ_m	γ_b	γ_s
EMBANKMENT	[illegible]	0.21	0.69	0	0.6	[illegible]	126	[illegible]	69
FOUNDATION	0.37	0.21	0.69	0	[illegible]	[illegible]	126	[illegible]	69
BASE	0	0.47	0.47	0.5	0	[illegible]	126	[illegible]	[illegible]
FIRM BASE	0	0.47	0.47	0.5	0	0	126	[illegible]	[illegible]

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

SLOPE STABILITY ANALYSIS

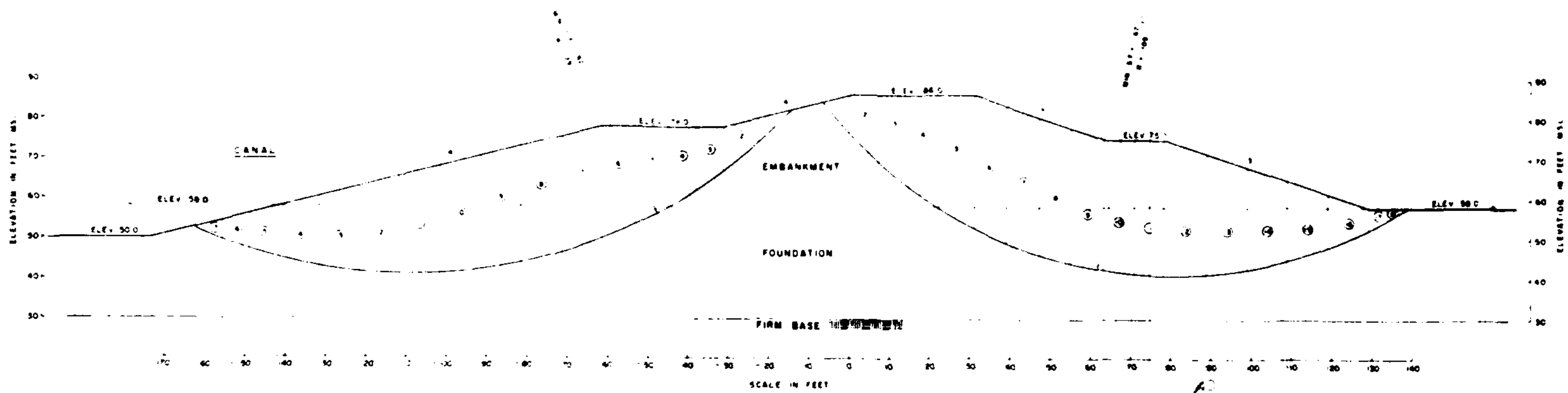
INTAKE CANAL

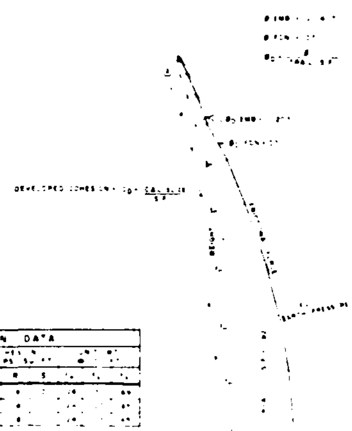
C/L STATION 335 + 00

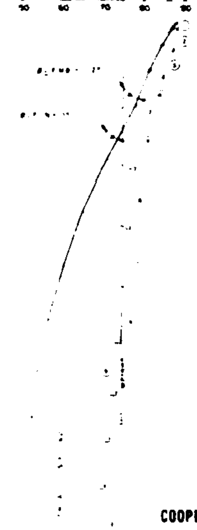
CASE: PARTIAL POOL
SLOPE: 1 ON 4H
METHOD: SLIP CIRCLE
DATE: APRIL 76

PLATE D 26

D







DESIGN DATA

1. CASE END OF CONSTRUCTION
2. METHOD FINITE SLICE (SLIP CIRCLE)
3. SLOPE 1 ON 4 UPSTREAM & 1 ON 3 DOWNSTREAM
4. HEIGHT 28'
5. BOTTOM OF CANAL ELEV 50.0' MSL
6. ORIGINAL GROUND ELEV 58.0' MSL
7. NORMAL GWT ELEV 58.0' MSL
8. ASSUME NO TENSION CRACKS

ADOPTED DESIGN DATA

MATERIAL	TAN Ø			COHESION KIPS SQ FT			UNIT WT		
	Q	R	S	Q	R	S			
EMBANKMENT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FOUNDATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FIRM BASE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

ADOPTED DESIGN DATA

MATERIAL	TAN Ø			COHESION KIPS SQ FT			UNIT WT		
	Q	R	S	Q	R	S			
EMBANKMENT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FOUNDATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FIRM BASE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

NOTES

1. ONLY Q STRENGTHS WERE USED FOR STRENGTH CALCULATIONS
2. E[illegible] PARALLEL TO SLOPE
3. N[illegible] EARTHQUAKE FORCES COMPUTED [illegible] TERM CONDITION

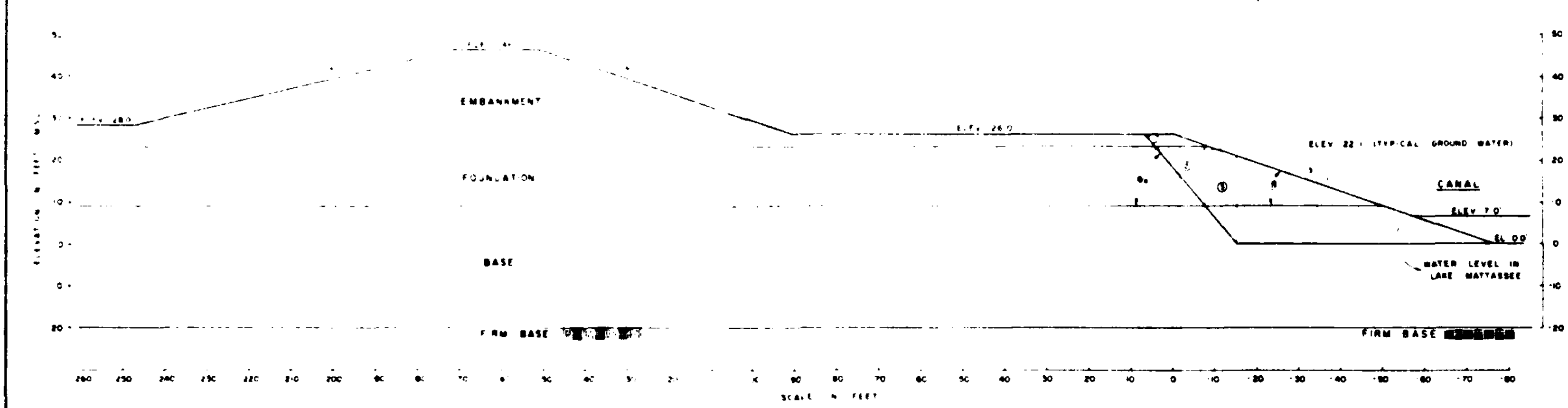
COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

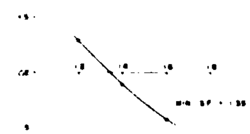
SLOPE STABILITY ANALYSIS

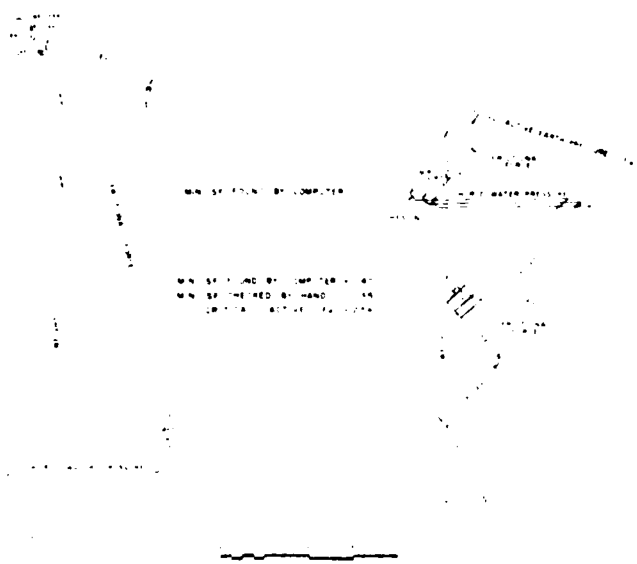
INTAKE CANAL

C/L STATION 335 + 00

CASE END OF CONSTRUCTION
SLOPE 1 ON 4H(US), 1 ON 3H(DS)
METHOD SLIP CIRCLE
DATE APRIL 76 PLATE D 27







GRAPHICAL SAFETY FACTOR

ADOPTED DESIGN DATA									
MATERIAL	TAN Ø			COHESION KIPS/SQ FT			UNIT WT K/CU FT		
	Q	R	S	Q	R	S	δ_m	δ_s	δ_b
EMBANKMENT	[illegible]	[illegible]	[illegible]		0.2	0	.120	.120	.130
FOUNDATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	[illegible]	[illegible]	.55
BASE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	[illegible]	.126	.62
FIRM BASE	[illegible]	[illegible]	[illegible]	0	0	[illegible]	[illegible]	.126	.62

DESIGN DATA

1. CASE END OF CONSTRUCTION
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT 26'
5. BOTTOM OF CANAL ELEV 0.0' MSL
6. ORIGINAL GROUND ELEV 23' MSL
7. NORMAL GWT ELEV 22' MSL
8. ASSUME NO TENSION CRACKS

NOTES

ONLY Q STRENGTHS WERE USED FOR STRENGTH CALCULATIONS

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

SLOPE STABILITY ANALYSIS

TAILRACE CANAL

C/L STATION 500 + 00

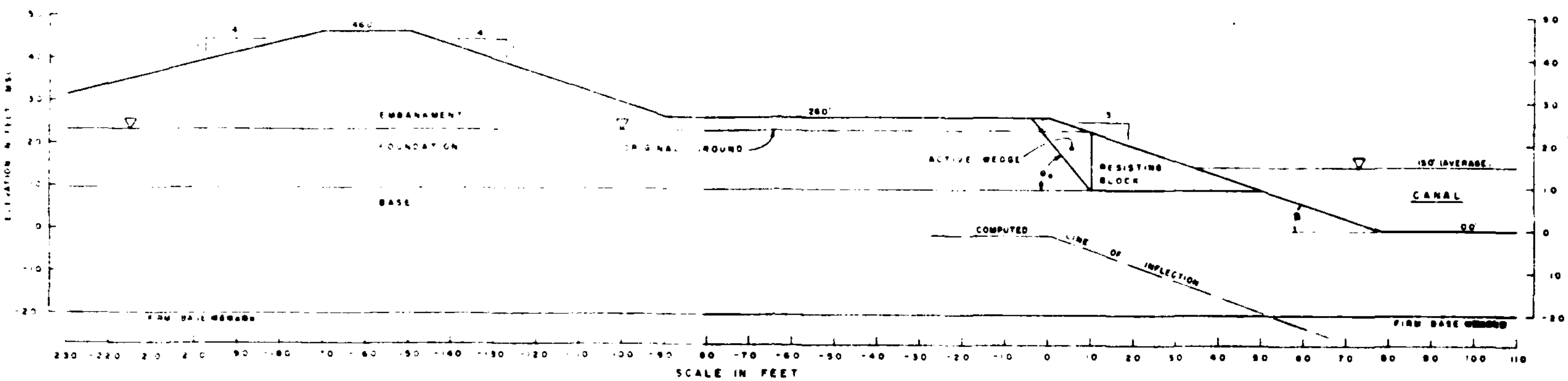
CASE: END OF CONSTRUCTION

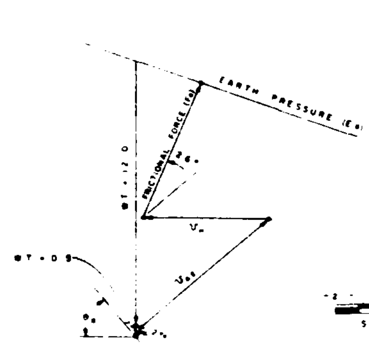
SLOPE: 1 ON 3H

METHOD: WEDGE

DATE APRIL 76

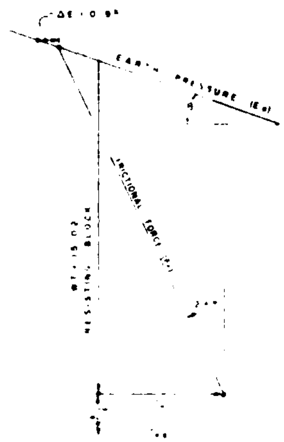
PLATE D 28





EARTH PRESSURE DIAGRAM

ACTIVE WEDGE



FORCE POLYGON

DESIGN DATA

1. CASE: STEADY SEEPAGE
2. METHOD WEDGE
3. SLOPE 1 ON 3H
4. HEIGHT = 26'
5. BOTTOM OF CANAL ELEV = 0.0' MSL
6. NORMAL CANAL SURFACE ELEV = 23.0'–7.0' MSL
7. NORMAL GWT ELEV = 23.0' MSL
8. ASSUME NO TENSION CRACKS

NOTES

1. NO EARTHQUAKE FORCES COMPUTED FOR STEADY SEEPAGE CASE DUE TO LOW PROBABILITY. SEE SUDDEN DRAWDOWN CASE PLATE NO D-30 OF FAILURE DURING EARTHQUAKE
2. Ea ACTS PARALLEL TO SLOPE OF CUT
3. ONLY "S" STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLATE NO D-33)

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

SLOPE STABILITY ANALYSIS

TAILRACE CANAL

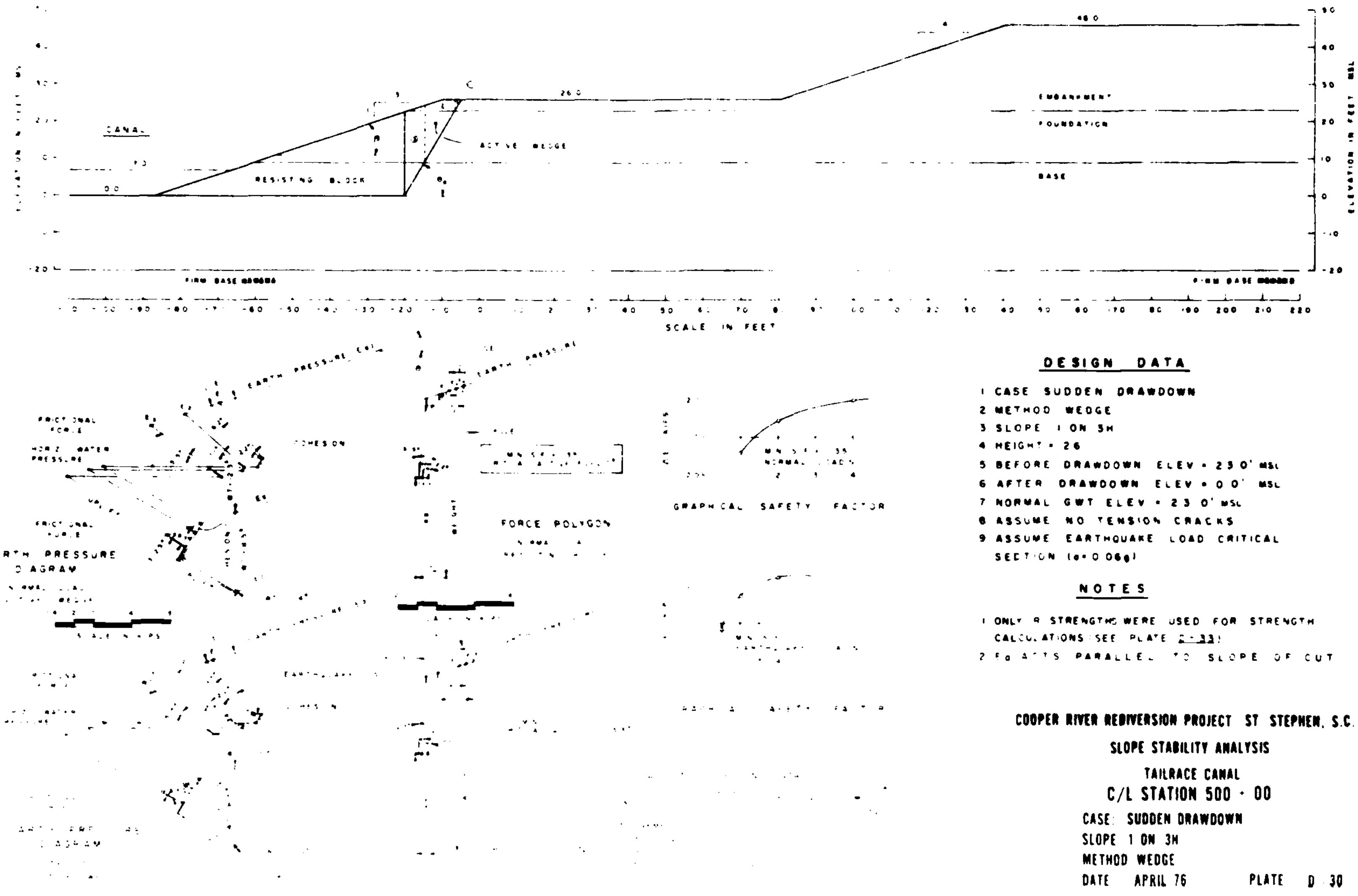
C/L STATION 500 + 00

CASE: STEADY SEEPAGE

SLOPE: 1 ON 3H

METHOD WEDGE

DATE APRIL 76 PLATE: D-29



DESIGN DATA

1 CASE SUDDEN DRAWDOWN
2 METHOD WEDGE
3 SLOPE 1 ON 3H
4 HEIGHT = 26
5 BEFORE DRAWDOWN ELEV = 23.0' MSL
6 AFTER DRAWDOWN ELEV = 0.0' MSL
7 NORMAL GWT ELEV = 23.0' MSL
8 ASSUME NO TENSION CRACKS
9 ASSUME EARTHQUAKE LOAD CRITICAL SECTION (a=0.06g)

