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VOLUME I OF II

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN

DETAIL DESIGN MEMORANDUM NO. 6
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

Prepared For

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

By

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS
NEW ORLEANS, LOUISIANA

JULY 1972

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APPENDICES

PERTINENT DATA

LOCATION OF PROJECT

Southeastern Louisiana in the vicinity of New Orleans; along the east shore of Lake Pontchartrain between the Chef Menteur Pass and the Rigolets in Orleans and St. Tammany Parishes.

PURPOSE OF PROJECT

The project presented herein is a segment of the Lake Pontchartrain Barrier Plan and will serve to protect areas contiguous to the shore of Lake Pontchartrain from flooding by limiting the uncontrolled entry of hurricane tides into the lake.

CONTROL STRUCTURE

Roadway Bridge

Open steel grid and floor beams on prestressed concrete girders.	
Total length between abutments	1,100 feet
Roadway width	11 feet
Top elevation	14.0*

Crane Bridge

Reinforced concrete framework with 16 gate openings, 50 feet on centers	
52 vertical lift gates, three per slot plus 4 spares	
Two - 100 ton gantry cranes on 24' - 0" travel way	
Top elevation (rail)	14.00

*Unless otherwise specified, all elevations herein are in feet and refer to mean sea level datum.

Foundation

Reinforced concrete slab footing supported by steel H - piles	
Sill elevation	-30.0
Bottom elevation	-38.0

APPROACH CHANNEL

Width at structure	800 feet
Maximum bottom width - Gulf side	1,590 feet
Maximum bottom width - Lake side	1,040 feet
Bottom elevation - Gulf side	-30.0
Bottom elevation - Lake side	-30.0
Side slopes	1 on 3

CLOSURE DAM

Earth filled with riprap slope protection	
Crown width	64 feet
Crest elevation	14.0
Side slopes	1 on 4

TIE-IN LEVEE

Earth filled embankment	
Crown width	20 feet
Side slopes	1 on 4
Crown elevation	9.0 to 14.0

RIGHTS-OF-WAY

Closure dam and control structure	135 acres
Levee	55 acres
Spoil disposal	60 acres

FIRST COST

Channels and canals	$ 264,000
Levees and floodwalls	7,236,200
Control structure	24,572,000
Engineering and design	3,047,000
Supervision and administration	2,084,800
Lands	184,000
Total	$37,388,000

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
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SECTION I - GENERAL

1. Authority. The Lake Pontchartrain Barrier is a part of the Lake Pontchartrain Barrier Plan feature of the project "Lake Pontchartrain and Vicinity, Louisiana", which is described in House Document No. 231, 89th Congress, 1st. Session, and in the "Interim Survey Report, Lake Pontchartrain, La. and Vicinity", dated 21 November 1962, as modified by LMNED-PP letter dated 13 March 1967 subject, "Lake Pontchartrain, La. and Vicinity - Evaluation of Alternate Plans Involving Modification in the Alignment of the Lake Pontchartrain Barrier", and indorsement thereto. The project was authorized by the following laws: Public Law 298-89th Congress, 1st Session, approved 27 October 1965; and 10 U.S.C. 2304 (a) (4). The report of the Chief of Engineers concurred in the recommendation of the Board of Engineers for Rivers and Harbors which provided for:

a. A gated control structure

b. An approach channel to the control structure

c. A closure dam

d. A tie-in levee

2. Purpose. This feature design memorandum presents the essential data, assumptions, and criteria used in the design

of the structural features of the project, and was prepared for the purpose of developing review of subsequent plans and specifications.

3. Project Location. The authorized project, shown on plate I-1 is located in southeastern Louisiana in the general vicinity of New Orleans. The project area comprises the low land and water areas between the Mississippi River alluvial ridge and the Pleistocene escarpment to the north and west. The dominant topographic feature is Lake Pontchartrain, a shallow landlocked tidal basin approximately 640 square miles in area and averaging 12 feet in depth. It connects with lesser Lake Maurepas to the west and through Lake Borgne and Mississippi Sound to the Gulf of Mexico to the east. The project is located in the Parishes of Orleans and St. Tammany.

4. Local Cooperation.

a. Local cooperation requirements. The conditions of local cooperation pertinent to the Lake Pontchartrain Barrier Plan, as specified in the report of the Board of Engineers for Rivers and Harbors and concurred in by the Chief of Engineers, are as follows:

> ". . . That the barrier plan for protection from hurricane floods of the shores of Lake Pontchartrain . . . be authorized for construction. . . Provided that prior to construction of each separable

independent feature local interests furnish assurance satisfactory to the Secretary of the Army that they will, without cost to the United States:

"(1) Provide all lands, easements, and rights-of-way, including borrow and spoil-disposal areas, necessary for construction of the project;

"(2) Accomplish all necessary alterations and relocations to roads, railroads, pipelines, cables, wharves, drainage structures and other facilities made necessary by the construction work.

"(3) Hold and save the United States free from damages due to the construction work;

"(4) Bear 30 percent of the first cost, to consist of the fair market value of the items listed in subparagraphs (1) and (2) above and a case contribution presently estimated at $14,384,000 for the Barrier Plan and $3,644,000 for the Chalmette Plan, to be paid either in a lump sum prior to initiation of construction or in installments at least annually in proportion to the Federal appropriation prior to start of pertinent work items, in accordance with construction schedules as required by the Chief of Engineers, or as a substitute for any part of the cast contribution, accomplish in accordance with approved construction schedules items of work of equivalent value as determined by the Chief of Engineers, the final apportionment of costs to be made after actual costs and values have been determined.

"(5) For the barrier plan, provide an additional cash contribution equivalent to the estimated capitalized value of operation and maintenance of the Rigolets navigation lock and channel to be undertaken by the United States, presently estimated at $4,092,000, said amount to be paid either in a lump sum prior to initiation of construction of the barrier or in installments at least annually in proportion to the Federal appropriation for construction of the barrier."

"(6) Provide all interior drainage and pumping plants required for reclamation and development of the protected areas;

"(7) Maintain and operate all features of the works in accordance with regulations prescribed by the Secretary of the Army, including levees, floodgates and approach channels, drainage structures, drainage ditches or canals, floodwalls, seawalls, and stoplog structures, but excluding the Rigolets navigation lock and channel and the modified dual-purpose Seabrook Lock; and

"(8) Acquire adequate easements or other interests in land to prevent encroachment on existing ponding areas unless substitute storage capacity or equivalent pumping capacity is provided promptly;

"Provided that construction of any of the separable independent features of the plan may be undertaken independently of the others, whenever funds for that purpose are available and the prescribed local cooperation has been provided . . . ".

b. Status of local cooperation. On 2 November 1965, the Governor of the State of Louisiana designated the State of Louisiana, Department of Public Works, as " . . . the agency to coordinate the efforts of local interests and to see that the local commitments are carried out promptly . . . "By State of Louisiana Executive Order dated 17 January 1966, the Board of Levee Commissioners of the Orleans Levee District was designated as the local agency to provide the required local cooperation for all portions of the "Lake Pontchartrain. La., and Vicinity", project in Orleans, Jefferson, St. Charles, and St. Tammany Parishes. Assurances covering all of the local cooperation required for the Lake Pontchartrain Barrier Plan were requested through the Department of Public Works from the Board of Levee Commissioners of the Orleans Levee District on 21 January 1966 and a satisfactory act of assurances, supported by a resolution of the Board of Levee Commissioners of the Orleans Levee District dated 28 July 1966, was approved and accepted on behalf of the United States on 10 October 1966. The principal officers currently responsible for the fulfillment of the conditions of local cooperation are as follows:

Mr. C. H. Downs, Director
State of Louisiana
Department of Public Works
Baton Rougs, Louisiana 70804

Mr. Edward N. Lennox, President
Board of Levee Commissioners
Orleans Levee District
Room 200, Wild Life and Fisheries Building
418 Royal Street
New Orleans, Louisiana 70130

c. Views of local interests. The Board of Levee Commissioners of the Orleans Levee District represents local interests and is in agreement with the general plan.

5. Previous Reports. For general information on the project and basic data, reference is made to General Design Memorandum No. 2, Supplement No. 1, Rigolets Control Structure Closure Dam and Adjoining Levels, Lake Pontchartrain Barrier Plan, dated March 1970, and approved by the Chief of Engineers on 10 November 1970.

6. Description. The project plan presented herein and indicated on plate I-2 consists of a control structure in

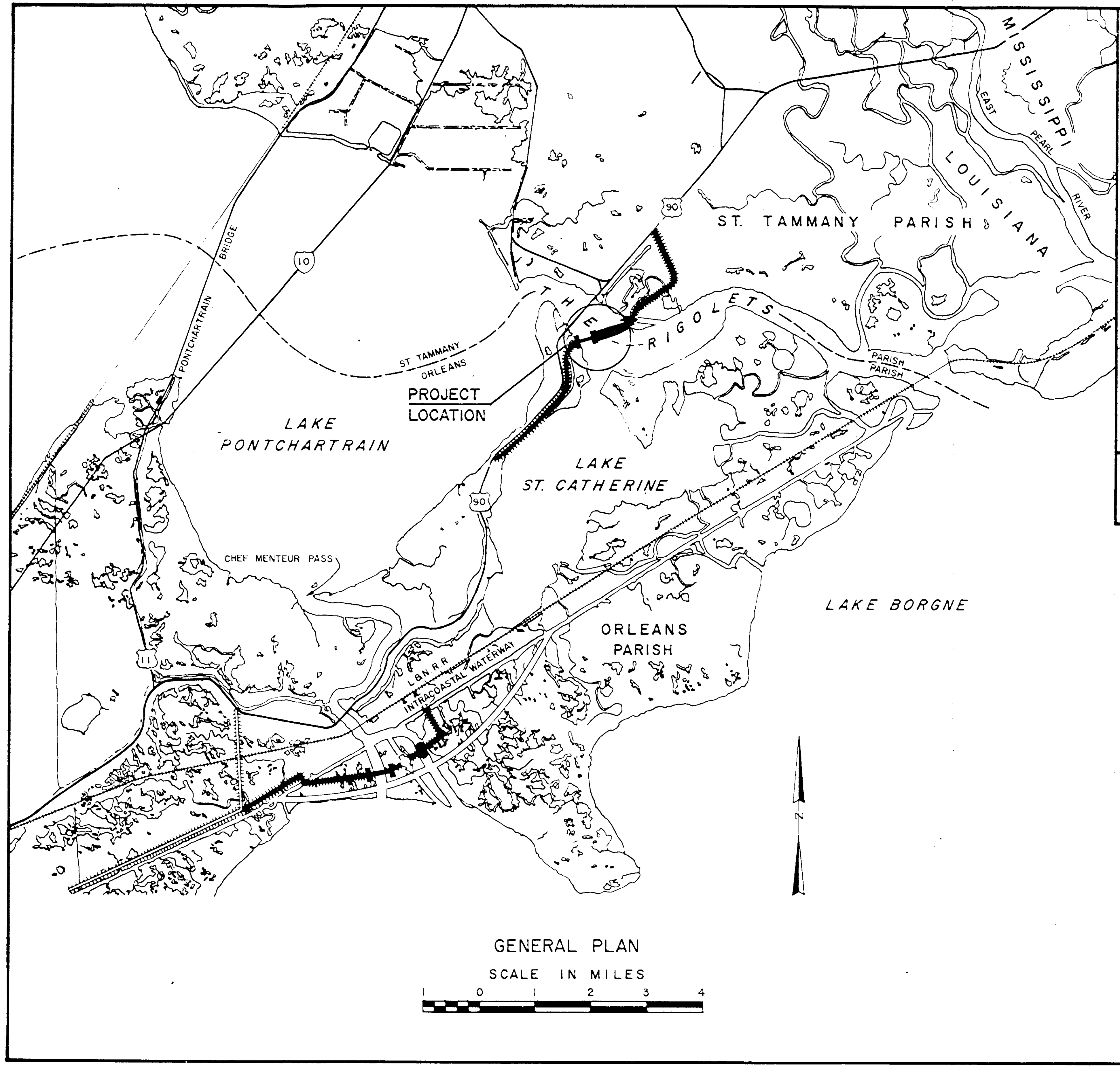
the Rigolets with approach channels, closure dam, and a tie-in levee embankment. The protective works are located between a point approximately 3.3 miles west of the west abutment of the existing bridge crossing at the Rigolets and a point approximately 500 feet west of the authorized Rigolets Lock. The flood protective works presented herein comprise a major feature of the Lake Pontchartrain Barrier Plan which provides for construction of a hurricane barrier along the east side of Lake Pontchartrain to limit uncontrolled ingress of hurricane tides into the Lake. The protective works are:

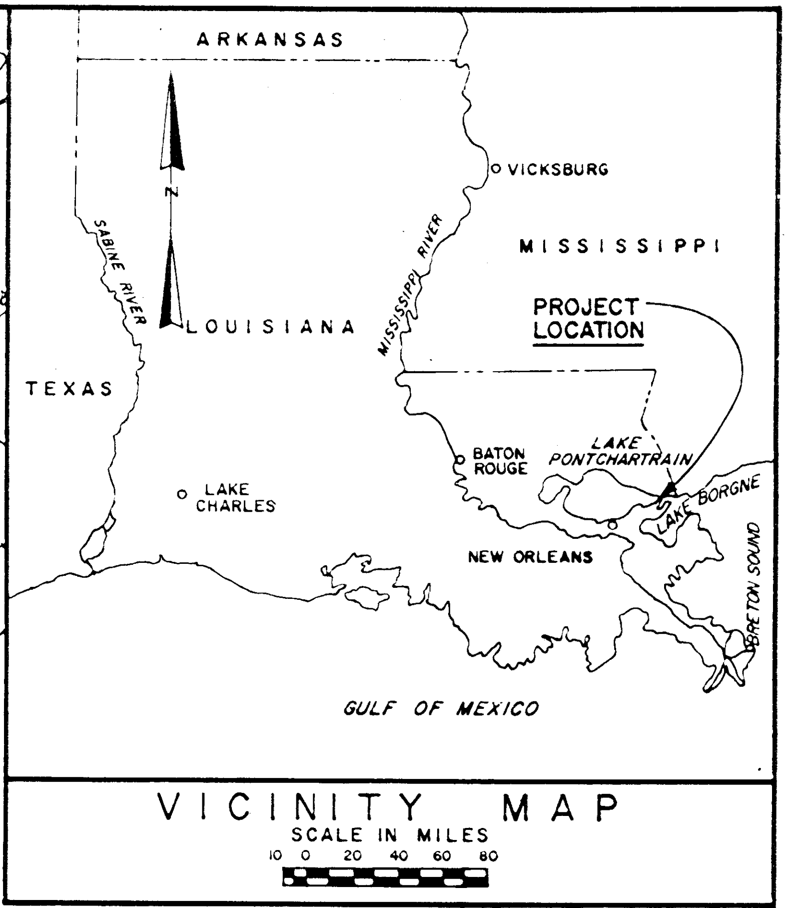
a. Control Structure. The gated control structure, as shown on plate I-3 is 1,100 feet long and 50 feet wide with the sill at elevation -30.0. The controlling elevation of the structure is 14.0.

b. Approach Channel. The approach channel to the control structure, as shown on plate I-4 will have a 800-foot bottom width at elevation -30.0 at the structure sill and will flare outward from the channel centerline at a 12.5 angle from each side of the structure. The elevation of -30.0 will be maintained for some 2,900 feet on the gulf side of the control structure and some 2,300 feet on the lake side. The channel side slopes will be 1 on 3 from the bottom of the channel to the surface of the ground, i.e., the existing bottom of the Rigolets Pass. The end slopes will be 1:10.

c. Closure Dam. The closure dam, as shown on plate IV-1 will be a hydraulic-filled structure with 1 on 4 side slopes and riprap slope protection. The closure dam 3,965 feet long with a crest elevation of 14.0.

d. Tie-in levee. The tie-in levee as shown on plate IV-3, is a protection levee that joins the connecting levee (not part of this design memorandum) and the west abutment of the control structure. The tie-in levee is 985 feet long with a controlling elevation varying from 9.0 to 14.0. The side slopes are 1 on 4.





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LAKE PONTCHARTRAIN BARRIER PLAN
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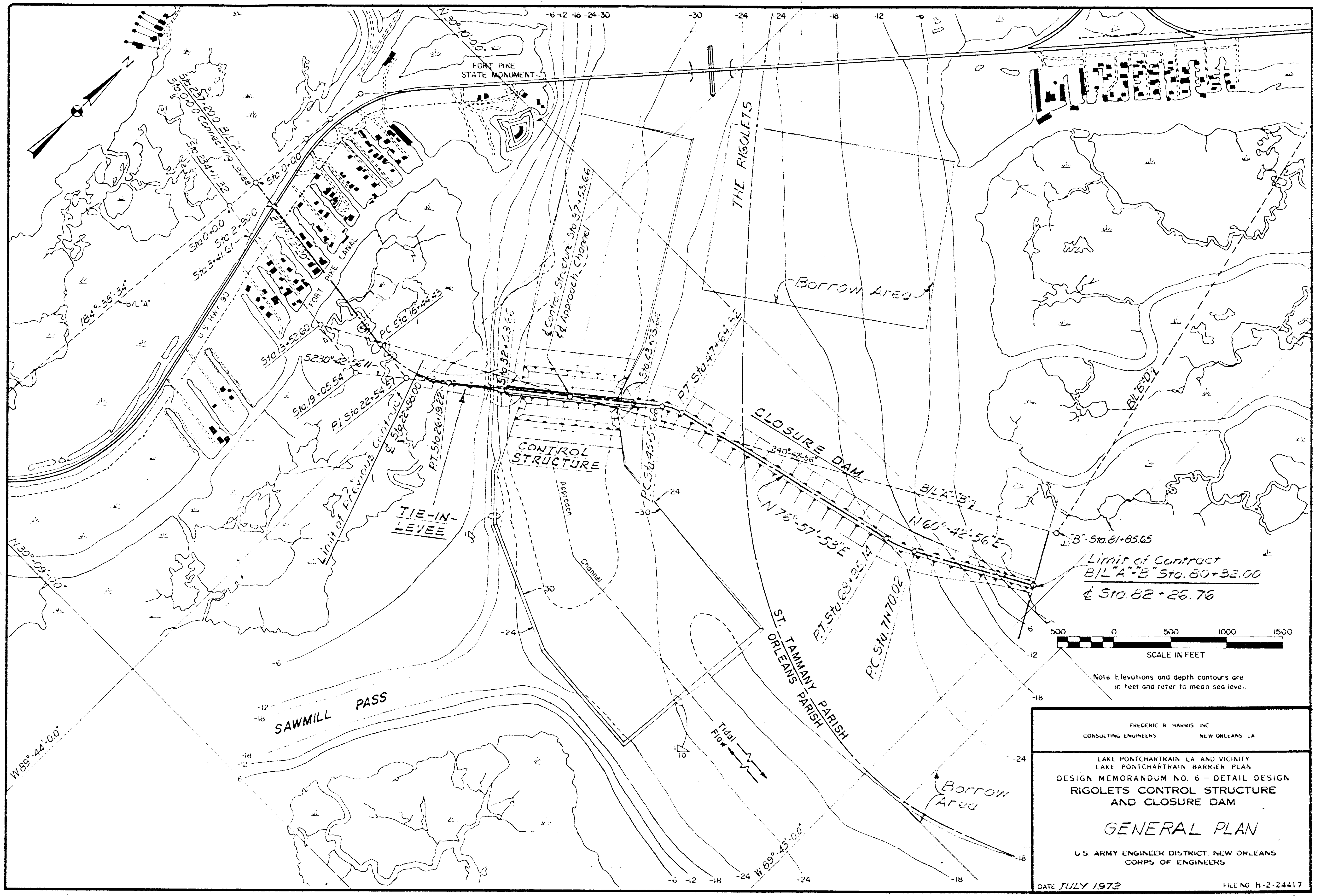
AND CLOSURE DAM

GENERAL PLAN AND VICINITY MAP

U S ARMY ENGINEER DISTRICT NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417

PLATE I-1


FORT PIKE
STATE MONUMENT
THE RIGOLETS
Borrow Area
FORT PIKE CANAL
U.S. HWY 90
CONTROL
STRUCTURE
CLOSURE DAM
TIE-IN-
LEVEE
SAWMILL PASS
ST. TAMMANY PARISH
ORLEANS PARISH
Tidal
Flow
Approach Channel
Limit of Contract
B/L "A"-"B" Sta. 80+32.00
℄ Sta. 82+26.76
SCALE IN FEET
Note: Elevations and depth contours are in feet and refer to mean sea level.
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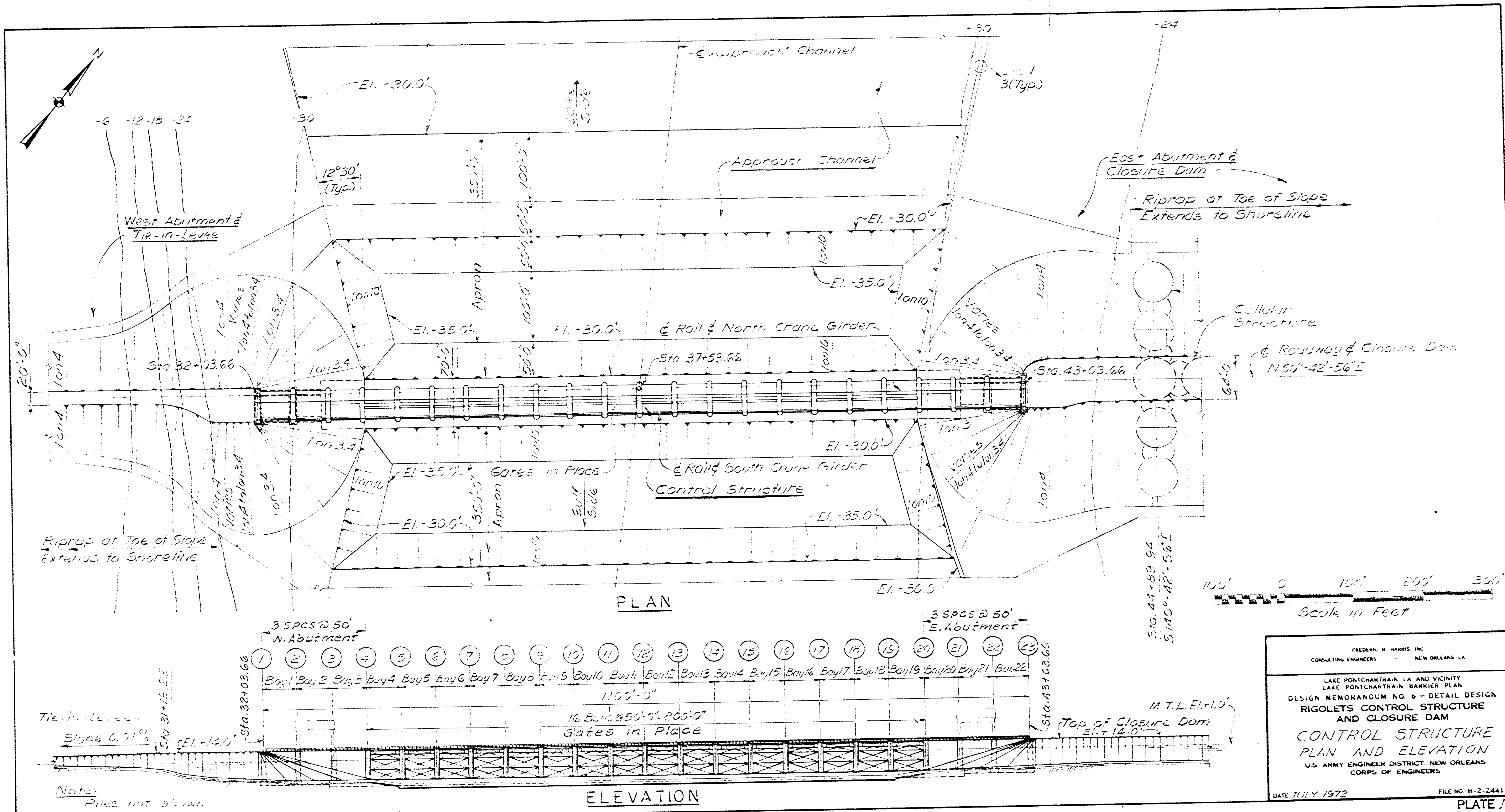
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE JULY 1972
FILE NO. H-2-24417