NOTES

1 ONLY R STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLATE D-33)
2 Ea ACTS PARALLEL TO SLOPE OF CUT

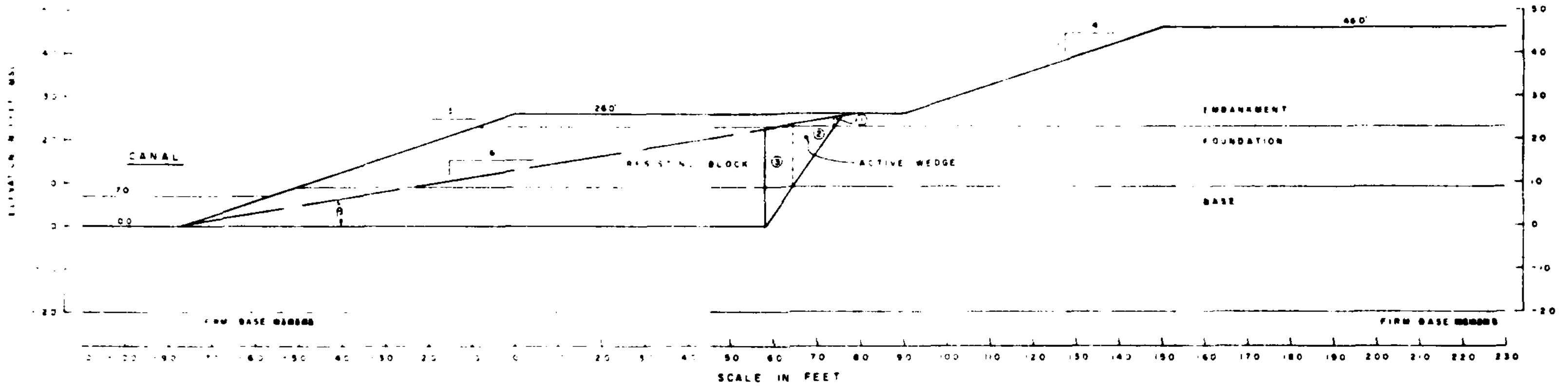
COOPER RIVER REDIVERSION PROJECT ST STEPHEN, S.C.

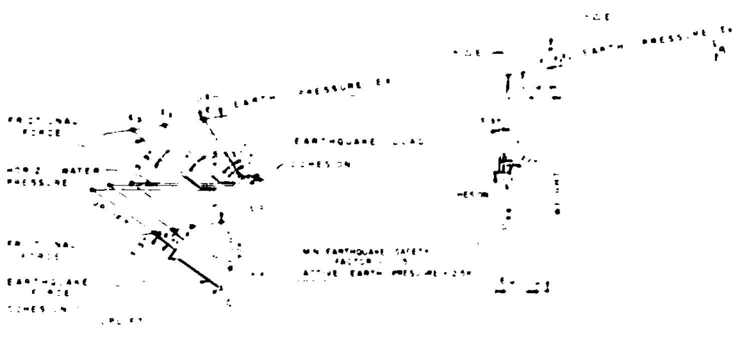
SLOPE STABILITY ANALYSIS

TAILRACE CANAL

C/L STATION 500 + 00

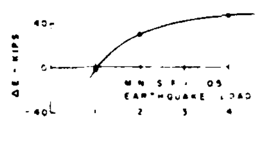
CASE: SUDDEN DRAWDOWN
SLOPE 1 ON 3H
METHOD WEDGE
DATE APRIL 76 PLATE D 30





EARTH PRESSURE DIAGRAM

FORCE POLYGON



GRAPHICAL SAFETY FACTOR

ADOPTED DESIGN DATA

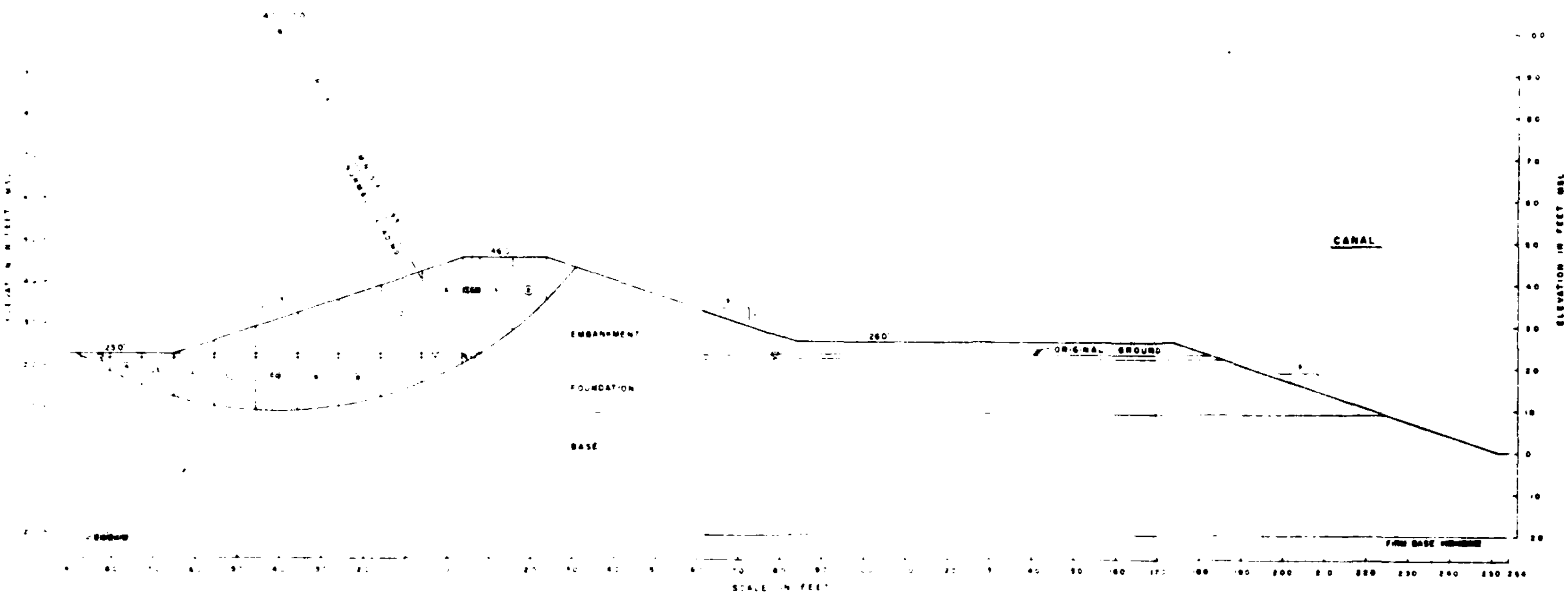
DESIGN DATA

1 CASE SUDDEN DRAWDOWN
2 METHOD WEDGE
3 SLOPE 1 ON 6H
4 HEIGHT = 26'
5 BEFORE DRAWDOWN ELEV = 23.0' MSL
6 AFTER DRAWDOWN ELEV = 7.0' MSL
7 NORMAL GWT ELEV = 23.0' MSL
8 ASSUME NO TENSION CRACKS
9 ASSUME EARTHQUAKE LOAD FOR CRITICAL SECTION (0 = 0.15g)

NOTES

1 SEE PLOTS FOR STRENGTH SELECTION ON PLATE D-33
2 Eo ACTS PARALLEL TO SLOPE

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C
SLOPE STABILITY ANALYSIS
TAILRACE CANAL
C/L STATION 500 + 00
CASE SUDDEN DRAWDOWN
SLOPE 1 ON 6H
METHOD WEDGE
DATE APRIL 76 PLATE D 31



DESIGN DATA

1 CASE END OF CONSTRUCTION
2 METHOD FINITE SLICE (SLIP CIRCLE)
3 SLOPE 1 ON 3H
4 HEIGHT = 23'
5 BOTTOM OF CANAL ELEV = 0.0' MSL
6 NORMAL CANAL SURFACE ELEV = 7.0' – 22.0' MSL
7 NORMAL GWT ELEV = 23.0' MSL
8 ASSUME NO TENSION CRACKS

NOTES

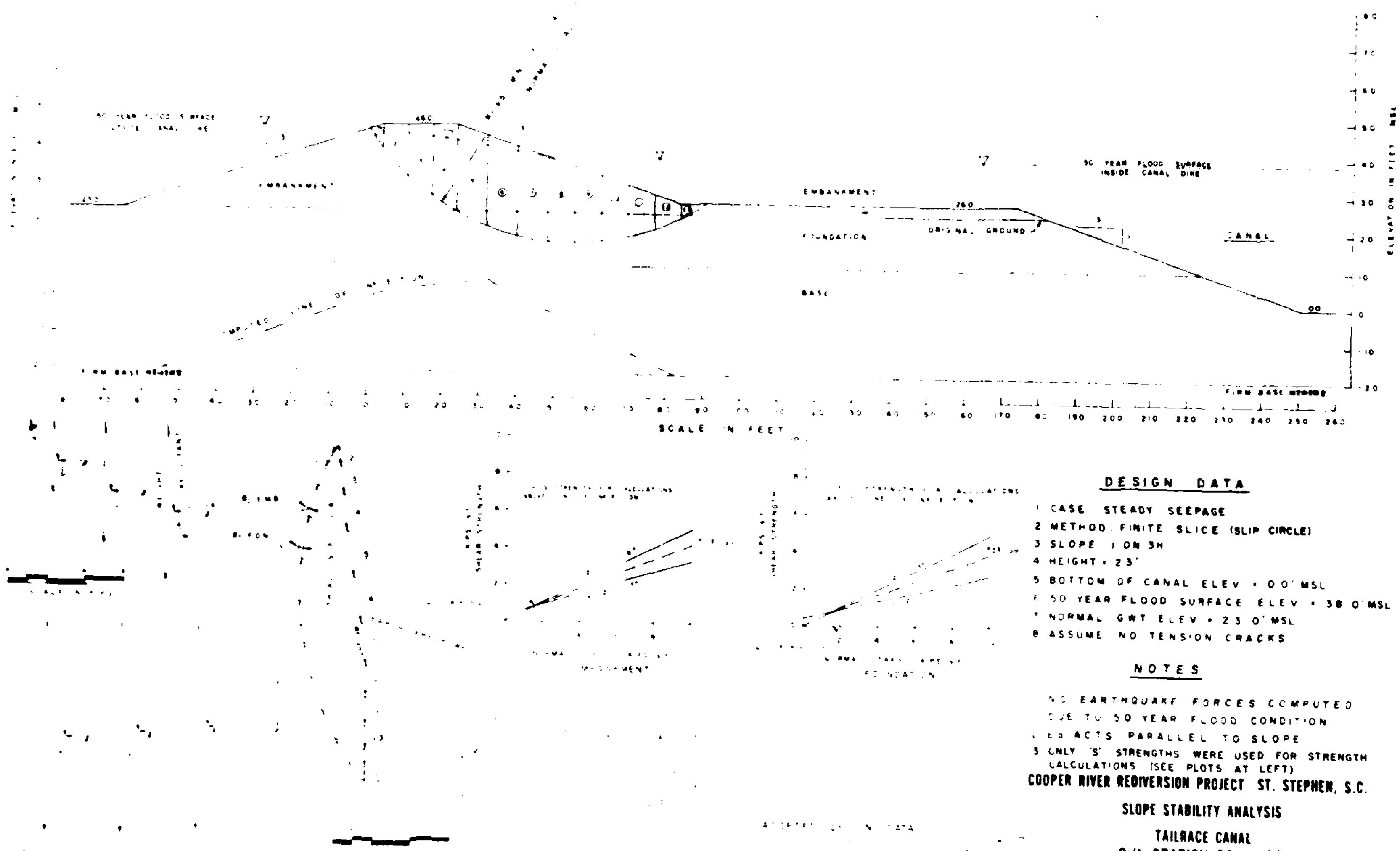
COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

SLOPE STABILITY ANALYSIS

TAILRACE CANAL

C/L STATION 500 + 00

CASE END OF CONSTRUCTION
SLOPE 1 ON 3H
METHOD SLIP CIRCLE
DATE APRIL 76 PLATE D-32



DESIGN DATA

1 CASE STEADY SEEPAGE
2 METHOD FINITE SLICE (SLIP CIRCLE)
3 SLOPE 1 ON 3H
4 HEIGHT = 23'
5 BOTTOM OF CANAL ELEV = 0.0' MSL
6 50 YEAR FLOOD SURFACE ELEV = 38.0' MSL
7 NORMAL GWT ELEV = 23.0' MSL
8 ASSUME NO TENSION CRACKS

NOTES

NO EARTHQUAKE FORCES COMPUTED DUE TO 50 YEAR FLOOD CONDITION
... ACTS PARALLEL TO SLOPE
3 ONLY 'S' STRENGTHS WERE USED FOR STRENGTH CALCULATIONS (SEE PLOTS AT LEFT)