PLATE I-2



N
¢ Approach Channel
El. -30.0'
Approach Channel
12°30' (Typ.)
3 (Typ.)
West Abutment & Tie-in-Levee
East Abutment & Closure Dam
Riprop at Toe of Slope Extends to Shoreline
Apron
El. -35.0'
El. -30.0'
¢ Rail & North Crane Girder
¢ Rail & South Crane Girder
Sta 32+03.66
Sta 37+53.66
Sta. 43+03.66
Cellular Structure
¢ Roadway & Closure Dam N 50°-42'-56" E
Gates in Place
Control Structure
Varies 1 on 4 to 1 on 3.4
1 on 3.4
1 on 10
1 on 4
20'-0"
64'-0"
Sta. 44+89.94 S 140°-42'-56" E
Scale in Feet
PLAN
3 Spcs @ 50' W. Abutment
3 Spcs @ 50' E. Abutment
Bay 1 Bay 2 Bay 3 Bay 4 Bay 5 Bay 6 Bay 7 Bay 8 Bay 9 Bay 10 Bay 11 Bay 12 Bay 13 Bay 14 Bay 15 Bay 16 Bay 17 Bay 18 Bay 19 Bay 20 Bay 21 Bay 22
1100'-0"
16 Bays @ 50'-0" = 800'-0"
Gates in Place
Tie-in-Levee
Slope 0.01%
Sta. 31+19.22
El. +14.0'
Sta. 32+03.66
Sta. 43+03.66
Top of Closure Dam El. +14.0'
M.T.L. El. +1.0'
Note: Piles not shown.
ELEVATION
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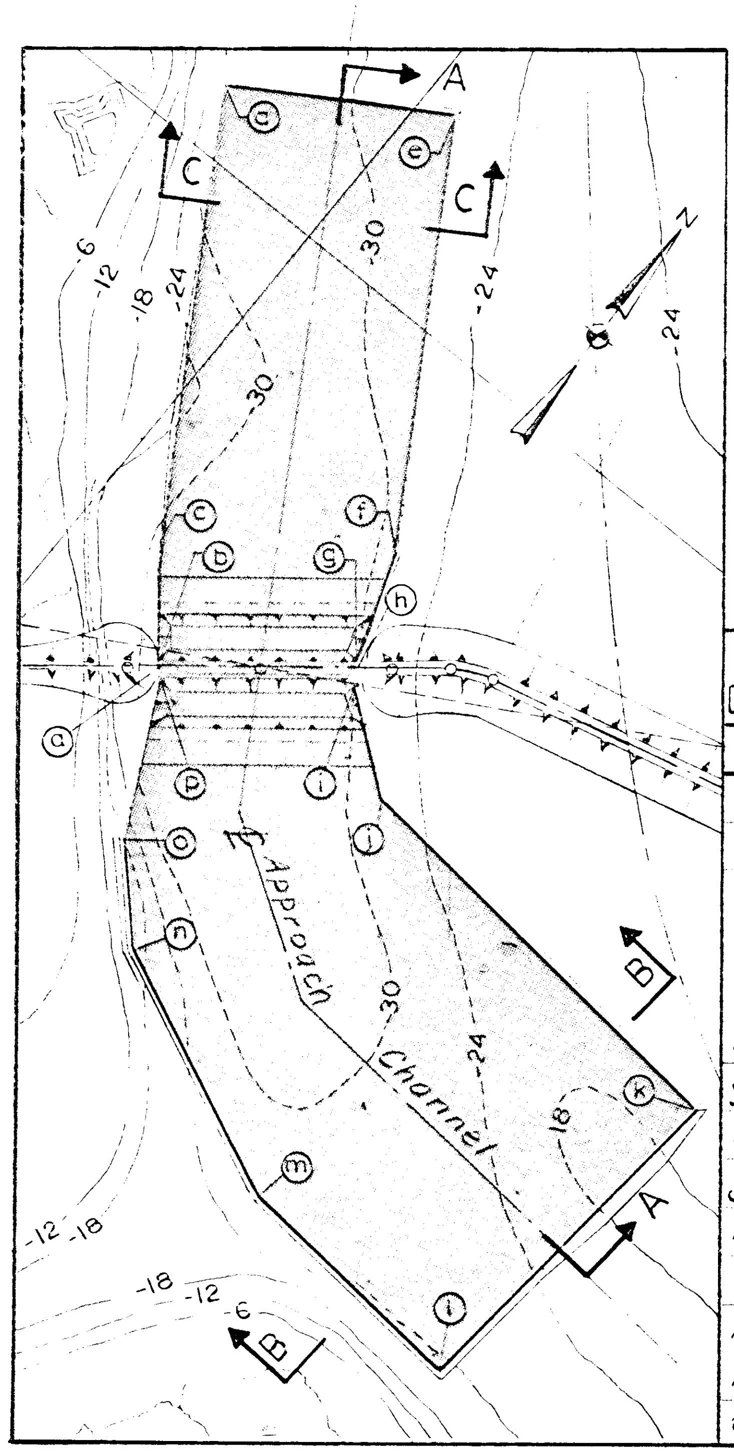
CONTROL STRUCTURE
PLAN AND ELEVATION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE JULY 1972
FILE NO. H-2-24417

PLATE I-3

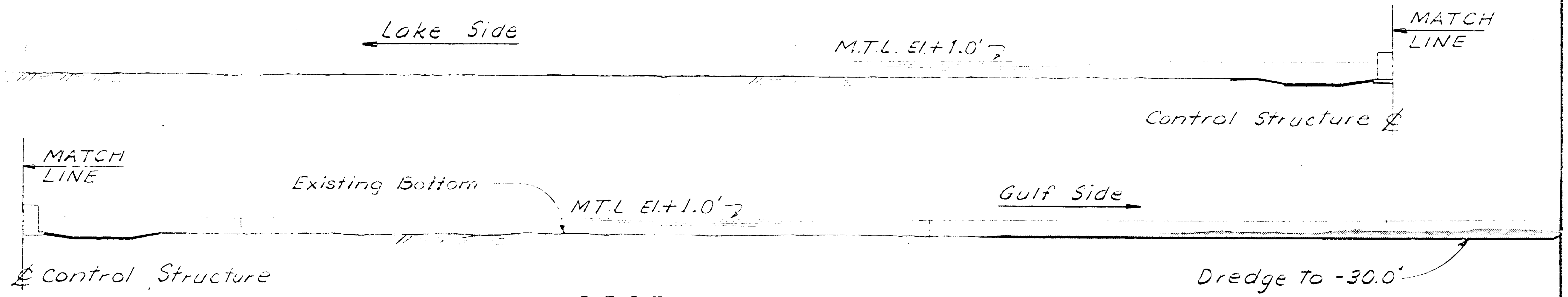


APPROACH CHANNEL DREDGE LIMITS DATA		
LINE	BEARING	DISTANCE
p - b	N39°-17'-04"W	70.00'
b - c	N 51°-47'-04"W	510.00'
c - d	N 29°-17'-04"W	2,040.00'
d - e	N60°-42'-56"E	1,040.00'
e - f	S 29°-17'-04"E	1,06[illegible].00'
f - g	S26°-47'-04"E	[illegible]
g - i	S39°-17'-04"E	70.00'
i - j	S 51°-47'-04"E	529.00'
j - k	S85°-47'-04"E	[illegible]
k - l	S04°-12'-56"W	1,590.00'
l - m	N85°-47'-04"W	1,084.00'
m - n	N64°-47'-04"W	1,178.50'
n - o	N 45°-47'-04"W	[illegible]
o - p	N26°-47'-04"W	59[illegible]

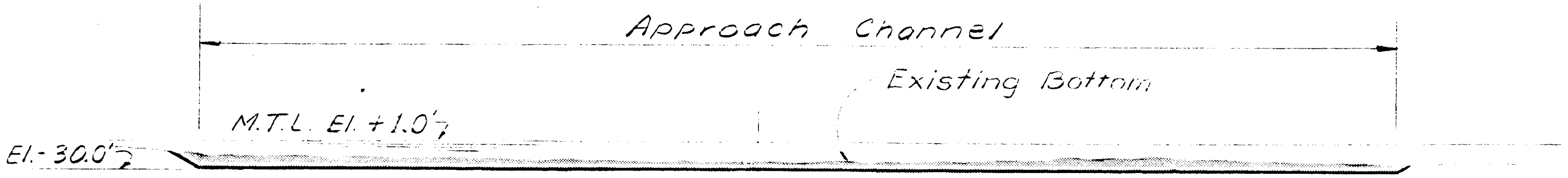
PLAN

500 0 500 1,000 1,500

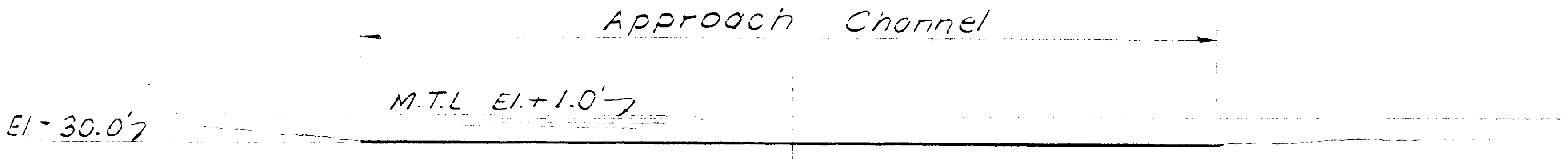
Scale in Feet



SECTION A-A



SECTION B-B



SECTION C-C

200 0 200 400 600

Scale in Feet

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PLAN AND SECTIONS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417

PLATE I-4

SECTION II - HYDRAULIC DESIGN

1. General. Detail hydrologic and hydraulic analyses and data are presented in "Lake Pontchartrain, Louisiana and Vicinity, Design Memorandum No. 1, Hydrology and Hydraulic Analysis, Part I - Chalmette, Part II - Barrier, and Part III - Lakeshore", dated 7 August 1967, and "Interim Survey Report, Lake Pontchartrain, Louisiana and Vicinity," dated 21 November 1962.

2. Design Criteria.

a. Design Hurricane.

(1) Tracks. The standard project hurricane (SPH) applicable to the design of the control structure is a synthetic storm occurring along Tracks A, C, and F with design hurricane characteristics shown in Table II-1.

TABLE II-1

DESIGN HURRICANE CHARACTERISTICS

Track*	C.P.I. inches	Radius of maximum winds nautical miles	Forward speed knots	Max.** wind m.p.h.	Direction of approach
A	27.6	30	6	100	South
C	27.6	30	5	100	SSE
F	27.6	30	11	100	East

*Tracks are shown on Figure II-1.
*Referenced to 30 feet above the surface.