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

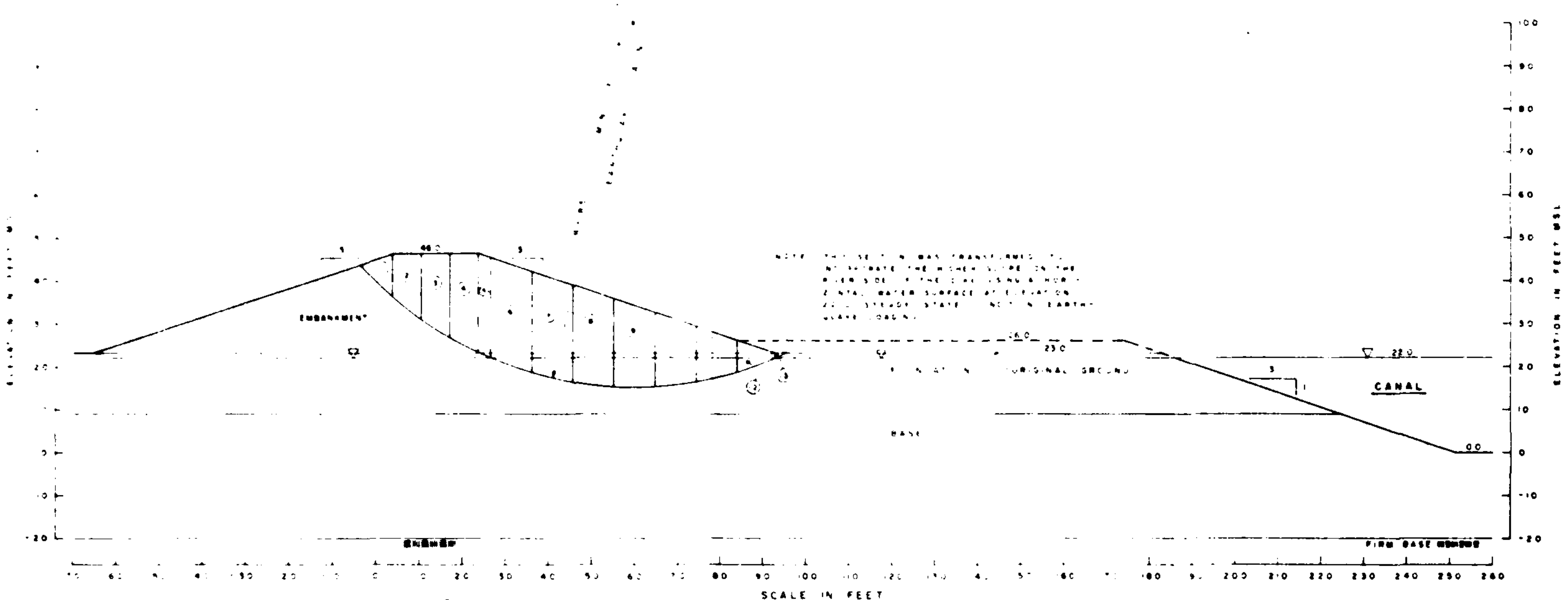
SLOPE STABILITY ANALYSIS

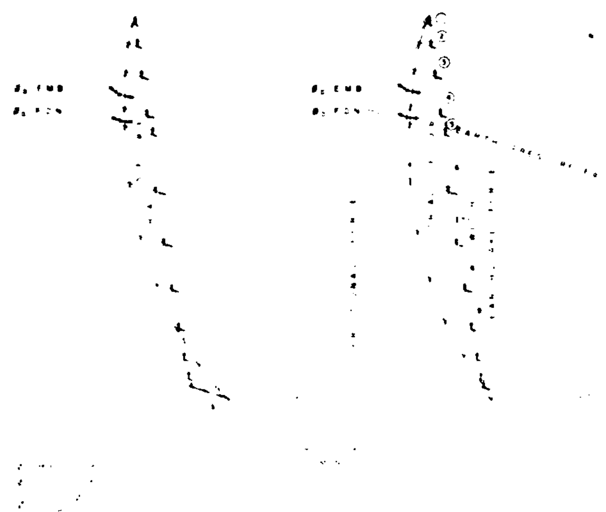
TAILRACE CANAL

C/L STATION 500 + 00

CASE STEADY SEEPAGE (50 YEAR FLOOD)
SLOPE 1 ON 3H
METHOD SLIP CIRCLE
DATE APRIL 76

PLATE D 33





DESIGN DATA

1. CASE: STEADY SEEPAGE
2. METHOD: FINITE SLICE (SLIP CIRCLE)
3. SLOPE: 1 ON 3H
4. HEIGHT = 26.0
5. BOTTOM OF CANAL ELEV = 0.0' MSL
6. NORMAL CANAL SURFACE ELEV = 22.0'
7. NORMAL GWT ELEV = 22.0'
8. ASSUME NO TENSION CRACKS
9. ASSUME EARTHQUAKE LOAD ON CRITICAL SECTION (a = 0.15g)

ADOPTED DESIGN DATA

NOTES

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.

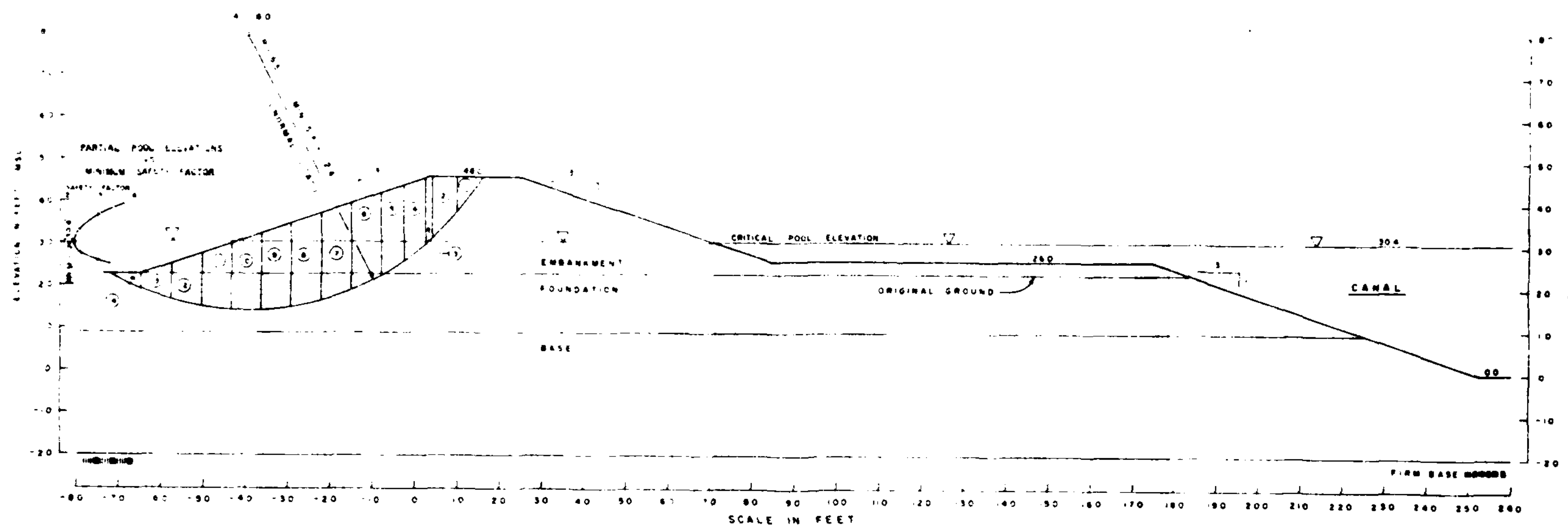
SLOPE STABILITY ANALYSIS

TAILRACE CANAL

C/L STATION 500 + 00

CASE: STEADY SEEPAGE
SLOPE: 1 ON 3H
METHOD: SLIP CIRCLE
DATE: APRIL 76

PLATE D 34



ADOPTED DESIGN DATA									
MATERIAL	TAN Ø			COHESION KIPS/SQ FT			UNIT WT		
	R	S	Q	R	S	Q			
EMBANKMENT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
FOUNDATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
BASE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26	[illegible]
FIRM BASE	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26	63

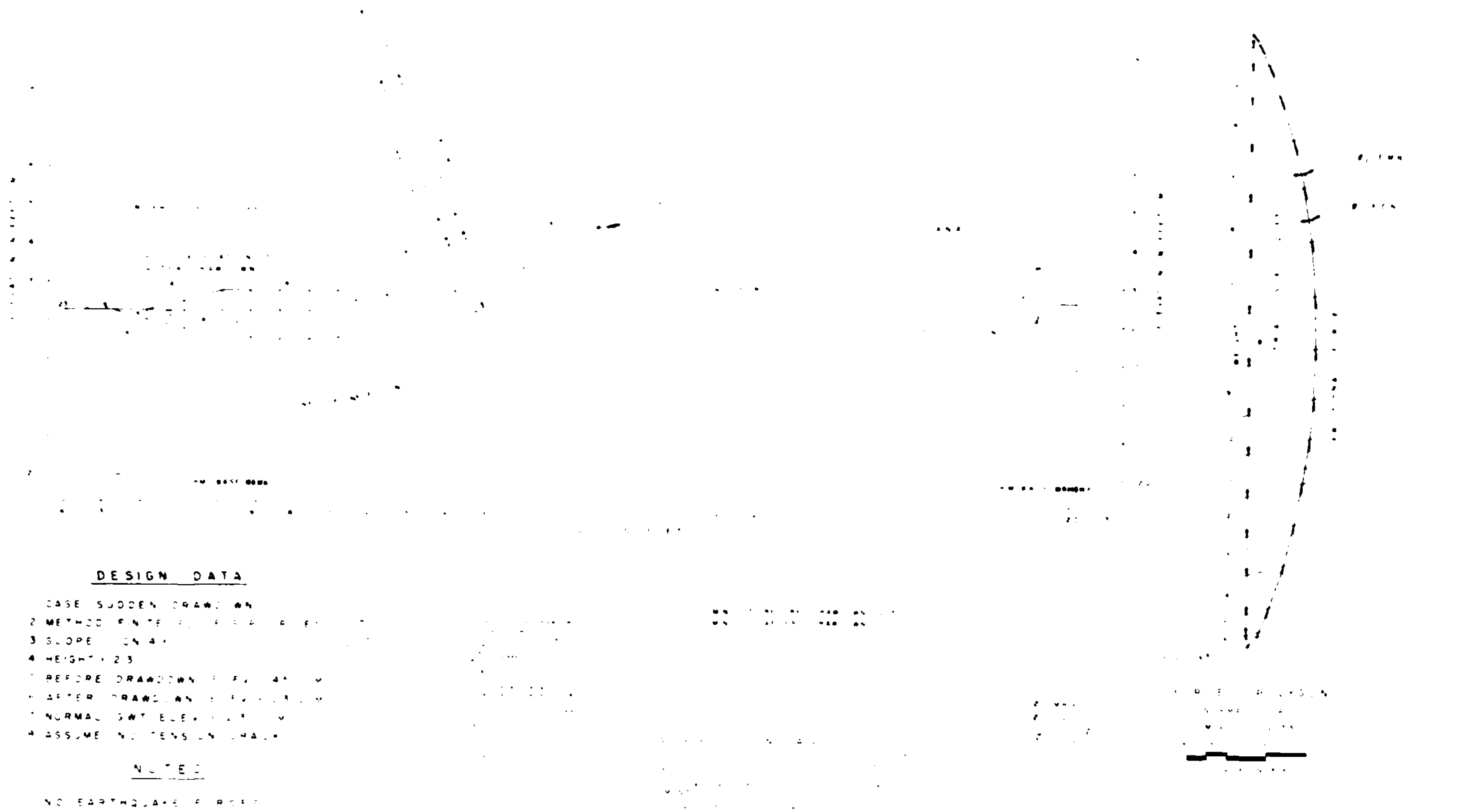
DESIGN DATA

1 CASE PARTIAL POOL
2 METHOD FINITE SLICE (SLIP CIRCLE)
3 SLOPE 1 ON 3H
4 HEIGHT = 26
5 PARTIAL POOL—CRITICAL WATER LEVEL = 30.4' MSL
6 NORMAL GWT ELEV = 23.0' MSL
7 ASSUME NO TENSION CRACKS
8 ASSUME NO COHESION

NOTES

[illegible]

COOPER RIVER REDIVERSION PROJECT ST. STEPHEN, S.C.
SLOPE STABILITY ANALYSIS
TAILRACE CANAL
C/L STATION 500 + 00
CASE PARTIAL POOL
SLOPE 1 ON 3H
METHOD SLIP CIRCLE
DATE APRIL 76 PLATE D 35



DESIGN DATA

CASE SUDDEN DRAWDOWN
2 METHOD FINITE
3 SLOPE 1 ON 4H
4 HEIGHT 23
BEFORE DRAWDOWN
AFTER DRAWDOWN
NORMAL SWT ELEV
ASSUME NO TENSION CRACK

NOTES

NO EARTHQUAKE FORCE
COMPUTED
TERM CONDITION

COOPER RIVER REDIVERSION PROJECT ST STEPHEN, S.C

SLOPE STABILITY ANALYSIS

TAILRACE CANAL

C/L STATION 500 00

CASE SUDDEN DRAWDOWN

SLOPE 1 ON 4H

METHOD SLIP CIRCLE

DATE APRIL 76 PLATE D 36

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX E

STRUCTURAL DESIGN COMPUTATIONS

U. S. ARMY ENGINEER DISTRICT, CHARLESTON

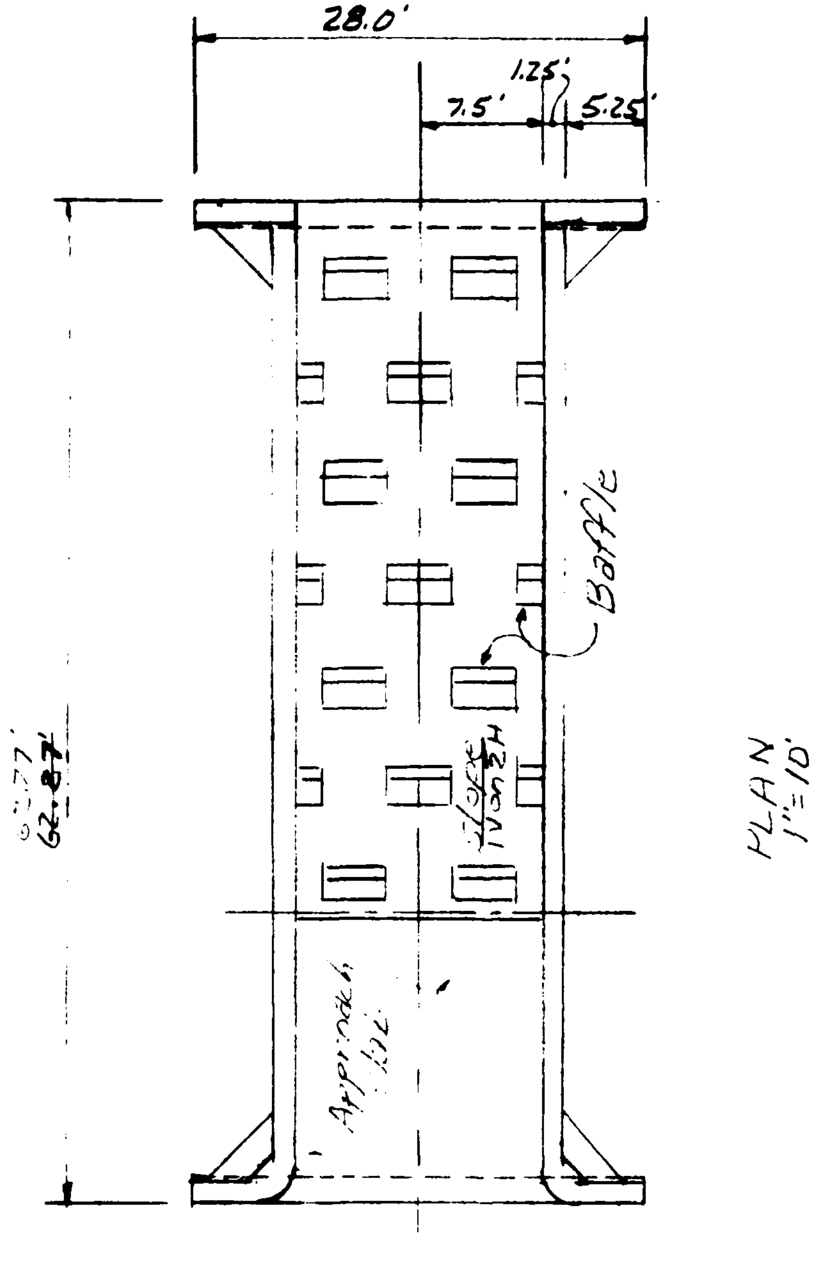
CORPS OF ENGINEERS

CHARLESTON, SOUTH CAROLINA

BY A M K DATE Mar 76 SUBJECT Cooper R. Rediversion Canals DM-Drop Str. below P.H. Access Rd

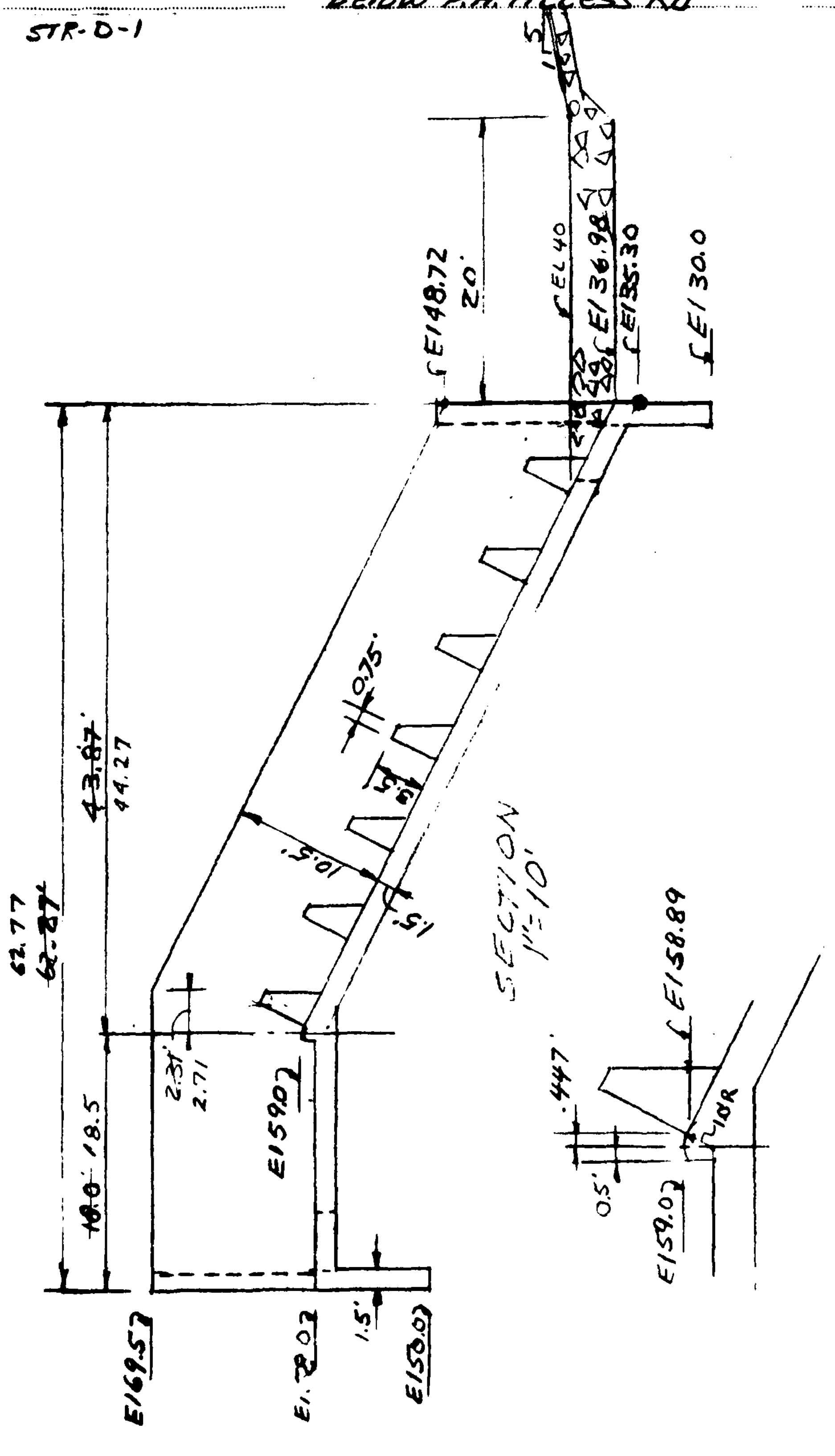
SHEET NO. 1 OF

CHKD. BY DATE JOB NO.



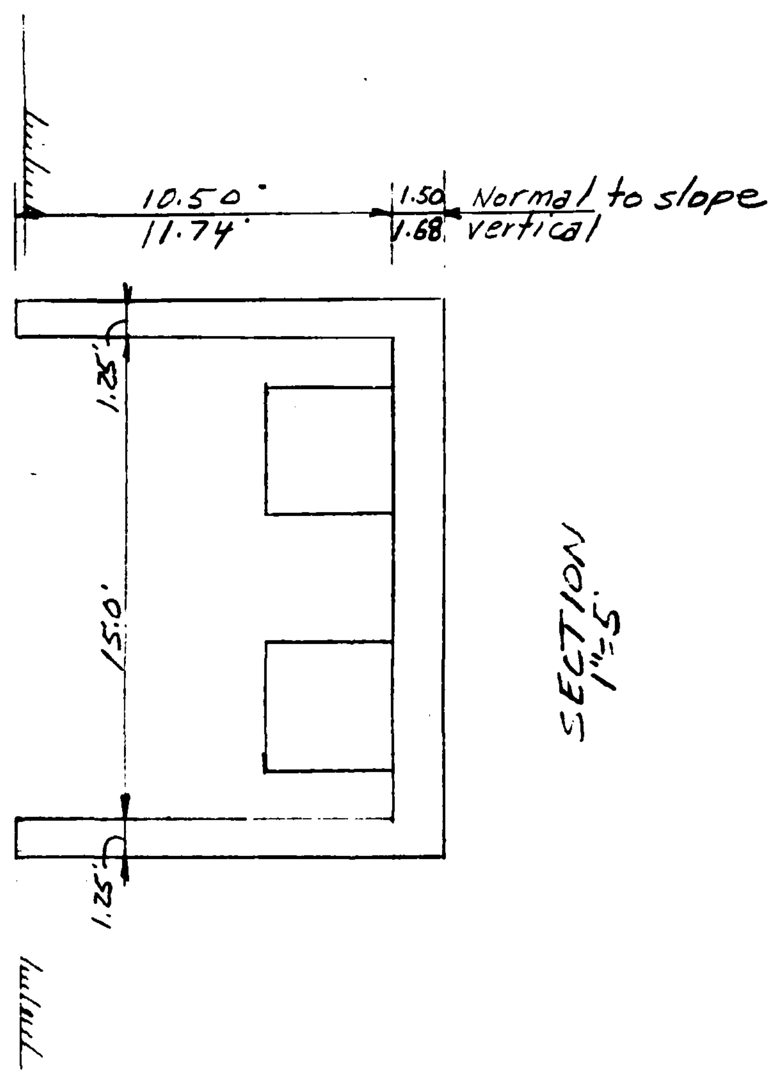
BY AMK DATE Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 2 OF
CHKD. BY DATE Canals DM Drop str JOB NO.
Below P.H. Access Rd

STR-D-1



E-2

BY AMK DATE Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 3 OF
CHKD. BY DATE Canals DM. Drop Str JOB NO.
below P.H. Access Rd



BY A M K DATE Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 4 OF
CHKD. BY DATE Canals DM Drop str JOB NO.
below PH Access Rd

Concrete	V	Arm	Mom
Apron slab: (18.0)(1.5)(17.5)(.15)	70.9 ᵏ	53.27	3776
Upper key: 6.5(1.5)(28.0)(.15)	41.0	61.52	2519
Apron walls 2(1.25)(18.0)(11.5)(.15)	77.6	53.27	4135
Upper wings 2(1.25)(13.0)(5.25)(.15)	25.6	61.65	1578
Chute slab 17.5(1.5)(49.5±)(.15)	194.9	22.14	4315
Lower key 5.5±(1.5)(28.0)(.15)	34.6	0.75	26
Chute walls 2(11.74)(1.25)(44.27)(.15)	194.9	22.14	4315
Lower wings 2(1.25)(13.1±)(5.25)(.15)	25.8	0.62	16
Baffles: 5.7□' in x-sect. net width = 15/2, 7 rows 7(5.7)(7.5)(.15)	44.9	23.55±	1057
Total Conc.	710.2 ᵏ	30.61'	21737

Case I - Construction Complete with no hydrostatic forces acting

$e = 62.27/2 - 30.61 = 0.52'$ dnstr

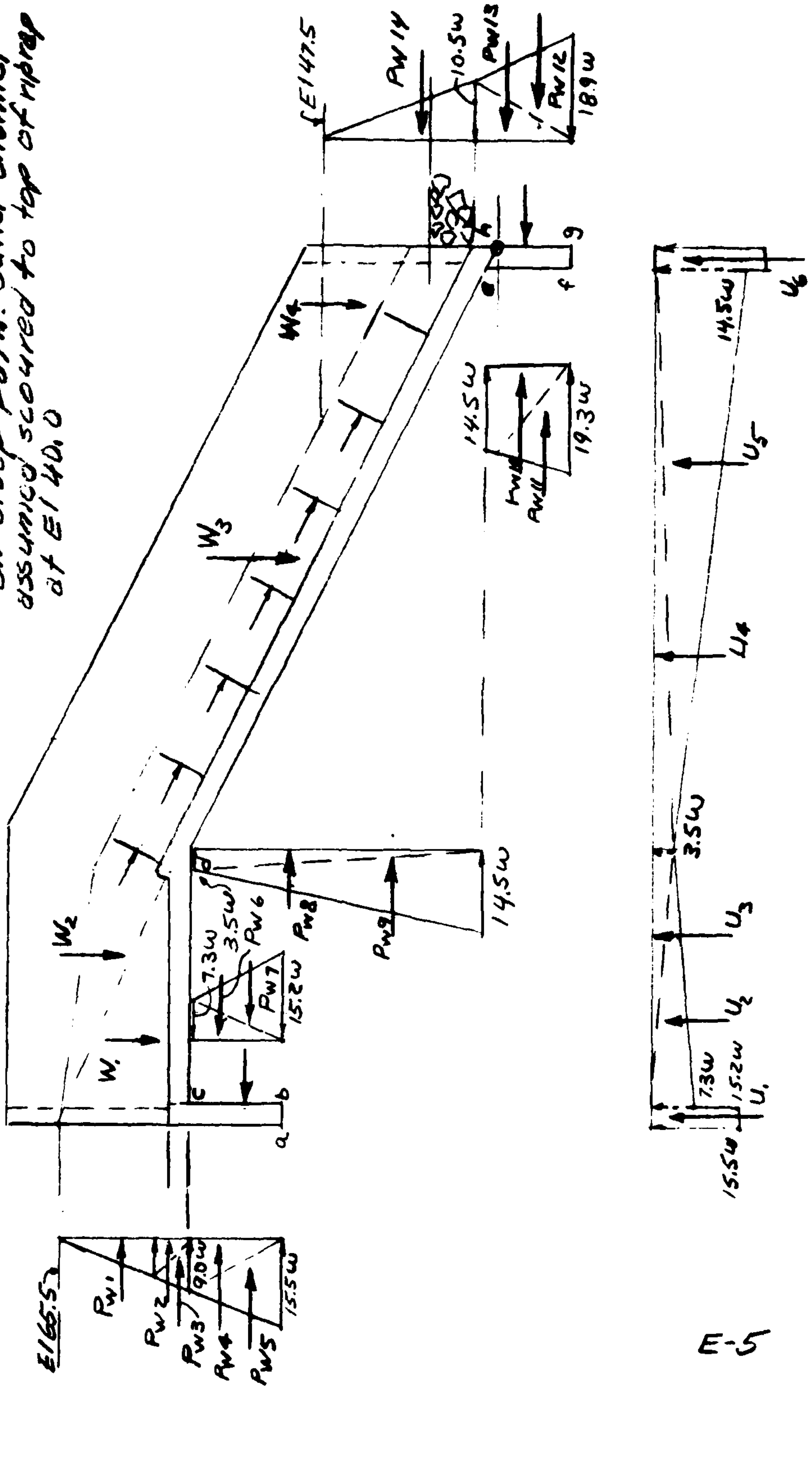
Fdn press: upstr $= \frac{710.2}{(17.5)(62.27)}\left(1 - \frac{6(0.52)}{62.27}\right)$

" $= 0.652(1 - 0.050) = \underline{0.62}$ k/SF

dnstr $= 0.652(1 + 0.050) = \underline{0.68}$ k/SF

BY A.M.K. DATE 11/81/76 SUBJECT Copper R Rediversion SHEET NO. 5 OF
CHKD. BY DATE Canals D.M. Drop Str below PH Access Rd JOB NO.

Case II - Operating Condition
50 yr Flood - Headwater El 165.5
Tailwater El 147.5 Uplift based on creep path. Outlet channel assumed scoured to top of riprap at El 140.0



BY AMK DATE Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 6 OF
CHKD. BY DATE Canals Dm Drop Str JOB NO.
below PH Access Rd

Uplift Pressures and Horiz water forces - Case II

HW = El 65.5

TW El 47.5

Net hd = 18.0'

Creep path pt "a" to "h"

$\overset{a}{}\ \overset{b}{1.5} + \overset{c}{6.5} + \overset{d}{18.5^{\pm}} + \overset{e}{46.0^{\pm}} + \overset{f}{6.0^{\pm}} + \overset{g}{1.5} + \overset{h}{7.0} = 87'$

Creep ratio $= \frac{87}{18} = 4.83$

seepage hd loss $= \frac{18}{87} = 0.207$ '/'

Pt	Creep Distance	Hd Loss	Gradient El	Pt El	Uplift hd
a	0	0	65.5	50.0	15.5'
b	1.5	.31	65.2	50.0	15.2'
c	8.0	1.66	63.8	56.5	7.3'
d	26.5	5.48	60.0	56.5	3.5'
e	72.5	15.0	50.5	36.0	14.5'
f	78.5	16.2	49.3	30.0	19.3'
g	80.0	16.6	48.9	30.0	18.9'
h	87.0	18.0	47.5	37.0	10.5'

BY HMK DATE Mar 16 SUBJECT Cooper R. Rediversion Canals Ent.-Drop str. below FH Access Rd SHEET NO. 7 OF
CHKD. BY DATE JOB NO.

Horizontal Water forces - Case II

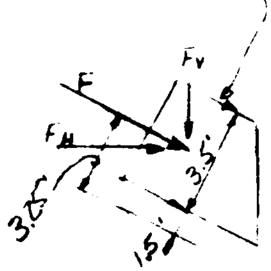
	H	Arm	$\Sigma M_{EL\,353}$
P_{W1} (on walls only)			
$.0625(6.5)^2/2(2)(1.25)$	3.3^{K} →	25.87	85 ↻
P_{W2} $.0625(6.5)(2.5/2)(17.5)$	8.9 →	22.87	203 ↻
P_{W3} $.0625(9.0)(2.5/2)(17.5)$	12.3 →	22.03	271 ↻
Assume ($P_{W4}+P_{W5}$) balances ($P_{W6}+P_{W7}$)			
P_{W8} $.0625(3.5)20.5/2(17.5)$	39.2 →	14.37	564 ↻
P_{W9} $.0625(14.5)(20.5/2)(17.5)$	162.6 →	7.53	1224 ↻
Assume ($P_{W10}+P_{W11}$) balances ($P_{W12}+P_{W13}$)			
P_{W14} Assume 60% effective $.0625(10.5)10.5/2(17.5)(.60)$	36.2 ←	5.2	188 ↺
ΣH_W	(190.1)		(2159 ↻)

Uplift - Case II

	V	Arm	ΣM
U_1 $.0625(15.4)(1.5)(28.0)$	40.4 ↑	61.51	2487 ↺
U_2 $.0625(7.3)(\frac{18.5}{2})(17.5)$	73.8 ↑	54.6	4032 ↺
U_3 $.0625(3.5)(\frac{18.5}{2})(17.5)$	35.4 ↑	48.44	1716 ↺
U_4 $.0625(3.5)(\frac{40.77}{2})(17.5)$	78.1 ↑	28.68	2238 ↺
U_5 $.0625(14.5)(\frac{40.77}{2})(17.5)$	323.3 ↑	15.69	4878 ↺
U_6 $.0625(19.1)(1.5)(28.0)$	50.1 ↑	0.74	37 ↺
Σ Uplift	601.1 (K)	(25.60)	(15388^{K} ↺)

BY F.N.K DATE N... 76 SUBJECT Cooper R... Diversion Canals DN - Drop str below F.H. Alaska SHEET NO. 8 OF
CHKD. BY DATE JOB NO.

Velocity Force on Baffles Ref: Design of Small Canal Structures" Model tests show 250 to 310 #/□'. Assume 280 #/□'



Baffle width = space between

Total width = $15/2 = 7.5'$/row

use 6 rows, assume no force on lower row

$F_H = \frac{2}{\sqrt{5}} F = 0.89F$

$F_V = \frac{1}{\sqrt{5}} F = 0.45F$

$F_H = .89(.280)(3.5)(7.5)(6) = 39.2^k \rightarrow$

$F_V = 0.45(.280)(3.5)(7.5)(6) = 19.6^k \downarrow$

$F = (.280)(3.5)(7.5)(6) = 44.1^k$

M abt El 35.3 (bot of slab)
$(44.1)(3.25) = 143'^k$

ΣH_w preceeding sheet . 190.1 →

F_H . 39.2 →

Total ΣH .= 229.3^k → To keys

Assumed passive resistance: Assume Δ distribution

$k_p = \tan^2(45+\frac{\phi}{2}) = \tan^2 60 = 3.0$

sub wt = 62.5 $P_p = 3.0(62.5) = 187$ #/□'.