(2) Parameters. The parameters for certain synthetic storms and methods for derivation of others were furnished by the U.S. Weather Bureau. Ther derivation of the hurricanes along Tracks A, C, and F are discussed in paragraph 8.c or Part I - Chalmette, DM No. 1.

b. Differential Heads.

(1) Normal conditions. Under normal conditions there is no differential head in the vicinity of the control structure or closure dam because the gates are stored above the water surface.

(2) Hurricane conditions. For hurricane conditions the various differential heads are determined primarily by the water surface elevations given in Table II-2. The differential heads 13.5 ft. for Case 2 and 13.0 ft. for Case 5, are considered to be of long enough duration to cause minor seepage under the control structure and negligible drawdown in the closure dam. The represent static loading conditions for the design of the gates and foundations. The differential heads of 15.8 ft. for Case 1 and 14.5 ft. for Case 4 are associated with hurricane waves and therefore, represent major loading conditions for the gates and foundations. The water elevations indicated are stillwater levels (SWL) which are "the elevation of the water surface if all wave action were to cease". The SWL is an elevation which is basic in determining the elevation a wave will attain in the vicinity of the barrier. It is expected that a standing wave will occur when a wave train is interrupted by the vertical wall

presented by the gates and a clapotis will form to some height which then becomes the basis for the development of a Sainflou pressure diagram. This diagram, in conjunction with the lower water level on the lee side of the gates, is then assumed to be the dynamic "differential head" for the design of the structure. It is assumed that no significant seepage occurs during the relatively short period of time that waves are in the vicinity of the structure. The SWL (Cases 1 and 4) is used as a differential head in the design of the pile foundation mainly to relate the dynamic change in reactions due to wave forces with a (assumed) static loading condition. The differential heads indicated in Cases 3 and 6 are 4.0 feet and 5.5 feet respectively. These differential heads are utilized during the placement of the gates in order to determine scour protection in the approach channel and to estimate the forces acting on the gates as they are being placed. The "long term" static head differentials occurring during hurricane conditions are shown in Cases 2 and 5. In Case 2, the Gulfside still water level (SWL) is +9.0 feet with the Lakeside SWL at -4.5 feet resulting in a differential head of 13.5 feet. In Case 5, the Lakeside SWL is +9.0 feet and the Gulfside SWL is -4.0 feet resulting in a differential head of 13.0 feet. These static head differentials are considered to be "long term" when compared with the "short term" head differentials associated with wave conditions. Cases 2 and 5 are primarily significant with respect

TABLE II-2
RIGOLETS COMPLEX
LAKE PONTCHARTAIN BARRIER PLAN
DESIGN DATA

CASE	WATER ELEVATIONS		STRUCTURAL DESIGN ALLOWABLE STRESS $\frac{\text{STEEL}}{\text{CONCRETE}}$		REMARKS
	GULFSIDE	LAKESIDE	DL+WL	DL+WL+WAVE	
1	+12.8	-3.0		$\frac{0.67\ F_y}{0.45\ f'_c}$	HURRICANE CONDITION — INCREASE ALLOWABLE STRESSES
2	+9.0	-4.5	$\frac{0.5\ F_y}{0.35\ f'_c}$		HURRICANE CONDITION — NORMAL ALLOWABLE STRESSES
3	+5.0	+2.5	$\frac{0.5\ F_y}{0.35\ f'_c}$		MAXIMUM DIRECT HEAD UNDER WHICH GATE WILL OPERATE – FOR MACHINERY DESIGN
4	-3.0	+11.5		$\frac{0.67\ F_y}{0.45\ f'_c}$	HURRICANE CONDITION — INCREASE ALLOWABLE STRESSES
5	-4.0	+9.0	$\frac{0.5\ F_y}{0.35\ f'_c}$		HURRICANE CONDITION — NORMAL ALLOWABLE STRESSES
6	-3.0	+2.5	$\frac{0.5\ F_y}{0.35\ f'_c}$		MAXIMUM DIRECT HEAD UNDER WHICH GATE WILL OPERATE – FOR MACHINERY DESIGN
7	+1.5	+1.5	$\frac{0.5\ F_y}{0.35\ f'_c}$		NORMAL WATER LEVEL

DL = DEAD LOAD WL = WATER LOAD

to seepage under the control structure and drawdown along the closure dam; both seepage and drawdown are considered to be minor due to the extensive use of stone protection for more severe loading conditions.

c. Waves.

(1) Parameters. The data given in Table II-3 below indicates the parameters which were used to determine hurricane wave characteristics.

TABLE II-3

DESIGN PARAMETERS FOR WAVES

Track*	C.P.I. inches	Radius of maximum winds nautical miles	Forward speed knots	Max** wind m.p.h.	Direction of approach
A	27.6	30	6	100	South
C	27.6	30	5	100	SSE
F	27.6	30	11	100	East

*Referenced to 30 feet above the surface

		Lake Borgne side	Lake Pontchartrain side
F	- Length of fetch (mi.)	5	2
U	- Windspeed (m.p.h.) *	90	80
swl	- Stillwater level (ft., m.s.l.)	12.8	11.5
d	- Average depth of fetch (ft.)	13.8	30.0

*Represents a 5-minute average referenced to 30 feet above the boundary surface.

(2) Characteristics. The wave characteristics are given in Table II-4 for waves from the Lake Borgne (Gulfside)

and from Lake Pontchartrain (Lakeside). The wave characteristics are modified in the vicinity of the structure for the actual depth of water into which the wave is traveling.

TABLE II-4

WAVE CHARACTERISTICS - DESIGN HURRICANE

		Lake Borgne	Lake Pontchartrain
H_s	- Significant wave height (ft.)	5.8	5.85
T	- Wave period (sec.)	5.8	7.75
L_o	- Deepwater wave length (ft.)	172.0	308.0
d/L_o	- Relative depth	0.08023	0.09740
H_s/H'_o	- Shoaling coefficient	0.9545	0.9349
H'_o	- Deepwater wave height (ft)	6.08	6.26
H'_o/T^2	- Wave steepness	0.181	0.104
d_b	- H'_o breaking depth (ft.)	7.19	8.87
H_b	- Wave height on breaking (ft.)	5.61	6.93
H_{10}	- Average of highest 10% of all waves (ft.)	7.37	7.43
H_1	- Average of highest 1% of all waves (ft.)	9.70	9.77

The wave characteristics given for Lakeside are less severe than those presented in GDM No. 2, Supplement No. 1 and reflect the relocation of the control structure from its initial location along highway U.S. 90. The revised wave characteristics results in substantial reductions in the wave forces from Lakeside.

d. Hydraulic loading conditions.

(1) Static Loads. What may be assumed to be relatively " long duration" static loading conditions are shown in Table II-2 for Cases 2 and 5. These water levels include astronomic tide effects, barometric pressure effects, wind setup, and rain; waves have not yet reached the area. In Case 2 the water level Gulfside is at 9.0 ft. and the water level Lakeside is at -4.5 ft. The differential head is 13.5 ft. which results in a net force of about 28 kips per foot to elevation -30.0 ft. The force will vary slightly depending on the actual density of the water which is basically brackish in nature. Consequently, a net horizontal force of about 1,400 kips act on each 50 ft. gate bay in the control structure. The force varies along the closure dam depending on the depth of water. In Case 5 the water level Lakeside is 9.0 ft. and the water level Gulfside is -4.0 ft. The net horizontal force is about 27 kips per foot which amounts to some 1,350 kips per gate bay. Since there is no essential difference in the static head differentials from either Gulfside or Lakeside any design governed by these loading conditions will be symmetrical, and the "long term" effect could have some effect on seepage.
The differential water levels in Cases 1 and 4 are related to wave trains which are expected to occur from Gulfside and Lakeside as

shown in Table II-4. These water levels are not true static head differentials in that they have to do with wave action; however, they are assumed to be static heads for the purpose of determining base pressures and pile loads in order to compare the effect of waves and wave forces on the foundation of the gated portion of the control structure. In Cases 3 and 6 the head differentials are used to determine flow velocities through the gated portion of the control structure as the gates are being placed into position. The velocities influence the weight of the gates and the frictional forces to contend with in the placement of the gates.

(2) Dynamic Loads. The dynamic loads acting on the structure due to wave action will be some combination of forces which are not readily determinable. As a wave train approaches the barrier, the deep water wave characteristics will be altered by the reduction in water depth; new wave properties are determined and applied to the various parts of the structure. As a nonbreaking wave train impinges on the vertical gated portion of the control structure, the gates prevent the waves from freely passing by. This causes each wave to form a standing wave which will rise to about double its normal height and is called a clapotis. This loading condition is approximated by a Sainflou pressure diagram which attempts to account for the dynamic effect of the rising wave by a modified static pressure

diagram. The Sainflou diagram for Cases 1 and 4 are shown on Plates V-5 and V-6. The actual height to which a wave will rise has been assumed to be $2H_1$, for the purpose of estimating a theoretical upper limit of wave height and wave action. The normal wave range for an H_1 wave from trough to crest is 9.7 feet for Hs = 5.8 feet; the maximum estimated range for the same H_1 wave forming a maximum clapotis is in the order of 19.4 feet. The wave profiles for Cases 1 and 4 show the estimated wave shapes for H_s, H_{10}, and H waves and a similar wave profile assuming the formation of a clapotis. The Sainflou diagrams are based on the assumption of a $2H_1$ wave. (See Figures II-2 thru II-7.)

The wave celerity gives the wave front forward motion, and while, normally there is very little water transport involved with wave motion, the fact that most of the wave will be above elevation +14.0 feet, which is the top of the gates, while the water level on the opposite side of the gates will be at -3.0 feet will result in a minimum drop of 17 feet permitting the wave to overtop the gates. The water overtopping the gates will be trapped between the concrete piers which have a clear opening of 44 feet. Air is displaced by the cascading water, consequently there will probably be a small reduction in air pressure on the downstream side of the gates which results in an increase in load to the gates. At the same time there will be considerable air turbulence as the air rushes in to fill the void caused by the cascading water. As the waves overtop the gates, there will be an additional drag force along the top of the gates at elevation +14.0 feet. As the wave train approaches the control structure from the Gulf side it must pass a substantial (5 ft. diameter) crane girder, and a wave train approaching from the Lake side must pass two (5 ft. diameter) girders and a roadway. These obstructions will cause considerable turbulence, as a wave engulfs them. In addition, the girder on the Gulf side on the girders and roadway on the Lake side, will also obstruct the attempt of the waves impinging on the vertical gate surface to form a clapotis; hence, there is resistance (and turbulence) acting in two directions almost simultaneously as each wave arrives at the gates. For assumptions and development of wave forces against the girders and roadway grating, see Figures II-8 thru II-41.

3. <u>Sequence of Gate Operation.</u>

a. Closing. The mathematical computer model was used to determine whether or not one sequence of placing the gates during closing operations would be more desirable than another. The conclusion of the study was that the individual gated bays were so small in comparison to the size of Lake Pontchartrain and Lake Borgne (the Gulf) that the velocity through the openings would be constant. Consequently, the sequence for the placement of gates is essentially a matter of logic.