$k_a = \tan^2(45-\frac{\phi}{2}) = \tan^2 30 = 0.33$

$P_a = 0.33(62.5) = 21$

Net $(P_p - P_a) = 166$ #/□'.

Conservative for wings

Max Allow P_p on upper key = $\frac{0.166(6.5)^2(28)}{2} = 130.8^k$

Assume outlet channel scoured to El 40?

Max Allow P_p on lower key = $\frac{0.166(10)^2(28)}{2} = 233.2$

Total resistance = 364.0^k

saftey Factor = $\frac{364.0}{229.3} = 1.59 > 1.5$ OK

Water - Case II

		Arm	Mom El 135.3
W_1 7.5(18.0/2)(15)(.0625)	63.3 ↓	56.27	3560
W_2 5±(18.0/2)(15)(.0625)	42.2 ↓	50.27	2120
W_3 Assume 4' deep + 60% effective - Deduct baffle volume 0.60[4(49.5±)(15) - 7(5.7)(7.5)].0625	100.2 ↓	22.14	2215
W_4 0.60(6±)(13/2)(15)(.0625)	20.2 ↓	4.0	81
Total Water	225.9k ↓	35.1	7976

Summary - Case II

	V	H		
Concrete	710.2k ↓		30.61	21737
Water	225.9 ↓		35.1	7976
Uplift	601.1 ↑		25.60	15388
Horiz. Water		190.1 →		2159
Velocity Force	19.6 ↓	39.2 →		143
H to upper key $\frac{130.8}{364.0}(229.3)$		82.4 ←	16.87	1390
H to lower key $\frac{233.2}{364}(229.3)$		146.9 ←	1.97	290
ΣV	354.6 ↓		37.0'	13123

$e = 37.0 - 62.27/2 = 5.86'$ upstr

Fdn press: upstr $= \frac{354.6}{(17.5)(62.27)}\left(1 + \frac{6(5.86)}{62.27}\right)$

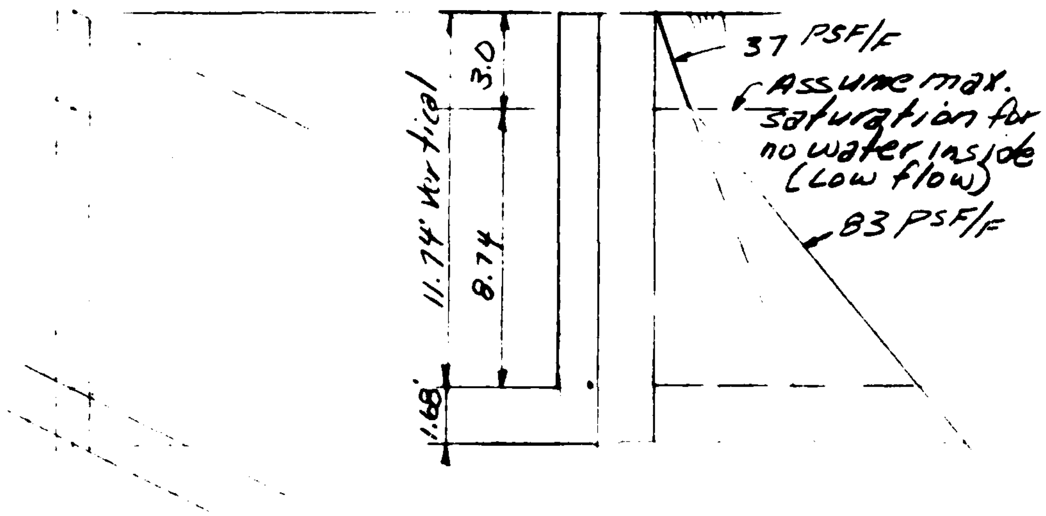
$= .326(1 + 0.565) = 0.51$ K/SF

dnstr $= .326(1 - 0.565) = 0.14$ K/SF

Floatation: safety Factor $= \frac{955.7}{601.1} = 1.59$

BY A.M.K. DATE Mar 7 SUBJECT Cooper R Rediversion SHEET NO. 10 OF
CHKD. BY DATE Walls Uni-Drop str below PH Access Rd JOB NO.

Chute side Wall Operating Condition Controls Bending in outside face



Backfill: Assume moist wt = 110 #/ft³, $\phi = 30°$

saturated wt - 125 #/ft³

$\tan^2(45 - \frac{\phi}{2}) = 0.33$

Earth Pressure: Moist active = 0.33(110) = 37 PSF/F

sub. active = 0.33(125-62.5) = 21 PSF/F

Total sub. + hydrostatic = 21+62.5 = 83

Max M at top of slab $= \frac{.037(11.74)^3}{6} + \frac{(\overset{.046}{.083-.037})(8.74)^3}{6}$

$= 10.0 + 5.1 = 15.1^{\text{IK}}$

$f'_c = 3000$ $f_c = 0.35 f'_c = 1050$ psi K = 152 a = 1.44

Min d = $\sqrt{\frac{15.1}{.152}}$ = 10" use 3" clr Cover

Min t = 10+3.5 = 13.5" use t = 15"
d = 11.5

$A_s = \frac{15.1}{1.44(11.5)} = 0.91^{\square''}$ use #7@12 + #5@12 = 0.91□"
outside face at top of slab

BY HMK DATE 11 Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 11 OF
CHKD. BY DATE Canal: DM - Drop Str. below PH Access Rd JOB NO.

Chute side Wall Cont'd

min A_s = .001 bt = .001(12)(15) = 0.18 $^{\square''}/'$
use #5 @ 12 = 0.31 $^{\square''}/'$ min

Good for M = 0.31(1.44)(11.5) = 5.1 $^{'K}$

Ck M 4' above slab = $\frac{.037(7.74)^3}{6} + \frac{.046(4.74)^2}{6}$

= 2.86 + 0.81 = 3.67 $^{'K}$ < 5.1

Drop #7's 4'+1' = 5' Above slab - continue #5's

Ck max shear $\frac{.037(11.74)^2}{2} + \frac{.046(8.74)^2}{2}$ = 2.58 + 1.76 = 4.34 ^{K}

$v = \frac{4340}{12(11.5)}$ = ...

Chute Slab: operation condition controls bending in bottom face of side wall

t = 18" 1.4 ... 4" Cl ... d_1 = ... 4.5 13.5"

Vert ... 13.5 $\frac{5}{2}$ = 15.1" ... $\frac{4.5(1.5)}{12(2)}$ 0.42' ... bot

Mom abt A_s:

Thrust = 0.037$(\frac{13.42}{2})^2$ = 3.33 ^{K} ... 4.05 = 13.5 $^{'K}$

0.046 $(\frac{10.42}{2})^2$ ... 2.50 ... 3.05 = 7.6

5.83

Wall wt 0.15(...) 2.01 ... =

Avg Fdn Press (sh 9) 0.32

Avg Uplift 9(.0625) = ...

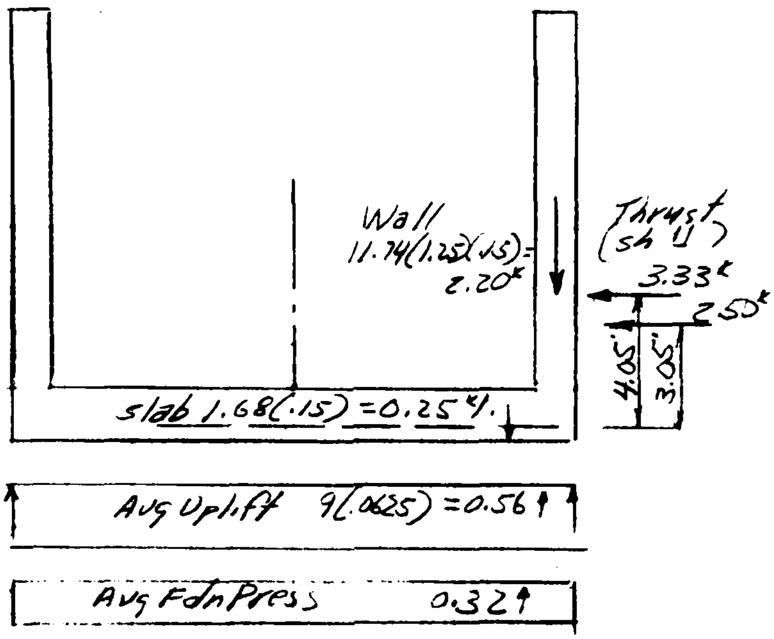
Net load ... 1.3 ... $\times \frac{(1.25)^2}{2}$ = 0.9 $^{'K}$

Total Mom abt bot A_s = 20.2 $^{'K}$

Min d reqd = $\sqrt{\frac{20.2}{0.157}}$ = 11.5" OK

$A_s = \frac{20.2}{1.44(15.1)} - \frac{5.83}{20}$ = 0.93 - 0.29 = 0.64 $^{\square''}/'$

slab Bot. face at wall



Wt of wall distributed across Fdn is only load causing slab moment to vary

Mom abt bot A_s at ℄ structure

Thrust	3.33 ←	@ 4.05	= 13.5 ik
	2.50 ←	@ 3.05	= 7.6
Wall	2.20↓	@ 8.12	17.9
Fdn press due to wall	2.20↑	@ 4.38	9.6
		Net Mom	12.8 ik

$$A_s = \frac{12.8}{1.44(15.1)} - \frac{5.83}{20} = 0.59 - 0.29 = 0.30 \ \square''/'$$

slab bot face at ℄

Carry #7 @ 12 from side wall across slab

Carry #5 @ 12 from side wall around corner and drop 3' from inside face of wall.

BY H.H.K. DATE Mar 76 SUBJECT Cooper R. Rediversion SHEET NO. 13 OF
CHKD. BY DATE Canals Dam - Drop Str JOB NO.
Below Bit Access Rd

Chute slab - Top Face - Construction condition controls. Str complete, no backfill.

Wt of walls distributed across Fdn is only load causing slab moment.

wall wt 11.74(1.25)(.15) = 2.20 k/'

moment at ℄ = 2.20(8.12) - 2.20(4.38) = 8.2 k'

3" clr cover vert d = 4.5(1.5/2) = 16.2"

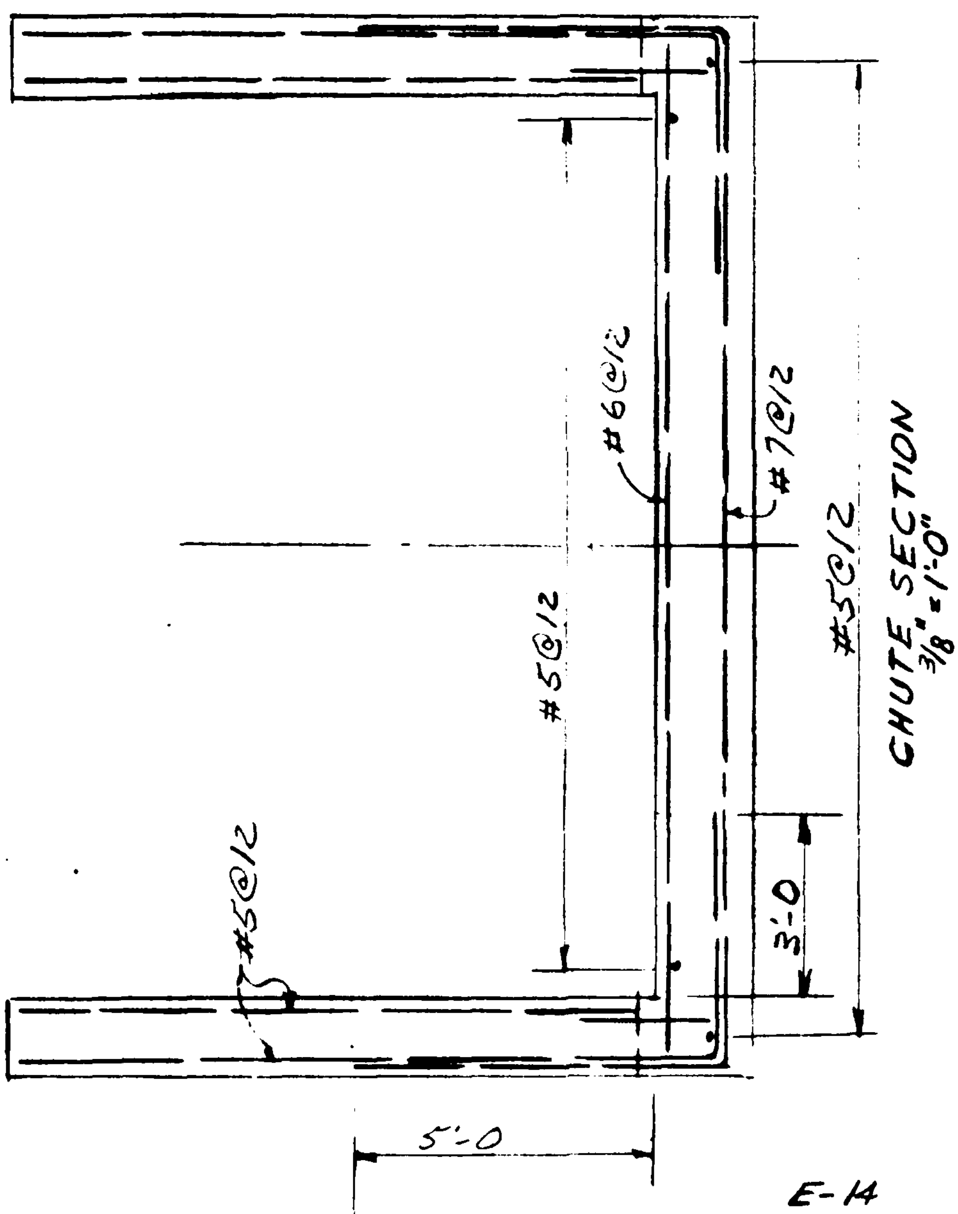
$A_s = \frac{8.2}{1.44(16.2)} = 0.35\ \square''/'$ use #6 @ 12 = 0.44 □''

Use Top Face

BY AMK DATE Mar 76 SUBJECT Cooper R Rediversion SHEET NO. 14 OF

CHKD. BY DATE Collins Cany-Drop str JOB NO.

below PH Access Rd



E-14

COOPER RIVER REDIVERSION PROJECT

INTAKE AND TAILRACE CANALS

APPENDIX F

WELL INVENTORY

U. S. ARMY ENGINEER DISTRICT, CHARLESTON

CORPS OF ENGINEERS

CHARLESTON, SOUTH CAROLINA

Cooper River Rediversion
Well Inventory

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
1	Albony Felt Co.	Singer-Layne-Atlantic Co.	G.D.-1500	3/13/74	1309'	15' Max drawdown 30'	60'24" Reduced 12" 1065'	Dom-Com	Deep well jet-9 stage 284' of 6" pipe with 60' - ss-screen	Flow, pumping, and air	X - 576600 Y - 2327600 Lat 33°24'49" Long 79°55'36"
2	City of St. Stephen, SC	Layne-Atlantic Co.	G.D. 1500	6/8/64	1265'	8'	400' of 16"	Dom-Com	Vertical turbine - 15 hp - 3 stage	Flow, pumping, and air	X - 573280 Y - 2328180 Lat 33°24'16" Long 79°55'30"
3	Miller Funk	M. C. Brassell	Sanderson Cyclone R-35	1940	120'	-	60'3"	Dom	Shallow well pump 1/3 hp	Air and pumping	X - 576675 Y - 2330270 Lat 33°24'50" Long 79°55'05"
4	Annie Funk	Owner	Steam driven hammer	1911	100'	-	40'5"	Dom	1/3 hp shallow well pump	Pumping only	X - 576875 Y - 2330160 Lat 33°24'51" Long 79°55'06"
5	Wesley Fulmore	M. C. Brassell	Sanderson Cyclone R-35	1975	150'	18'	126'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577875 Y - 2330450 Lat 33°25'02" Long 79°55'03"
6	Wesley Fulmore	-	-	-	-	-	-	-	-	-	X - 577950 Y - 2330620 Lat 33°25'02" Long 79°55'00"
7	Wilson T. Baggett	-	-	1935	100'	23'	50'	Dom	1/2 hp deep well jet	-	X - 574610 Y - 2332150 Lat 33°24'29" Long 79°54'42"
8	Huey Wadford	M. C. Brassell	Sanderson Cyclone R-35	1964	150'	25'	30'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 575160 Y - 2331700 Lat 33°24'34" Long 79°54'47"
9	Carl Wadford	Owner	Hand and tractor driven	1965	50'	12'	30'2"	Dom	1/3 hp shallow well pump	Pumping only	X - 575300 Y - 2331710 Lat 33°24'36" Long 79°54'47"
10	Furman Wadford	Mixon	Hand driven and jetted	1963	120'	18'	85'2"	Dom	1/3 hp shallow well pump	Air and pumping	X - 575400 Y - 2331600 Lat 33°24'36" Long 79°54'48"
11	Kelly Wadford	M. C. Brassell	Sanderson Cyclone R-35	1964	125'	21'	80'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 576090 Y - 2332020 Lat 33°24'44" Long 79°54'44"
12	Thomas Wadford	M. C. Brassell	Sanderson Cyclone R-35	1964	125'	21'	80'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577420 Y - 2333250 Lat 33°24'56" Long 79°54'29"