While it is true that the details of the design of the gates, gate slots, and gantry cranes are such that all 48 gates can be placed in a 12 hour period by a single crane, it is evident that there will be very little margin for things to go wrong and still meet this schedule as can be seen in Section VI, Mechanical Design. First, let it be assumed that two cranes are available (and desirable) to set the gates. Since the weather disturbance will be some distance away at the start of operations, it may be approaching rapidly by the time the last gates are being placed at which time rain, wind, poor visibility, and some wave action may make operations hazardous. It is, therefore, considered to be prudent for the two cranes to proceed towards the middle of the structure immediately and place three gates in each bay as they work their way towards their individual tie-down areas on each abutment. In this manner the time required to tie-down the cranes will be kept to a minimum at a time when weather conditions are becoming more adverse. Since two cranes cannot operate next to each other, it would be logical for (say) the west crane to place gates in Bay 11-12 while the east crane places gates in Bay 13-14, then when the west crane moves to Bay 10-11 the east crane can move to Bay 12-13 thence to Bay 14-15. If only one crane is operable it would be advisable for it to start at the opposite end of the structure in order to avoid needless travel to its tie-down position under adverse weather conditions. As explained previously, the sequence of placing gates will not have any noticeable effect on the hydraulics in the Rigolets. It is recommended that consideration be given to providing a safe haven in each crane for the operating personnel. There is a serious doubt whether these personnel will be able to leave the area when the gates have been placed, consequently, since the cranes will be the safest place to be, secure against wind and a cab area above the waves, it could provide a refuge. Included in each crane should be some basic supplies including first aid, food, water, bunks, and emergency lighting. Personnel should be brought to the site by one vehicle which may leave the area when it is endangered by overtopping waves or high winds. It is not impossible that the vehicle could be required to return to a safe area before all of the gates have been set in which case, a safe haven in the cranes will be of the utmost urgency.

b. Reopening. Hydraulically there is no preferred sequence to the removal of the gates. While there may be some urgency to remove the gates in order to alleviate potential flooding conditions in Lake Pontchartrain, it probably is not quite the same as during the approach of a hurricane. After the passage of a hurricane there may be some very real problems affecting the sequence of removal of gates such as damage to gates, cranes, girders, etc. It is possible that foreigh objects are trapped in a manner which will prevent the removal of some gates and/or possibly the ability of a crane to navigate along the craneway. It would seem to be expedient to provide an external boom at the landside end of each crane to remove any debris which would hamper operations.

4. Velocities and Scour.

a. Approach Channels.

(1) Normal conditions. Under normal conditions the tidal range in Lake Pontchartrain is 0.5 foot and 1.0 foot in Lake Borgne. Lake Pontchartrain has an average elevation of 1.0 foot and Lake Borgne has an average elevation of 0.9 foot. All elevations are referenced to Mean Sea Level (MSL) datum unless otherwise noted. The term Mean Tide Level (M.T.L.) is used to indicate the average normal water level in the Rigolets. Under normal conditions the control structure will be open and it is expected that the normal tidal action will produce a current velocity of 4.6 fps through the gate openings. The normal velocity at the

site prior to construction, is about 2 fps; after construction, this velocity will be achieved by diffusion at a distance of approximately 1,350 feet when the 800 feet of control structure is open and a tidal current of 4.6 fps is passing through it. This distance is about 900 feet before Fort Pike consequently the control structure and closure dam will not have a scour effect on either Fort Pike or highway U.S. 90 which is about 3,000 feet away. Plate II-1 shows the flow velocities in the area before construction and Plate II-2 shows the flow velocities after construction.

(2) Hurricane conditions. When hurricane conditions result in an alert to the Rigolets control structure and an order to place the gates into position to close off the Rigolets Cases 3 and 6 have been given as the basic water elevations to be used to determine flow velocities through the control structure and the related forces acting on the gates during this operation. A mathematical computer model was developed to determine the flow velocities through the control structure under the circumstances given, but also to determine the flow velocities in the area of the closure dam which will occur during its construction. The flow velocities affecting the construction of the closure dam were based on two methods of construction, the first being to build the dam from the east abutment towards the shore, and the second being to build the dam in layers along its entire length. Although the anticipated reuse of the steel cells (which will form the cofferdam for the construction

of the control structure) as a basic part of the construction procedure for the closure dam will obviate the second method of construction, the data is included in order to permit a comparison of the flow velocities related to each method of construction. A horizontal grid system was devised for the model study which is shown in Plate II-1; the velocity from one segment to another is shown on the various typical computer runs included in the appendix. Flow velocities determined for the control structure for Cases 3 and 6, are shown in Plates II-3 and II-4. Flow velocities at Control Structure during gate closure are shown on Figure II-46. In Case 3 the indicated water levels are +5.0 ft. Gulfside and +1.0 ft. Lakeside which results in a flow velocity of 10.7 fps through the control structure. In Case 6 the indicated water levels are -3.0 ft. Gulfside and +2.5 ft. Lakeside which results in a flow velocity of 12.8 fps through the control structure. Since there are no appreciable differences in the flow velocities from either side of the structure during the placement of gates, the scour protection for the approach channel on each side is the same.

b. Through the Control Structure.

(1) Normal conditions. Under normal conditions the flow through the control structure is about 4.6 fps. Since this velocity is considerably less than the velocities encountered during the closing of the structure, it will not influence the scour protection in the approach channels.

(2) Closing conditions. During closing condition the velocities which occur were mentioned previously as being 10.7 fps for Case 3 and 12.8 fps for Case 6. These velocities are basic in the determination of scour protection in the approach channels on each side of the control structure. The sill elevation of the base slab is -30.0 ft. The rigolets bottom then slopes downward and away from the sill for 50 ft. at a slope of 1:10 to elevation -35.0 ft. where it remains level for 100 ft. and then slopes upward at a 1:10 slope for another 50 ft. to elevation -30.0 ft. The pocket thus created extends a distance of 185 ft. from the center of the structure at which point the average velocity is estimated to be about 5.25 fps. In this zone there is a 4.0 ft. riprap set on a one foot bed of shell with an average stone diameter of 36 inches. From 185 ft. to 285 ft. from the center of the control structure it is estimated that the velocity has reduced to about 4.0 fps and there is a 2.5 ft. thick riprap protection with an average diameter of 24 inches. From 285 ft. to 385 ft. from the center of the control structure the velocity has reduced to about 3.2 fps and an 18 inch depth of riprap with an average diameter of 13 inches is used for scour protection. The scour protection in the approach channel Apron is shown in Plates II-5 & 6. The cellular cofferdam which will be used for the construction of the control structure has been located in a manner which will permit the placement of riprap in the dry. See Section III, Method of Construction.

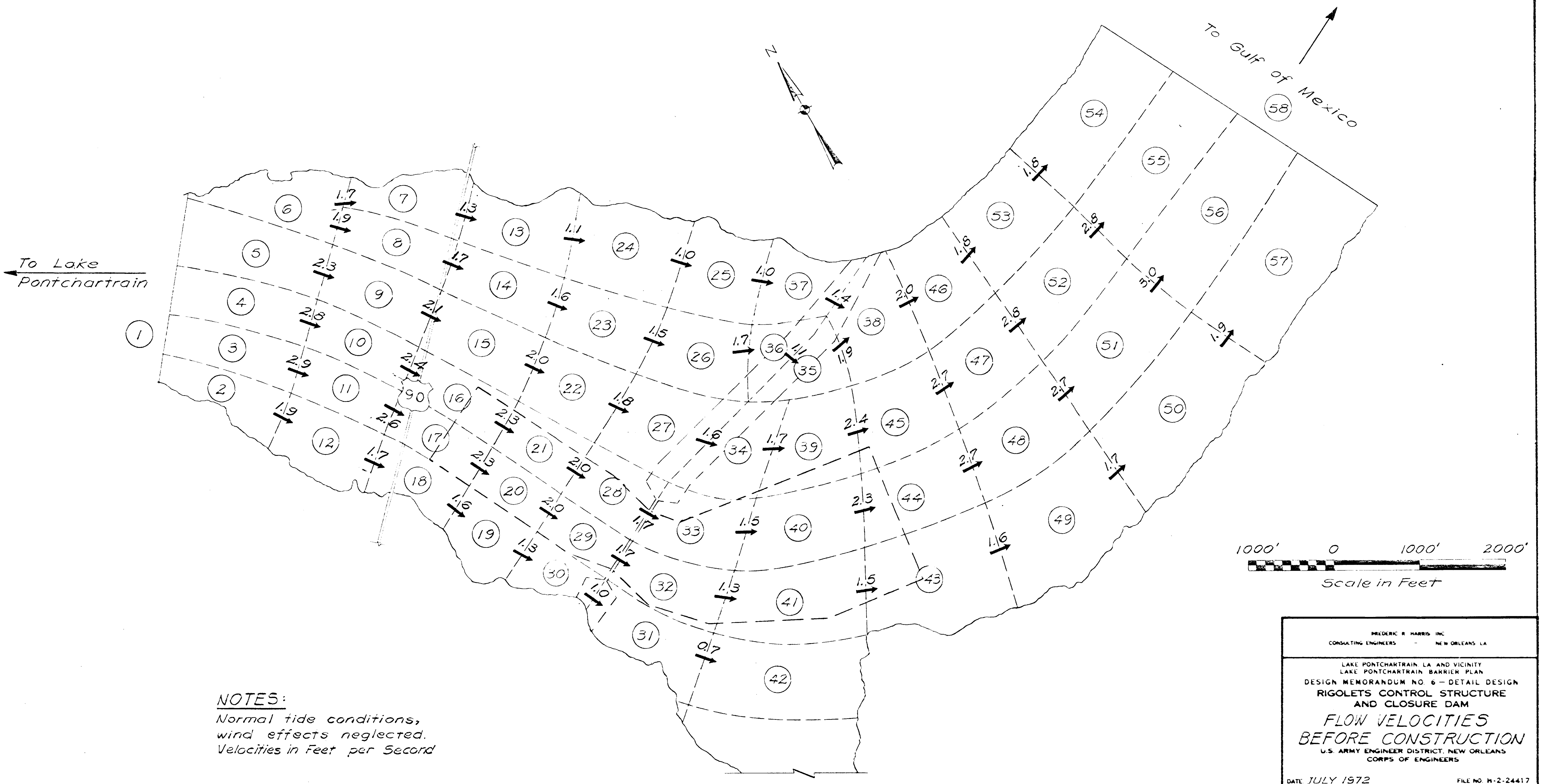
(3) Reopening conditions. The conditions for reopening of the control structure are not established. It is probable, however, that the velocities through the control structure will not be as severe as those used during the closing of the structure.

(4) Hurricane conditions. During hurricane conditions there is actually no flow through the control structure. There will be, however, an overtopping of the gates by waves which will cause severe turbulence on the downstream side of the gates. An attempt has been made to evaluate this condition and the conclusion is that the depth of tailwater is more than adequate to dissipate the energy of the overfall and not require additional protection at the toe of the sill.

c. Turbulence over and under the gates in closing. It appears from the mathematical computer model study made that the relative areas of Lake Pontchartrain and Lake Borgne (and Mississippi Sound) are so large that the closing of the control structure will not cause any noticeable tailwater condition; consequently, the velocities through the control structure will remain fairly constant depending only on the relative heads between the adjoining bodies of water. Therefore, it is felt that turbulence will not be a serious difficulty in placing the gates. It has been recommended by the hydraulics consultant that a velocity head of 5 ft. acting on the underside of the gates be more than sufficient to allow for turbulence. This 5 ft. head has been added to the other forces acting on the gates during placement to be certain that the gates will be heavy enough to be set without difficulty.