Cooper River Rediversion
Well Inventory
(continued)

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
13	Albert Cooper	M. C. Brassell	Sanderson Cyclone R-35	1940	200'	25'	85'	Dom	1/2 hp deep well jet 42' drop pipe	Air pumping	X - 578440 Y - 2325320 Lat 33°25'07" Long 79°56'03"
14	Albert Cooper	Same	Hand driven	1940	45'	12'	30'	Dom	Pitcher	Pumping only	X - 577890 Y - 2325860 Lat 33°25'02" Long 79°55'56"
15	Albert Cooper	Same	Hand driven and jetted	1939	45'	15'	30'	Dom	Pitcher	Pumping only	X - 577960 Y - 2326900 Lat 33°25'05" Long 79°55'44"
16	Charles Lail	M. C. Brassell	Sanderson Cyclone R-35	1975	165'	20'	132'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air pumping	X - 573600 Y - 2328720 Lat 33°24'20" Long 79°55'23"
17	L. C. Poston	M. C. Brassell	Sanderson Cyclone R-35	1940	130'	-	47'2"	Dom	1/3 hp shallow well 26' drop pipe	Air pumping	X - 573430 Y - 2331690 Lat 33°24'29" Long 79°54'48"
18	Donald Funk	Same	Hand driven and jetted	1939	40'	12'	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 579120 Y - 2322875 Lat 33°25'15" Long 79°56'32"
19	Donald Funk	M. C. Brassell	35-B-	1947	11[illegible]	15'	52'2"	Dom-Com	1/3 hp shallow well jet	Pumping only	X - 577640 Y - 2322700 Lat 33°25'00" Long 79°56'34"
20	H. O. Browder	Smith and Rembert	Hand driven	1939	25'	13'	20'	Dom	1/3 hp shallow well pump	Pumping only	X - 579675 Y - 2322410 Lat 33°25'20" Long 79°56'37
21	Lewis Edwards	Smith and Rembert	Hand driven	1950	60'	15'	40'	Dom	1/3 hp shallow well pump	Pumping only	X - 579320 Y - 2322110 Lat 33°25'17" Long 79°56'41"
22	Tom Addison	Same	Hand - open well	1939	30'	13'	0 - Open well	Dom	1/3 hp shallow well pump	Pumping only	X - 579000 Y - 2321960 Lat 33°25'14" Long 79°56'42"
23	Jim Dingle	M. C. Brassell	Sanderson Cyclone R-35	1975	150'	25'	120'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 578930 Y - 2321960 Lat 33°25'13" Long 79°56'42"
24	Shirley May Addison	Same	Hand "open well"	1947	30'	10'	0	Dom	1/3 hp shallow well pump	Pumping	X - 580130 Y - 2320510 Lat 33°25'25" Long 79°56'59"

Cooper River Rediversion
Well Inventory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
25	Willie Merrell, Jr.	Myer's	Hand driven and jetted	1970	65'	18'	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 580320 Y - 2320050 Lat 33°25'25" Long 79°57'05"
26	Mary Lee Davis	Smith and Rembert	Hand driven	1970	66'	16'	36'2"	Dom	1/3 hp shallow well pump	Pumping only	X - 580180 Y - 2320100 Lat 33°25'24" Long 79°57'04"
27	Frank Jefferson	Smith and Rembert	Hand "open well"	1940	35'	10'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 580120 Y - 2320130 Lat 33°25'23" Long 79°57'04"
28	Willie Howard, Jr.	Same	Hand "open well"	1940	35'	12'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 580100 Y - 2320040 Lat 33°25'23" Long 79°57'05"
29	Elizabeth Ford	Same	Hand driven	1939	30'	-	-	Dom	1/3 hp shallow well pump	Pumping only	X - 580560 Y - 2319680 Lat 33°25'29" Long 79°57'10"
29A	Elizabeth Ford	Same	Hand driven	1939	30'	-	-	Dom	1/3 hp shallow well pump	Pumping only	X - 580490 Y - 2319580 Lat 33°25'28" Long 79°57'11"
30	Mose Bryant	Myers	-	1940	156'	-	30'2"	Dom	1/2 hp deep well jet 30' drop pipe	Air and pumping	X - 581030 Y - 2320210 Lat 33°25'44" Long 79°57'03"
31	St. Matthew's Baptist Church	Same	Hand driven	1939	35'	15'	29'	Dom	1/3 hp shallow well pump	Pumping only	X - 580600 Y - 2320050 Lat 33°25'30" Long 79°57'05"
32	Violita Addison	Same	Hand driven	1960	70'	17'	70'	Dom	1/3 hp shallow well pump	Pumping only	X - 579600 Y - 2320080 Lat 33°25'20" Long 79°57'05"
33	Elizabeth Addison	Same	Hand driven	1940	40'	-	0	Dom	1/3 hp shallow well pump	Pumping only	X - 579100 Y - 2320380 Lat 33°25'14" Long 79°57'02"
33A	Elizabeth Addison	Same	Hand driven	1940	40'	-	0	Dom	1/3 hp shallow well pump	Pumping only	X - 579175 Y - 2320420 Lat 33°24'15" Long 79°57'01"
34	John Addison	M. C. Brassell	Sanderson Cyclone R-35	1970	150'	-	34'	Dom	Deep well jet 42' drop pipe	Air and pumping	X - 584350 Y - 2318920 Lat 33°26'07" Long 79°57'18"

Cooper River Rediversion
Well Inventory
(Continued)

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
35	Thomas Dingle	Same	Hand "open well"	1940	30'	-	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 584190 Y - 2318900 Lat 33°26'06" Long 79°57'18"
36	Primas Singletary	M. C. Brassell	Sanderson Cyclone R-35	1973	135'	18'	86'	Dom	1/2 hp deep well jet	Air and pumping	X - 583250 Y - 2318800 Lat 33°25'56" Long 79°57'20"
37 & 38	Abraham Broughton	Same	Hand	1939	30'	-	-	Dom	1/3 hp shallow well pump	Pumping only	(37) X - 583100 Y - 2318710 Lat 33°25'55" Long 79°57'21" (38) X - 582890 Y - 2319050 Lat 33°25'53" Long 79°57'13"
39	Douglas Addison	M. C. Brassell	Sanderson Cyclone R-35	1974	140'	16'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 582630 Y - 2319340 Lat 33°25'50" Long 79°57'13"
40	Willie Addison	Same	Hand driven	1938	25'	12'	21'+4' sand screen	Dom	Pitcher	Pumping only	X - 582200 Y - 2319100 Lat 33°25'46" Long 79°57'16"
41	Ruffos Sumpter	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	19'	105'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 582450 Y - 2318660 Lat 33°25'49" Long 79°57'19"
42	Charles Sumpter	Same	Hand "open well"	1929	30'	-	-	Dom	1/3 hp shallow well pump	Pumping only	X - 582350 Y - 2318550 Lat 33°25'48" Long 79°57'23"
43	Jedon Davis	M. C. Brassell	Sanderson Cyclone R-35	1975	150'	18'	93'	Dom	1/2 hp deep well jet	Air pumping	X - 581680 Y - 2318420 Lat 33°25'41" Long 79°57'25"
44	William Jefferson	Same	Hand "open well"	1929	30'	12'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 580880 Y - 2318320 Lat 33°25'33" Long 79°57'26"
45	Jamie Sumpter	Same	Hand "open well"	1940	30'	12'	-	Dom	1/3 hp shallow well pump	Pumping only	X - 581700 Y - 2319180 Lat 33°25'41" Long 79°57'15"

Cooper River Rediversion
Well Inventory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
46	Pumis Addison	Same	Hand "open well"	1947	50'	13'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 581300 Y - 2319050 Lat 33°25'37" Long 79°57'17"
47	Izia Addison	Same	Hand	1940	50'	-	0	Dom	1/3 hp shallow well pump	Pumping	X - 581100 Y - 2319340 Lat 33°25'35" Long 79°57'13"
48	Jack O. Locklair	M. C. Brassell	Sanderson Cyclone R-35	1945	150'	-	100'2"	Dom - Cattle Only	Tractor mounted pump	Air and pumping	X - 582850 Y - 2321460 Lat 33°25'52" Long 79°56'48"
49	Josephine Addison	Same	Hand driven	1947	81'	16'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 579180 Y - 2321700 Lat 33°25'15" Long 79°56'46"
50	Sulie McKiry	Same	Hand driven	1945	78'	17'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 578850 Y - 2321280 Lat 33°25'12" Long 79°56'50"
51	Minnie Simmons	Same	Hand driven	1960	80'	17'	0	Dom	1/3 hp shallow well pump	Pumping only	X - 579530 Y - 2320900 Lat 33°25'19" Long 79°56'55"
52	Mary Dingle	M. C. Brassell	Sanderson Cyclone R-35	1969	158'	-	120'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 579110 Y - 2320900 Lat 33°25'15" Long 79°56'55"
53	Sealy Robinson	Same	Hand driven and jetted	1940	100'	-	80'2"	Dom-Com	1/2 hp deep well jet	Pumping only	X - 579090 Y - 2320950 Lat 33°25'14" Long 79°56'54"
54	Dasie Maxwell	Same	Hand driven	1940	73'	-	0	Dom	1/3 hp shallow well pump	Pumping only	X - 578700 Y - 2321150 Lat 33°25'11" Long 79°56'52"
55	Rev. Mack	Owner	Hand driven	1940	80'	12'	80'	Dom	1/3 hp shallow well pump	Pumping only	X - 576950 Y - 2323240 Lat 33°24'53" Long 79°56'28"
56	Robert D. Owens	M. C. Brassell	Sanderson Cyclone R-35	1967	156'	16'	120'2"	Dom	1/2 hp deep well jet	Air and pumping	X - 577250 Y - 2322730 Lat 33°24'56" Long 79°56'34"
57	M. W. Browder	M. C. Brassell	Sanderson Cyclone R-35	1945	150'	16'	110'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577380 Y - 2322450 Lat 33°24'57" Long 79°56'36"

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
58	Henry McKelvey (frame church on 52)	Same	Hand driven	1940	42'	16'	42'	Dom	1/3 hp shallow well pump	Pumping only	X - 577850 Y - 2322000 Lat 33°25'02" Long 79°56'42"
59	Jefferson Gourdin	M. C. Brassell	Sanderson Cyclone R-35	1971	156'	16'	112'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 578000 Y - 2321290 Lat 33°25'04" Long 79°56'50"
60	Alvin Jones	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	17'	98'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 580530 Y - 2318450 Lat 33°25'29" Long 79°57'24"
61	Jessy Addison	M. C. Brassell	Sanderson Cyclone R-35	1954	140'	19'	90'2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 580120 Y - 2318430 Lat 33°25'26" Long 79°57'25"
62	Mary Holdman	Smith and Rembert	Hand driven and jetted	1974	76'	15'	40' 1 1/4"	Dom	1/3 hp shallow well pump	Pumping only	X - 576000 Y - 2324500 Lat 33°24'44" Long 79°56'13"
63	Mose Middleton	Smith and Rembert	Hand driven	1974	98'	16'	38'	Dom	1/3 hp shallow well pump 76' 3/4" drop pipe	Pumping only	X - 576500 Y - 2322910 Lat 33°24'49" Long 79°56'32"
64	Frank Gwinn	Myers	-	1947	35'	10'	35'	Dom	1/3 hp shallow well pump	Pumping only	X - 574500 Y - 2324110 Lat 33°24'29" Long 79°56'17"
65	L. Davis	Smith and Rembert	Hand driven	1970	50'	17'	30' 1 1/4"	Dom	1/3 hp shallow well pump	Pumping only	X - 573820 Y - 2312110 Lat 33°24'23" Long 79°58'39"
66, 67 & 68	Georgia Pacific Corp-Resin Division	Bilton well drilling Eutawville, SC	Quick drill "DSI"	3/1/72	400' 20" wells grav. packed to 8"	-	400' 8" min. 40' ss. screen	Dom-Com	1-20 hp 80-3L - 2-8" sub.	Air and pumping	(66) X - 573250 Y - 2310630 Lat 33°24'18" Long 79°58'57" (67) X - 573550 Y - 2310650 Lat 33°24'21" Long 79°58'57" (68) X - 573720 Y - 2310680 Lat 33°24'23" Long 79°58'56"

Cooper River Rediversion
Well Inventory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
69, 70 & 71	Georgia Pacific - Plywood Division	Bilton well driller	Quick Drill "DSI"	3/1/72	400' 20" wells grav. packed to 8"	20'	400' min. 40' ss screen	Dom-Com	3 sub. 8" pumps with 200 GPM min.	Air and pumping	(69) X - 572900 Y - 2311030 Lat 33°24'15" Long 79°58'53" (70) X - 572820 Y - 2311250 Lat 33°24'14" Long 79°58'50" (71) X - 572400 Y - 2311120 Lat 33°24'10" Long 79°58'51"
72	Clair Judge	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	16'	100' 2" 42' drop pipe	Dom	1/2 hp deep well jet	Air and pumping	X - 577750 Y - 2310400 Lat 33°25'03" Long 79°58'59"
73	Geraldine Gourdin	Bill Bilton	Sanderson Cyclone R-35	1974	95'	18'	50' 2"	Dom	1/2 hp deep well jet	Air pumping	X - 577580 Y - 2309490 Lat 33°25'01" Long 79°59'10"
74	Ordell Middleton	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	18'	100' 2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 578300 Y - 2309550 Lat 33°25'09" Long 79°59'09"
75	Mamie Smith	Smith and Rembert	Hand driven	1940	40'	10'	40'	Dom	Pitcher	Pumping only	X - 574990 Y - 2311440 Lat 33°24'35" Long 79°58'47"
76	Sile West	Smith and Rembert	Hand driven	1940	40'	10'	40'	Dom	Pitcher	Pumping only	X - 575320 Y - 2311950 Lat 33°24'39" Long 79°58'41"
77	-	-	-	-	-	-	-	-	-	-	X - 578050 Y - 2308500 Lat 33°25'06" Long 79°59'21"
78	Clarance Prioleau	Smith and Rembert	Hand driven	1974	90'	16'	50'	Dom	1/3 hp shallow well pump	Pumping only	X - 577100 Y - 2310650 Lat 33°24'56" Long 79°58'56"
79	George Mazyck	Smith and Rembert	Hand driven	1954	90'	16'	50'	Dom	1/2 hp deep well jet 42' drop pipe	Pumping only	X - 576990 Y - 2310390 Lat 33°24'55" Long 79°58'59"

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Well Inventory
(Continued)