5. Approach Channel Design. (See Figures II-43 thru II-60.)

6. Hydraulic Computations. (See Figures II-61 thru II-83.)



NOTES:
Normal tide conditions,
wind effects neglected.
Velocities in Feet per Second

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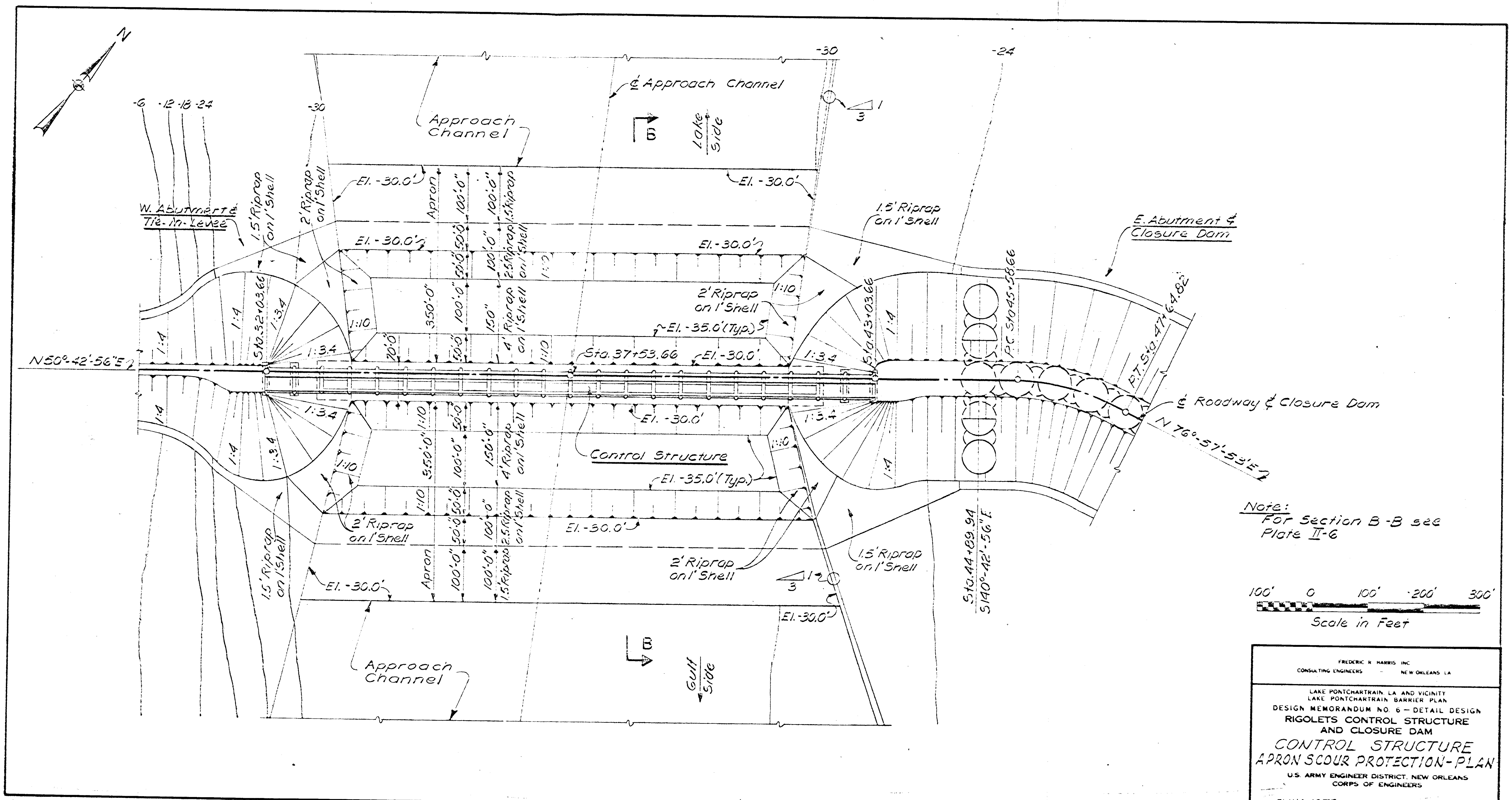
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

FLOW VELOCITIES
BEFORE CONSTRUCTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE II-1



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LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

CONTROL STRUCTURE
APRON SCOUR PROTECTION - PLAN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

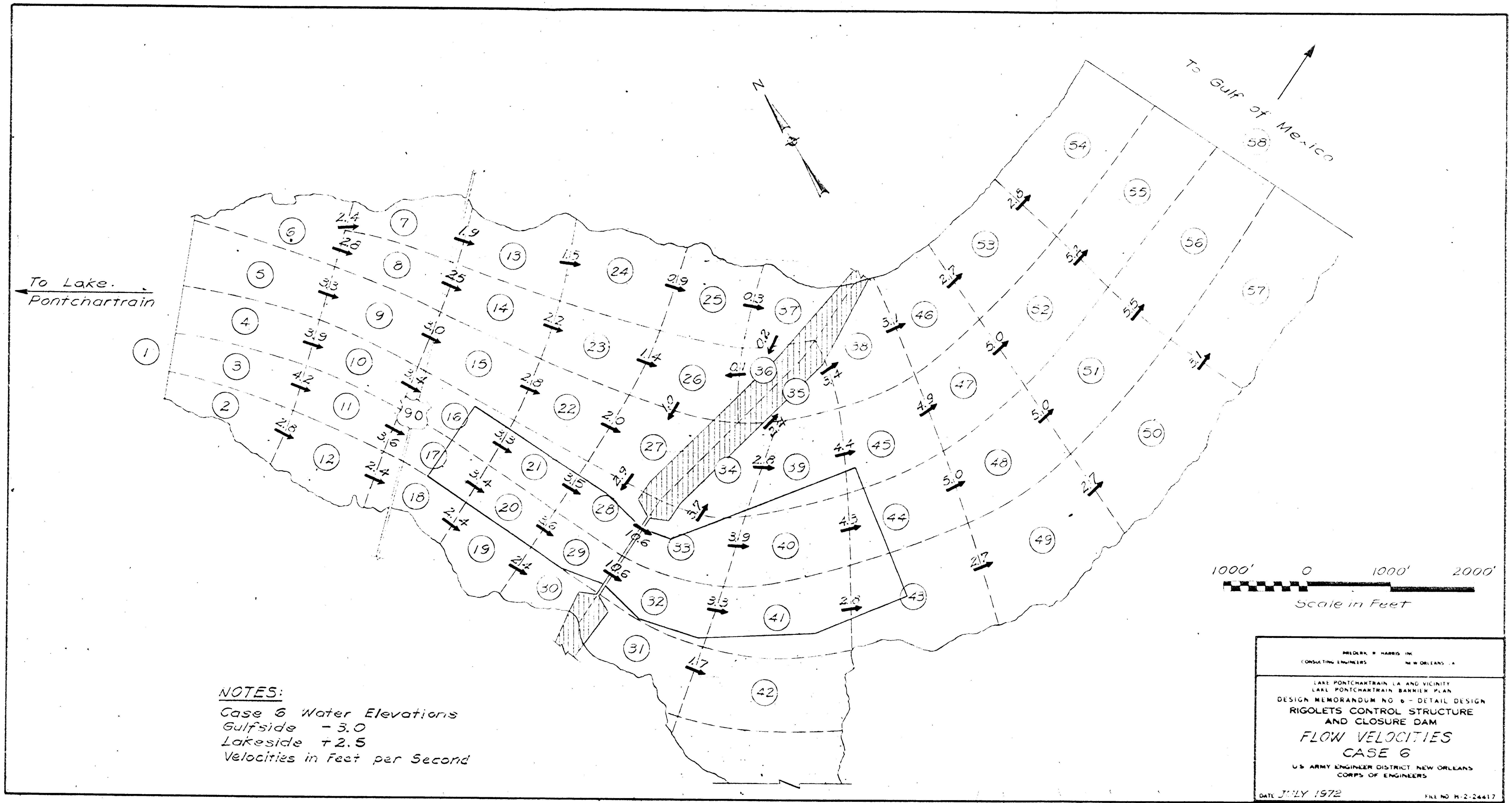
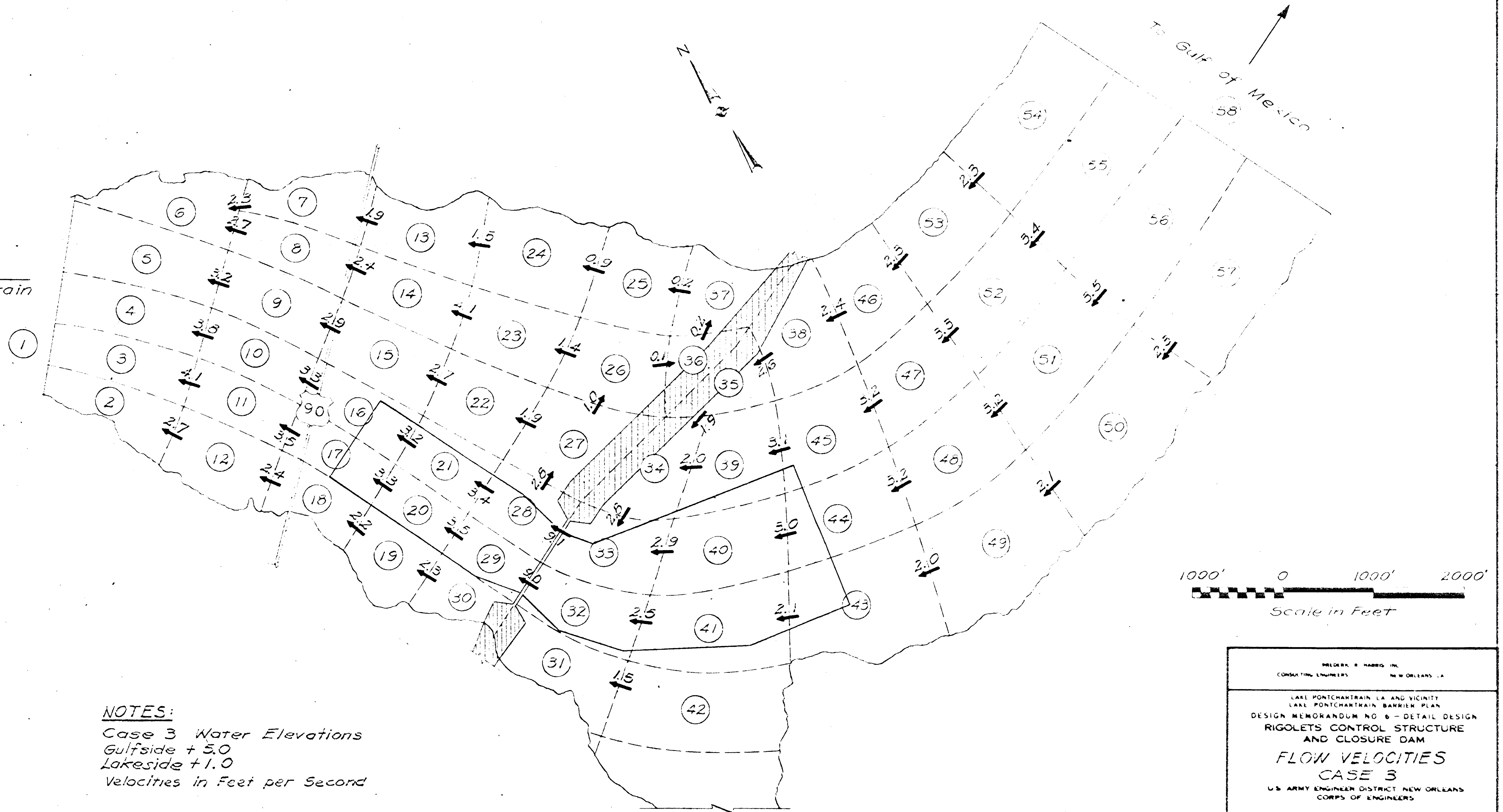

To Gulf of Mexico
N
To Lake Pontchartrain
1000' 0 1000' 2000'
Scale in Feet
NOTES:
Case 6 Water Elevations
Gulfside -3.0
Lakeside +2.5
Velocities in Feet per Second
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
FLOW VELOCITIES
CASE 6
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE JULY 1972
FILE NO. H-2-24417

PLATE II-4



NOTES:

Case 3 Water Elevations
Gulfside + 5.0
Lakeside + 1.0
Velocities in Feet per Second

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RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

FLOW VELOCITIES
CASE 3

U.S. ARMY ENGINEER DISTRICT NEW ORLEANS
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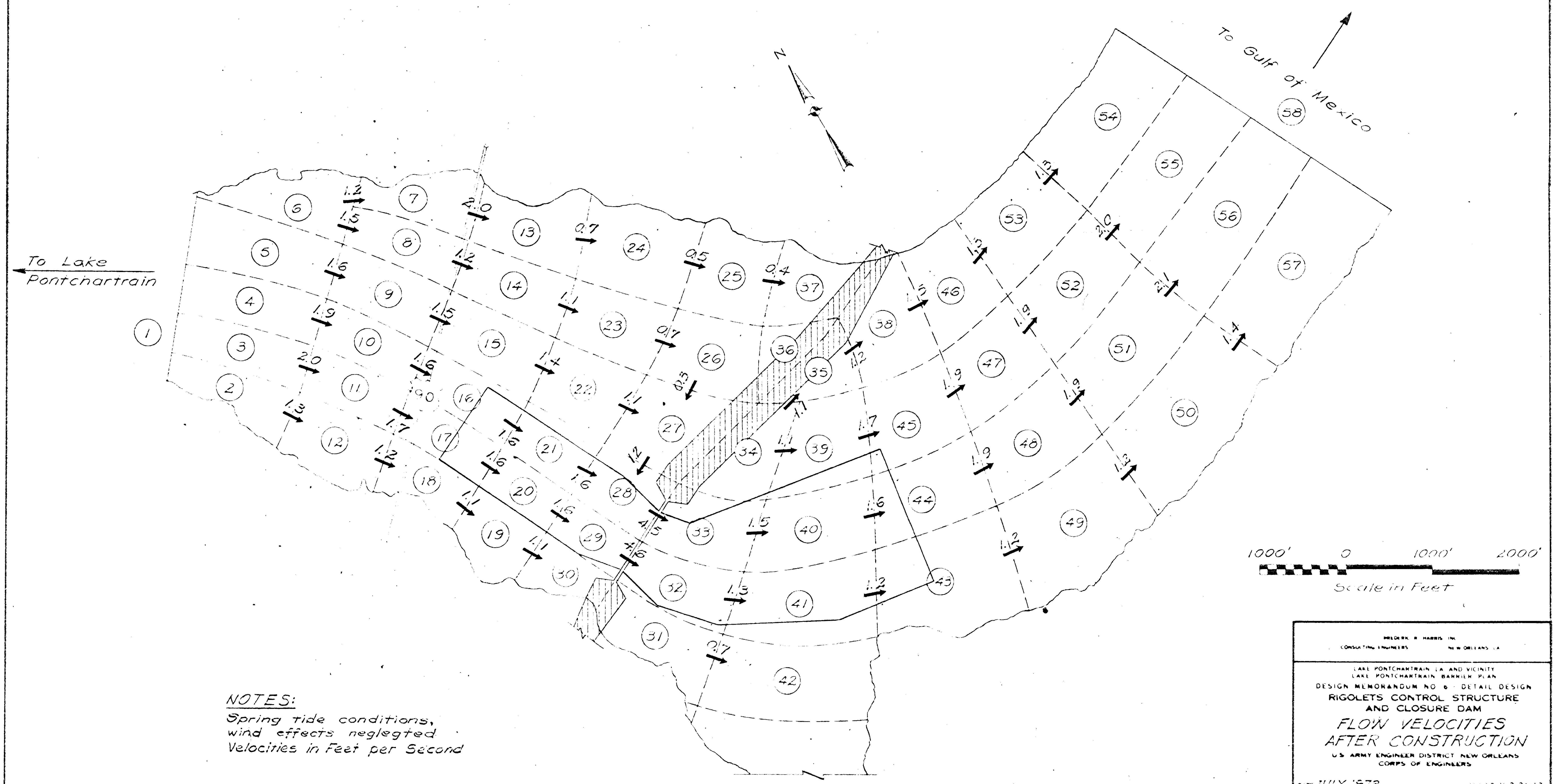
DATE JULY 1972 FILE NO H-2-24417



NOTES:
Spring tide conditions, wind effects neglegted
Velocities in Feet per Second

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LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

FLOW VELOCITIES
AFTER CONSTRUCTION

U.S. ARMY ENGINEER DISTRICT NEW ORLEANS
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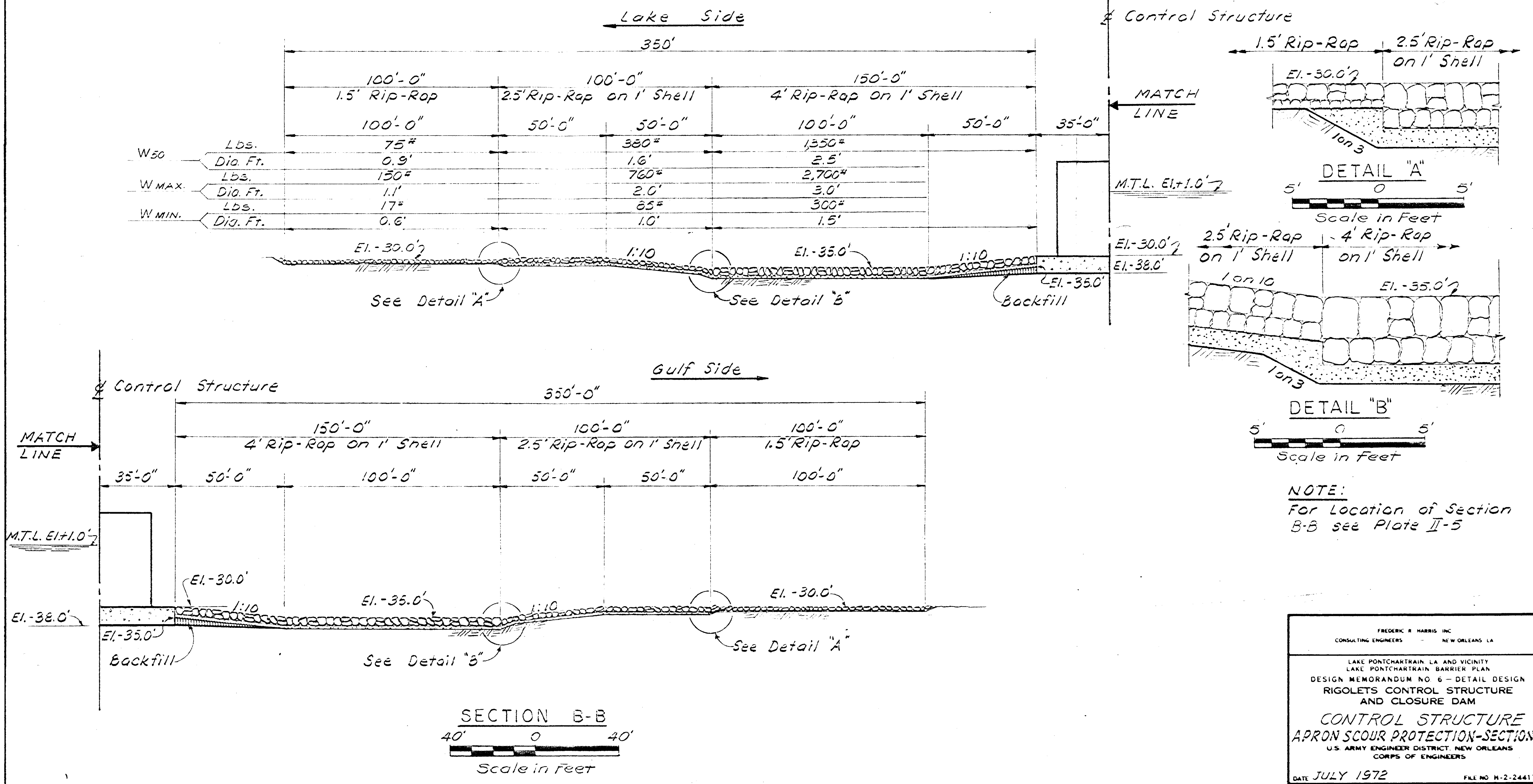
DATE JULY 1972 FILE NO H-2-24417

PLATE II-2



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LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

CONTROL STRUCTURE
APRON SCOUR PROTECTION-SECTIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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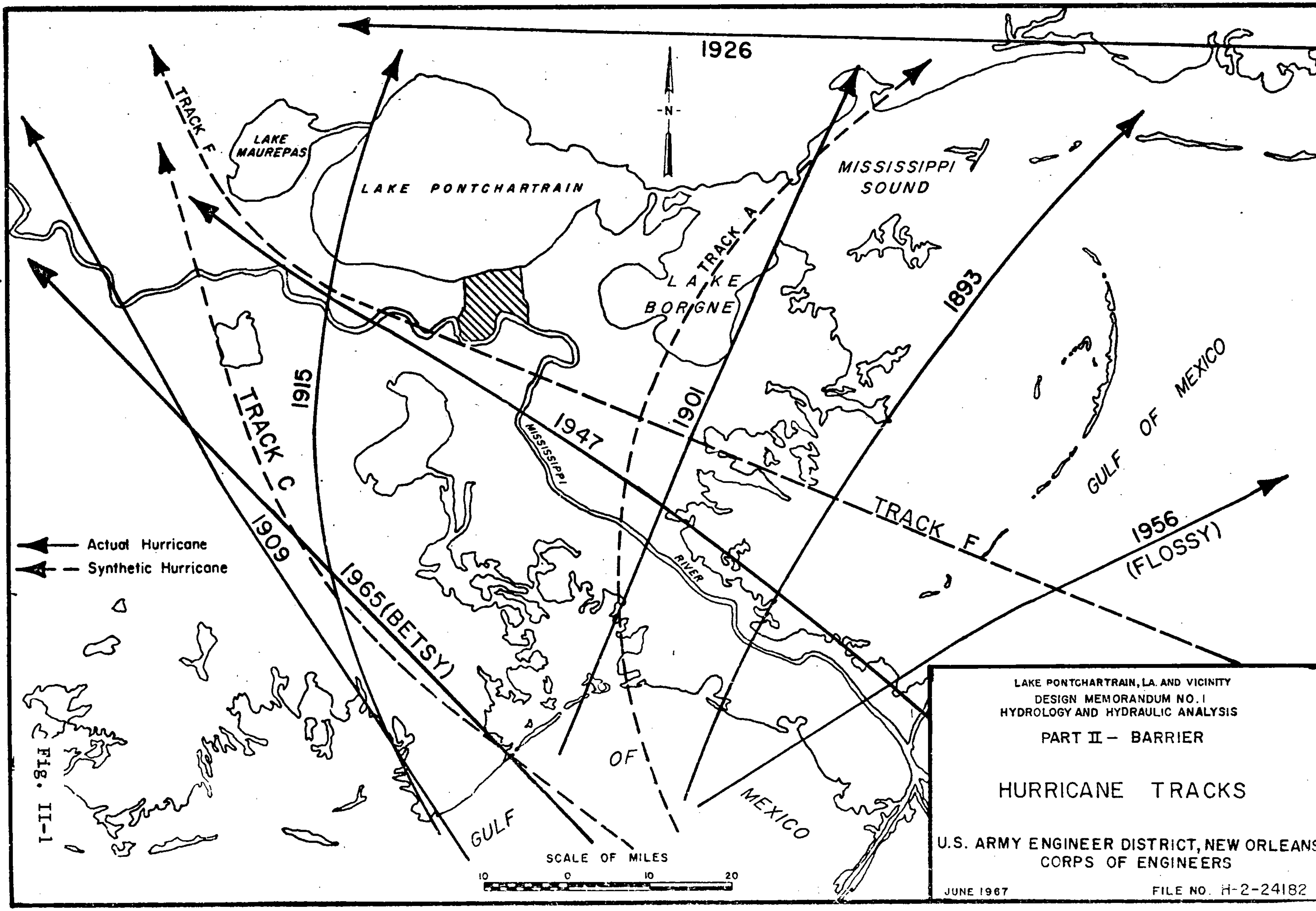
DATE JULY 1972 FILE NO. H-2-24417