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well Was Developed	State Plane Coordinates
80	Henry Prioleau	Smith and Rembert	Hand driven	1940	40'	15'	40'	Dom	Pitcher	Pumping only	X - 577120 Y - 2310290 Lat 33°24'56" Long 79°59'01"
81	Gwendolyn White	Smith and Rembert	Hand driven	1945	90'	16'	50'	Dom	1/3 hp shallow well pump	Pumping only	X - 574710 Y - 2311700 Lat 33°24'33" Long 79°58'44"
82	Salum West	Smith and Rembert	Hand driven	1947	90'	17'	50'	Dom	Pitcher	Pumping only	X - 574950 Y - 2311775 Lat 33°24'35" Long 79°58'43"
83	Elizabeth Bryant	Smith and Rembert	Hand driven	1960	90'	17'	50'	Dom	1/3 hp shallow well pump	Pumping	X - 575550 Y - 2312500 Lat 33°24'40" Long 79°58'35"
84	John Myzon	Smith and Rembert	Hand driven	1954	90'	18'	50'	Dom	1/3 hp shallow well pump 30' drop pipe	Pumping only	X - 575675 Y - 2312280 Lat 33°24'42" Long 79°58'37"
85	Alvin Manigault	M. C. Brassell	Sanderson Cyclone R-35	1948	200'	17'	100' 3"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574450 Y - 2311920 Lat 33°24'30" Long 79°58'41"
86	Harry Lessington	Smith and Rembert	Hand driven	1962	165'	15'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Pumping only	X - 574860 Y - 2312900 Lat 33°24'33" Long 79°58'30"
87	Stanley Lessington	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574920 Y - 2312980 Lat 33°24'34" Long 79°58'29"
88	Margaret Glover	M. C. Brassell	Sanderson Cyclone R-35	1972	150'	16'	90'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 575050 Y - 2312900 Lat 33°24'35" Long 79°58'30"
89	John Williams	Smith and Rembert	Hand driven	1972	100'	15'	40'	Dom	1/3 hp shallow well pump	Pumping only	X - 575120 Y - 2313130 Lat 33°24'36" Long 79°58'27"
90	Mary Davis	Smith and Rembert	Hand driven	1962	100'	17'	70'	Dom	1/3 hp shallow well pump	Pumping only	X - 574700 Y - 2313180 Lat 33°24'32" Long 79°58'27"
91	Lovene Gourdine	Smith and Rembert	Hand driven	1963	90'	16'	40'	Dom	1/3 hp shallow well pump	Pumping only	X - 574300 Y - 2313550 Lat 33°24'28" Long 79°58'23"

Cooper River Rediversion
Well Inventory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
92	James Perry	M. C. Brassell	Sanderson Cyclone R-35	1970	150'	17'	90'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574550 Y - 2313620 Lat 33°24'30" Long 79°58'21"
93	William Bennett	Smith and Rembert	Hand driven	1964	90'	16'	45'	Dom	1/3 hp shallow well pump	Pumping only	X - 574350 Y - 2313910 Lat 33°24'29" Long 79°58'18"
94	Andy Burgress	Smith and Rembert	Hand driven	1969	90'	15'	40'	Dom	1/3 hp shallow well pump 30' drop pipe	Pumping only	X - 574310 Y - 2313990 Lat 33°24'28" Long 79°58'17"
95	Joe Williams	Smith and Rembert	Hand driven	1944	90'	17'	50'	Dom	1/3 hp shallow well pump 42' drop pipe	Pumping only	X - 574500 Y - 2313875 Lat 33°24'30" Long 79°58'18"
96	James Walker	M. C. Brassell	Sanderson Cyclone R-35	1972	90'	15'	50' 2"	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577300 Y - 2314240 Lat 33°24'56" Long 79°58'14"
97	George Walker	Smith and Rembert	Hand driven	1947	90'	16'	50'	Dom	1/3 hp shallow well pump	Pumping	X - 576980 Y - 2314150 Lat 33°24'54" Long 79°58'15"
98	Henry Griffin	Smith and Rembert	Hand driven	1974	90'	17'	50'	Dom	1/3 hp shallow well pump	Pumping only	X - 576920 Y - 2313980 Lat 33°24'53" Long 79°58'17"
99	Freddie James	Tom Bilton	Sanderson Cyclone R-35	1974	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577150 Y - 2314120 Lat 33°24'56" Long 79°58'15"
100	L. T. Gaston	Smith and Rembert	Hand jet & driven	1945	80'	18'	42'	Dom	1/3 hp shallow well	Pumping only	X - 577500 Y - 2314190 Lat 33°24'59" Long 79°58'14"
101	John Neal	M. C. Brassell	Sanderson Cyclone R-35	1973	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577050 Y - 2313980 Lat 33°24'55" Long 79°58'17"
102	Lou Scott	M. C. Brassell	Sanderson Cyclone R-35	1973	150'	16'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577130 Y - 2313650 Lat 33°24'56" Long 79°58'21"
103	George Jefferson	M. C. Brassell	Sanderson Cyclone R-35	1972	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577375 Y - 2313490 Lat 33°24'58" Long 79°58'23"

Cooper River Rediversion
Well Inventory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
104	F. P. McKeethan	Tom Bilton	Sanderson Cyclone R-35	1973	140'	17'	100' 2"	Dom	1/2 hp deep well jet 30' drop pipe	Air and pumping	X - 577700 Y - 2313960 Lat 33°25'01" Long 79°58'17"
105	P. P. Green	Tom Bilton	Sanderson Cyclone R-35	1974	140'	17'	90'	Dom	1/2 hp deep well jet 30' drop pipe	Air and pumping	X - 577400 Y - 2313850 Lat 33°24'59" Long 79°58'19"
106	Johnie Prioleau	Tom Bilton	Sanderson Cyclone R-35	1973	114'	18'	90'	Dom	1/3 hp shallow well jet 30' drop pipe	Air and pumping	X - 576820 Y - 2312100 Lat 33°24'53" Long 79°58'39"
107	James Gaillard	M. C. Brassell	Sanderson Cyclone R-35	1974	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 576730 Y - 2312375 Lat 33°24'52" Long 79°58'36"
108	John A. White	M. C. Brassell	Sanderson Cyclone R-35	1973	150'	18'	90'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 576680 Y - 2312560 Lat 33°24'52" Long 79°58'34"
109 & 110	William White	Same	Hand driven	1973	90'	17'	50'	Dom	1/3 hp shallow well pump	Pumping only	(109) X - 576590 Y - 2312710 Lat 33°24'51" Long 79°58'32" (110) X - 576800 Y - 23123[illegible]0 Lat 33°24'53" Long 79°58'36"
111	V. E. Bennett	M. C. Brassell	Sanderson Cyclone R-35	1972	150'	17'	90'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 577060 Y - 2312180 Lat 33°24'55" Long 79°58'38"
112 & 113	Viola King	Smith and Rembert	Hand driven	1962	125'	16'	90'	Dom	1/3 hp shallow well jet 45' drop pipe	Pumping only	(112) X - 576980 Y 2312510 Lat 33°24'55" Long 79°58'35" (113) X - 576900 Y - 2312450 Lat 33°24'54" Long 79°58'35"
114	Grady Davis	Smith and Rembert	Hand driven	1964	55'	12'	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 578320 Y - 2320280 Lat 33°25'07" Long 79°57'03"

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Well Intentory
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Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
115	Lovine Davis	Smith and Rembert	Hand driven	1963	60'	19'	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 578475 Y - 2320120 Lat 33°25'09" Long 79°57'05"
116	Bill Addison	Corps of Engineers	Failing 314	1964	-	-	-	Dom	-	-	X - 578760 Y - 2319790 Lat 33°25'12" Long 79°57'09"
117	Oliver Davis	Smith and Rembert	Hand driven	1963	90'	18'	40'	Dom	1/3 hp shallow well pump	Pumping only	X - 578160 Y - 2320530 Lat 33°25'06" Long 79°56'59"
118	Tom Crawford	M. C. Brassell	Sanderson Cyclone R-35	1961	150'	18'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 575300 Y - 2319050 Lat 33°24'37" Long 79°57'17"
119	Eadie Herman	Mitchum	-	1974	90'	17'	60'	Dom	1/2 hp deep well jet 42' drop pipe	Pumping only	X - 575350 Y - 2320610 Lat 33°24'37" Long 79°56'59"
120	Jack Locklair	M. C. Brassell	Sanderson Cyclone R-35	1969	90'	17'	50'	Dom	Pitcher	Air and pumping	X - 576700 Y - 2316480 Lat 33°24'52" Long 79°57'47"
121	Orland L. Brown	Bilton	Sanderson Cyclone R-35	1969	135'	18'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 578560 Y - 2309010 Lat 33°25'11" Long 79°59'15"
122	E. H. Gaston	Smith and Rembert	Hand driven	1968	100'	16'	30'	Dom	1/3 hp shallow well pump	Pumping only	X - 578600 Y - 2309100 Lat 33°25'12" Long 79°59'14"
123	Mary Rodgers	M. C. Brassell	Sanderson Cyclone R-35	1971	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 578310 Y - 2308780 Lat 33°25'09" Long 79°59'18"
124	Isack White	Same	Hand driven	1948	20'	12'	20'	Dom	Pitcher	Pumping only	X - 577320 Y - 2307750 Lat 33°24'58" Long 79°59'30"
125	John White	Same	Hand driven	1948	20'	12'	20'	Dom	Pitcher	Pumping only	X - 577100 Y - 2307700 Lat 33°24'57" Long 79°59'31"
126	John Edwards	Same	Hand driven	1948	20'	12'	20'	Dom	Pitcher	Pumping only	X - 577490 Y - 2307820 Lat 33°25'01" Long 79°59'29"

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How well was Developed	SC State Plane Coordinates
127	Jeffro Gourdine	Same	Hand driven	1948	20'	12'	20'	Dom	Pitcher	Pumping only	X - 577420 Y - 2307190 Lat 33°25'00" Long 79°59'36"
128 (3 wells)	J. W. Wadford	Mixon	-	1942	130'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	(a) X - 574600 Y - 2333600 Lat 33°24'28" Long 79°54'24" (b) X - 574840 Y - 2333575 Lat 33°24'31" Long 79°54'26" (c) X - 574900 Y - 2333620 Lat 33°24'32" Long 79°54'25"
129	Herbert Wiggins	Mixon	-	1943	100'	18'	80'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574610 Y - 2333620 Lat 33°24'29" Long 79°54'25"
130	David Wadford	M. C. Brassell	Sanderson Cyclone R-35	1965	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574460 Y - 2333450 Lat 33°24'27" Long 79°54'27"
131	David Wadford	Mixon	-	1940	100'	18'	80'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 574340 Y - 2333350 Lat 33°24'26" Long 79°54'28"
132	-	-	-	-	-	-	-	-	-	-	X - 574150 Y - 2333175 Lat 33°24'24" Long 79°54'30"
133	M. Guerry	-	-	-	-	-	-	-	-	-	X - 573550 Y - 2333300 Lat 33°24'18" Long 79°54'29"
134	Ed Brown	Same	Hand driven	1940	40'	17'	30'	Dom	Pitcher	Pumping only	X - 573530 Y - 2333780 Lat 33°24'18" Long 79°54'24"
135	May Renn	M. C. Brassell	Sanderson Cyclone R-35	1969	76'	19'	50'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 573490 Y - 2334690 Lat 33°24'17" Long 79°54'13"

Cooper River Rediversion
Well Inventory
(continued)

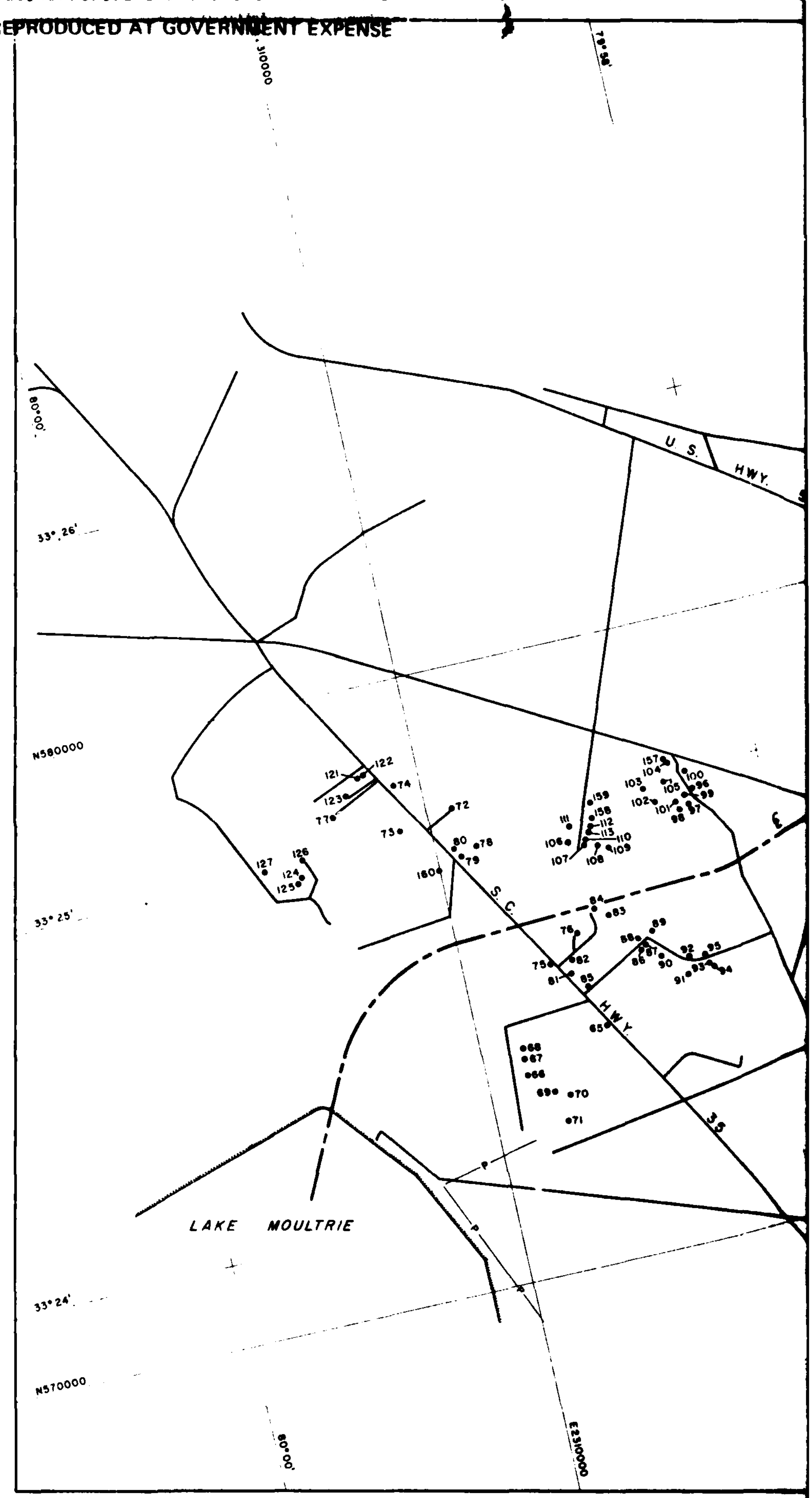
Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
136	Miller Funk (Old Ice House)	Same	Hand driven	1963	60'	18'	50'	Dom	1/2 hp deep well jet 42' drop pipe	Pumping only	X - 573510 Y - 2335200 Lat 33°24'17" Long 79°54'07"
137	Dale Funk	Corps of Engineers	Failing 314	1970	120'	30'	80' 6"	Dom	1/2 hp deep well jet 60' drop pipe	Air and pumping	X - 573550 Y - 2336300 Lat 33°24'18" Long 79°53'53"
138	Doc Hiott	M. C. Brassell	Sanderson Cyclone R-35	1947	80'	17'	50'	Dom	1/2 hp deep well jet	Air and pumping	X - 573830 Y - 2336510 Lat 33°24'21" Long 79°53'50"
139	William Ford	Same	Hand driven	1939	40'	12'	30'	Dom	Pitcher	Pumping only	X - 573400 Y - 2337350 Lat 33°24'16" Long 79°53'41"
140	Gadson Hood	M. C. Brassell	Sanderson Cyclone R-35	1945	100'	19'	70'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 573020 Y - 2338020 Lat 33°24'12" Long 79°53'33"
141	Bill Datson	M. C. Brassell	Sanderson Cyclone R-35	1972	150'	17'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 573175 Y - 2338100 Lat 33°24'14" Long 79°53'32"
142	Lewis Guerry	M. C. Brassell	Sanderson Cyclone R-35	1949	100'	19'	30'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 572300 Y - 2338960 Lat 33°24'10" Long 79°53'22"
143	L. M. Keller	M. C. Brassell	Sanderson Cyclone R-35	1960	150'	20'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 572650 Y - 2329390 Lat 33°24'09" Long 79°53'17"
144 & 145	C. R. Mozingo	-	-	-	-	-	-	-	-	-	(144) X - 572250 Y - 2340400 Lat 33°24'05" Long 79°53'05" (145) X - 572100 Y - 2340500 Lat 33°24'03" Long 79°53'04"
146	A. L. Jeringan	Same	Hand dug "open well"	1940	40'	16'	0	Dom	1/3 hp shallow well pump 21' drop pipe	Pumping only	X - 571780 Y - 2341300 Lat 33°23'59" Long 79°52'55"

Well Inventory
(Continued)

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
147	Bud Jeringan	M. C. Brassell	Sanderson Cyclone R-35	1964	150'	19'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 572500 Y - 2341420 Lat 33°24'07" Long 79°52'55"
148	Harry Carr	Same	Hand driven	1960	40'	16'	30'	Dom	1/3 hp shallow well pump	Pumping	X - 571450 Y - 2341690 Lat 33°23'56" Long 79°52'50"
149	Joe W. Middleton	M. C. Brassell	Sanderson Cyclone R-35	1968	150'	19'	100'	Dom	1/2 hp deep well jet 42' drop pipe	Air and pumping	X - 571140 Y - 2342120 Lat 33°23'53" Long 79°52'45"
150	T. H. Jaudon	-	-	-	-	-	-	-	-	-	X - 570980 Y - 2342500 Lat 33°23'52" Long 79°52'40"
151	Clarence Funk	M. C. Brassell	Sanderson Cyclone R-35	1968	90'	13'	50'	Dom	Pitcher	Air and pumping	X - 572600 Y - 2345800 Lat 33°24'07" Long 79°52'02"
152	Robert Mathue	Mitchum	-	1957	105'	19'	85'	Dom	1/3 hp shallow well jet	Air and pumping	X - 569350 Y - 2346700 Lat 33°23'35" Long 79°51'52"
153	E. Shaw	Same	Hand driven	1940	23'	15'	23'	Dom	Pitcher	Pumping only	X - 568675 Y - 2347550 Lat 33°23'30" Long 79°51'42"
154	Riley Keller	Mitchum	-	1948	68'	21'	50'	Dom	1/3 hp shallow well pump	Air and pumping	X - 571450 Y - 2348780 Lat 33°23'55" Long 79°51'27"
155	Hood & Orvin (farm @ River Mon-13-13-A)	M. C. Brassell	Sanderson Cyclone R-35	1969	85'	21'	50'	Dom	Pitcher	Air and pumping	X - 572400 Y - 2350930 Lat 33°24'03" Long 79°51'02"
156	Hood & Orvin (@ H.W.-45)	M. C. Brassell	Sanderson Cyclone R-35	1948	85'	18'	50'	Dom	Pitcher	Air and pumping	X - 568010 Y - 2349680 Lat 33°23'22" Long 79°51'17"
157	J. A. Brouton	Smith and Rembert	Hand driven and jetted	1975	80'	18'	40'	Dom	1/3 hp shallow well	Pumping only	X - 577760 Y - 2313930 Lat 33°25'02" Long 79°58'17"
158	Namon Perkins	Smith and Rembert	Hand driven and jetted	1960	101'	15'	60'	Dom	1/3 hp shallow well	Pumping only	X - 577100 Y - 2312560 Lat 33°24'56" Long 79°58'28"

Cooper River Rediversion
Well Inventory
(continued)

Well No.	Owner	Driller	Drill Rig	Date	Depth of Well	Water Table	Depth of Casing	Water Use	Pump	How Well was Developed	SC State Plane Coordinates
159	Shep Dingle	Smith and Rembert	Hand driven and jetted	1968	117'	12'	70'	Dom	1/3 hp shallow well	Pumping only	X - 577350 Y - 2312600 Lat 33°24'58" Long 79°58'28"
160	-	-	-	-	-	-	-	-	-	-	X - 576820 Y - 2309990 Lat 33°24'54" Long 79°59'04"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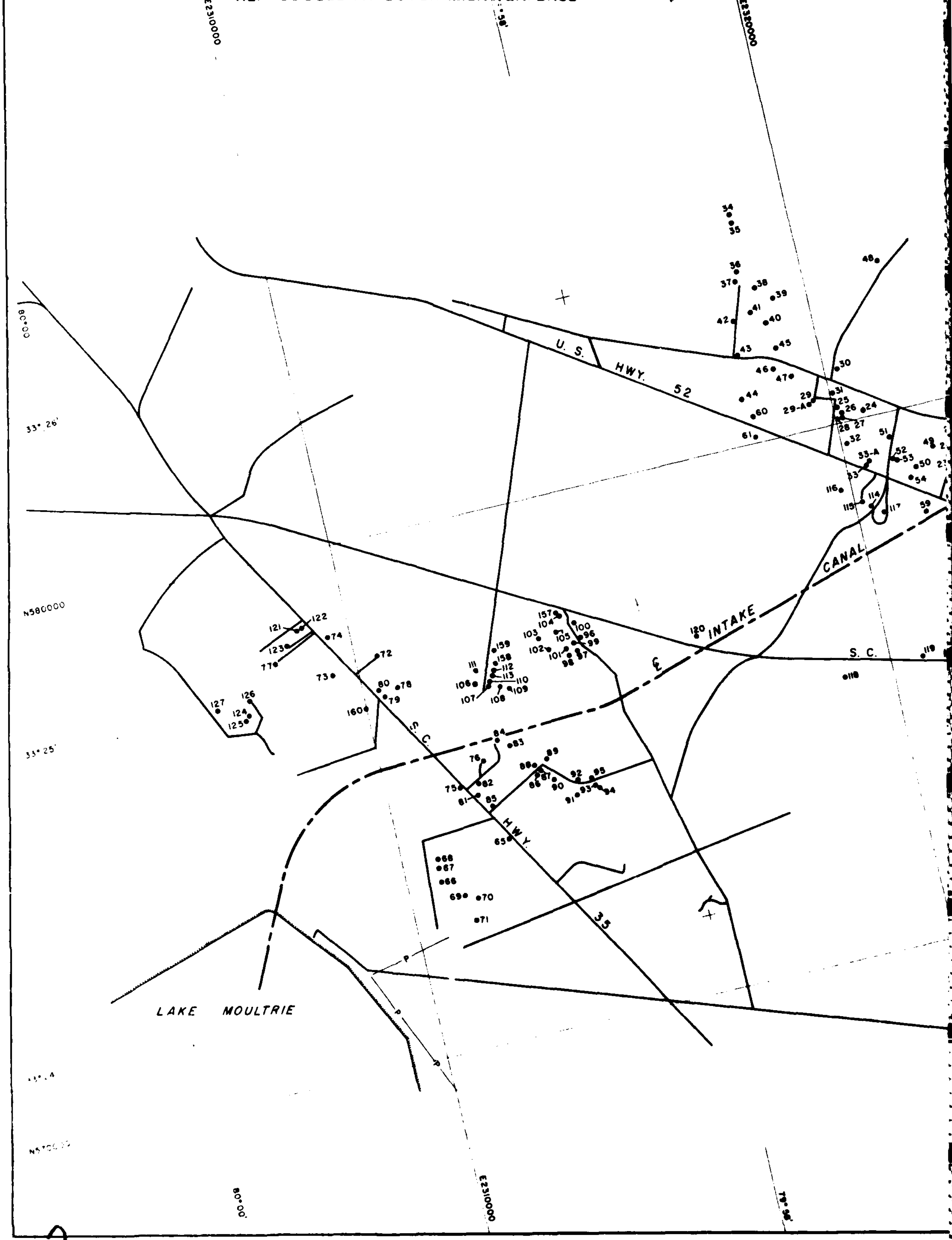
S.C. HWY. 35
LAKE MOULTRIE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S. C.
LAKE MOULTRIE

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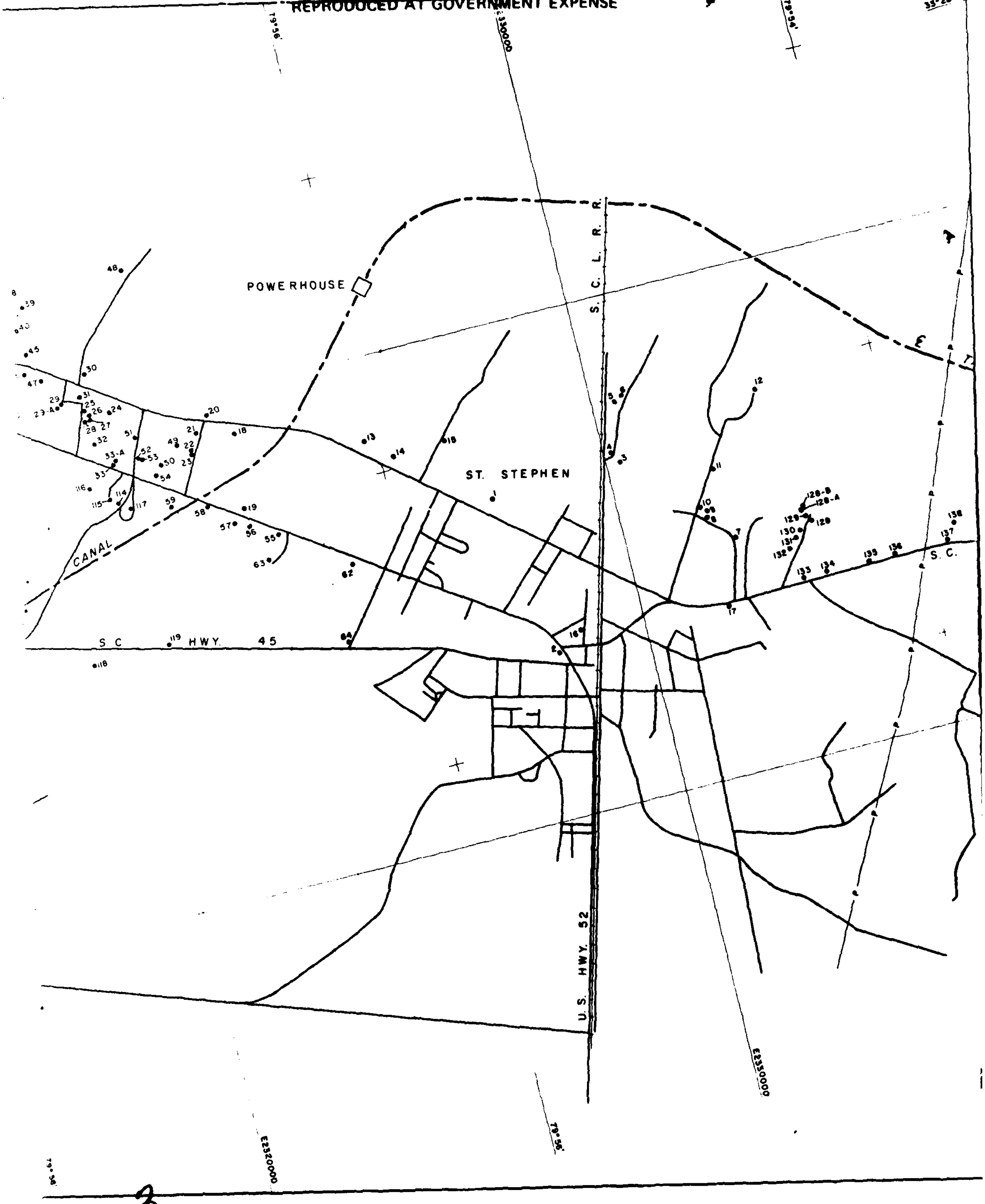
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S.C.
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U S ARMY ENGINEER DISTRICT, CHARLESTON
CORPS OF ENGINEERS
CHARLESTON, SOUTH CAROLINA

WELL INVENTORY

COOPER RIVER REDIVERSION PROJECT
LAKE MOULTRIE & SANTEE RIVER SOUTH CAROLINA

SCALE AS SHOWN	DM 9 INTAKE AND TAILRACE CANALS	PLATE
DATE NOV 1977		FILE NO 10043

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COOPER RIVER REDIVERSION PROJECT

INT^"E AND TAILRACE CANALS

APPENDIX G

REVISED INTAKE LEVEE VICINITY OF POWERHOUSE

U. S. ARMY ENGINEER DISTRICT, CHARLESTON

CORPS OF ENGINEERS

CHARLESTON, SOUTH CAROLINA

APPENDIX G

REVISED INTAKE LEVEE VICINITY OF POWERHOUSE

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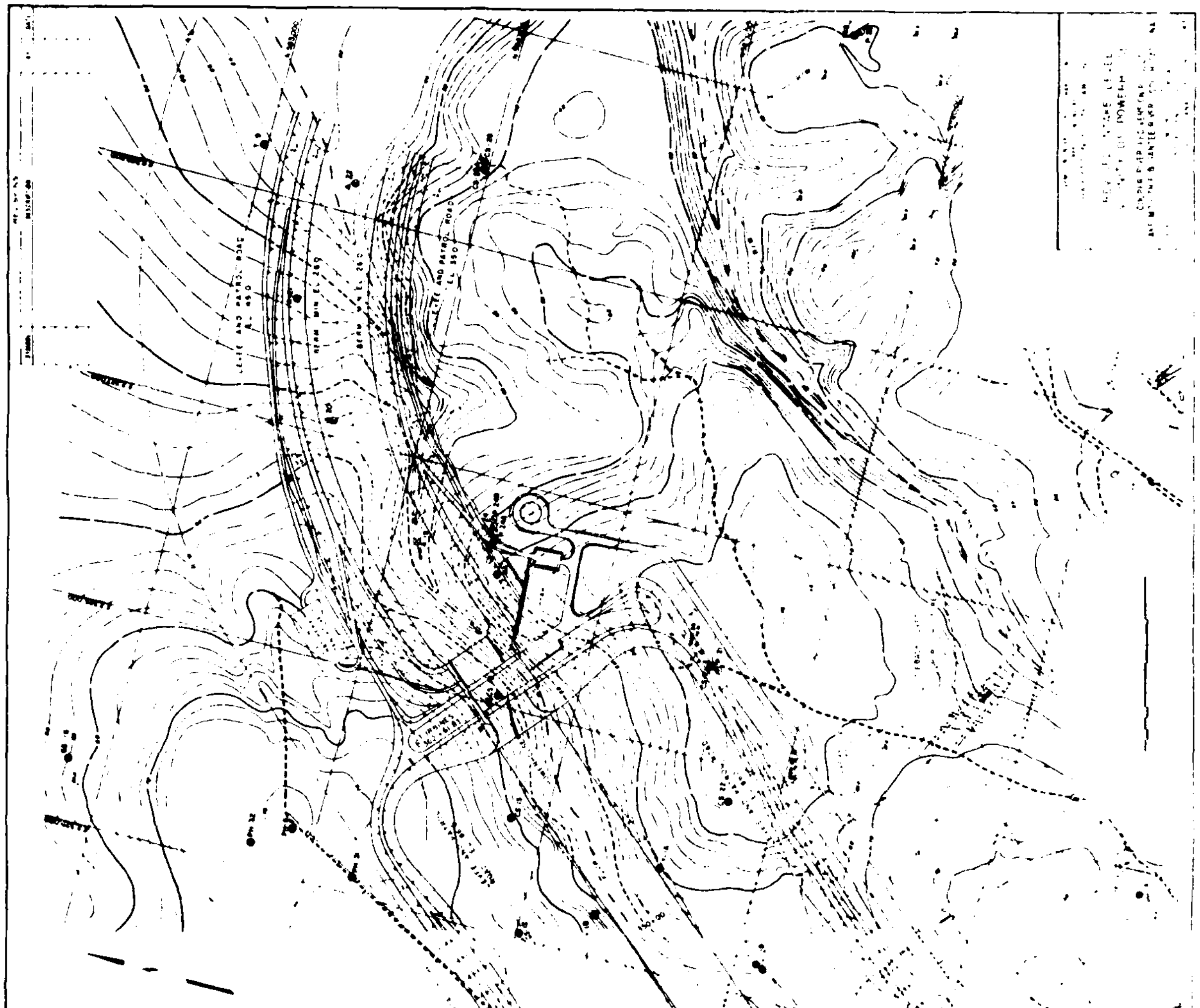
COOPER RIVER REDIVERSION PROJECT
INTAKE AND TAILRACE CANALS

APPENDIX G

REVISED INTAKE CANAL VICINITY OF POWERHOUSE

1. General. The intake canal revisions are shown on Plate G-1 for information in this report. These changes are to be formally presented in revisions to the Powerhouse Foundation Analysis Report, February 1976, since construction of a major portion of this area of the canal will be included in the powerhouse contract.

2. Summary of changes. It was proposed to site the levees just upstream of the powerhouse on higher ground, resulting in a wider intake canal. This alternate scheme would use less material provide the same or better stability, cost less, and would not affect other phases of the project unfavorably.



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