LAKE PONTCHARTRAIN, LA. AND VICINITY
DESIGN MEMORANDUM NO. I
HYDROLOGY AND HYDRAULIC ANALYSIS
PART II - BARRIER

HURRICANE TRACKS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

JUNE 1967 FILE NO. H-2-24182

Fig. II-1

CASE I (Waves from Gulf)

REF.
1,2,3,5

S.W.L. = +12.8' [illegible]' H_s = 5.8' H_{10} = 7.37' H_1 = 9.7'
L_o = 172' T = 5.8 sec. $\frac{d}{L_o} = \frac{42.8}{172} = 0.250$

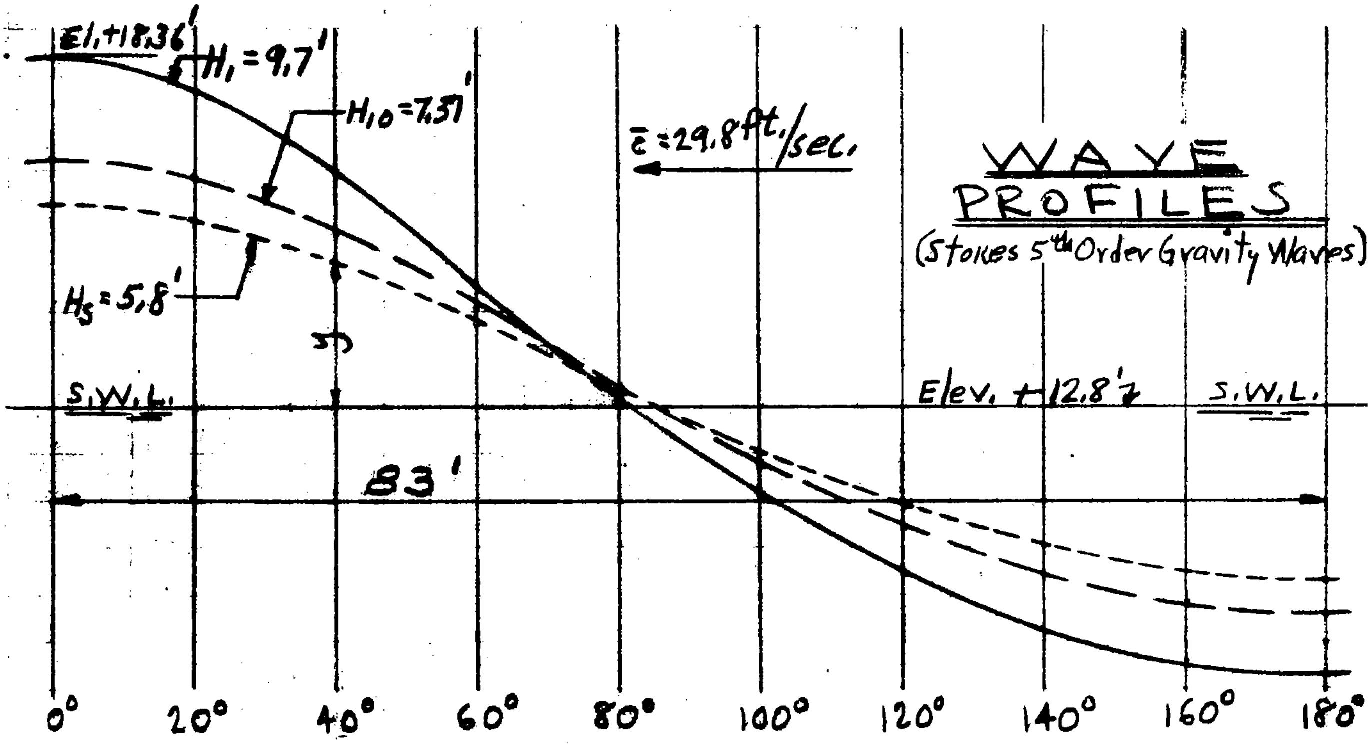
$d = 30 + 12.8 = 42.8'$ $\frac{d}{T^2} = \frac{42.8}{5.8^2} = 1.27$ $y = 42.8(Y-1)$

Wave Profile Data

H	H/d	d/T²	d/L	Y / y	θ 0°	20°	40°	60°	80°	100°	120°	140°	160°	180°
H_s, 5.8'	0.135	1.27	0.259	Y	1.075	1.070	1.054	1.031	1.007	0.983	0.963	0.948	0.939	0.935
				y	3.21	3.00	2.31	1.33	0.30	-0.73	-1.58	-2.23	-2.61	-2.78
H_{10} 7.37'	0.172	1.27	0.256	Y	1.093	1.086	1.065	1.037	1.007	0.978	0.955	0.937	0.927	0.923
				y	3.98	3.68	2.78	1.58	0.30	-0.94	-1.92	-2.70	-3.12	-3.30
H_1, 9.7'	0.226	1.27	0.254	Y	1.130	1.118	1.087	1.046	1.005	0.968	0.939	0.917	0.904	0.900
				y	5.56	5.05	3.72	1.97	0.21	-1.37	-2.61	-3.55	-4.06	-4.28

165'
167'
169'

Use L = 166' for all wave heights



note → reverse pt of wave direction

Vert. Scale: 1" = 3'

CASE 1 (Waves from Gulf)

Assume:

A clapotis is formed from a standing wave as the incoming wave impinges on the closure structure gates and wave height = 2H.

S.W.L. = +12.8' $H_0' = 6.08'$ $H_s = 5.8'$ $H_{10} = 7.37'$ $H_1 = 9.7'$

$L_o = 172'$ $T = 5.8\,sec.$ $d/L_o = 42.8/172 = 0.250$

$d = 30.0 + 12.8 = 42.8'$ $\frac{d}{T^2} = \frac{42.8}{5.8^2} = 1.27$ $y = 42.8(\Upsilon - 1)$

Wave Profile Data

2H	2H/d	d/T^2	d/L	Υ / y	θ 0°	20°	40°	60°	80°	100°	120°	140°	160°	180°
$2H_s$ 11.6'	0.271	1.27	0.251	Υ	1.156	1.140	1.101	1.051	1.003	0.961	0.928	0.905	0.891	0.886
				y	6.67	6.00	4.28	2.14	0.13	-1.67	-3.08	-4.06	-4.66	-4.89
$2H_{10}$ 14.7'	0.343	1.27	0.246	Υ	1.203	1.180	1.123	1.057	0.996	0.947	0.910	0.885	0.869	0.863
				y	8.70	7.70	5.26	2.44	-0.17	-2.27	-3.85	-4.93	-5.61	-5.86
$2H_1$ 19.4'	0.453	1.27	0.236	Υ	1.283	1.243	1.152	1.057	0.981	0.925	0.885	0.858	0.840	0.833
				y	12.12	10.40	6.50	2.44	-0.81	-3.22	-4.93	-6.08	-6.85	-7.15

USE L = 176' for all waves

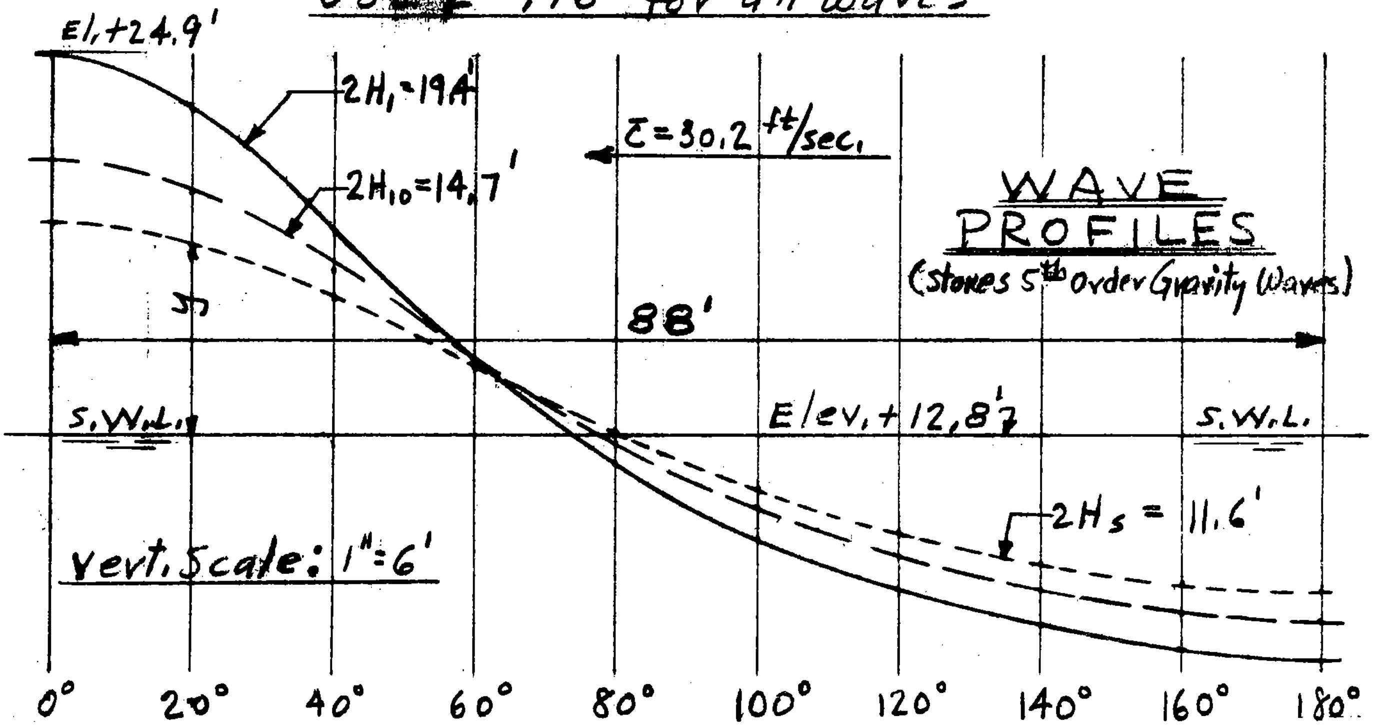


Fig. II - 3

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Subject: Hurricane Barrier — Wave Profiles
Computed by: JKK Checked by:
Sheet No. W3 of
Job No. 18-111-02
Date 4/72

(Waves from Lake Ponchartrain)

S.W.L. = 11.5' | $H_{c} = 6.26'$ $H_s = 5.85'$ $H_{10} = 7.43'$ $H_1 = 9.77'$

$L_o = 308'$ $T = 7.75$ sec. $d/L_o = 41.5/308 = 0.1345$

$d = 30.0' + 11.5' = 41.5'$ $\frac{d}{T^2} = \frac{41.5}{7.75^2} = 0.69$ $y = 41.5(Y-1)$

Wave Profile Data

H	H/d	d/T²	d/L		θ: 0°	20°	40°	60°	80°	100°	120°	140°	160°	180°
H_s 5.85	0.141	0.69	0.171	Y	1.075	1.070	1.053	1.030	1.005	0.982	0.962	0.948	0.939	0.936
				y	3.10	2.90	2.20	1.24	0.21	-0.75	-1.57	+2.20	-2.50	-2.66
H_{10} 7.43	0.179	0.69	0.170	Y	1.100	1.091	1.068	1.037	1.005	0.975	0.951	0.934	0.924	0.920
				y	4.50	3.76	2.82	1.53	0.21	-1.09	-2.03	-2.74	-3.15	-3.30
H_1 9.77	0.236	0.69	0.168	Y	1.135	1.122	1.089	1.045	1.002	0.965	0.936	0.915	0.909	0.900
				y	5.60	5.07	3.70	1.87	0.08	-1.45	-2.66	-3.52	-4.00	-4.15

Use $L = 244$ for all waves

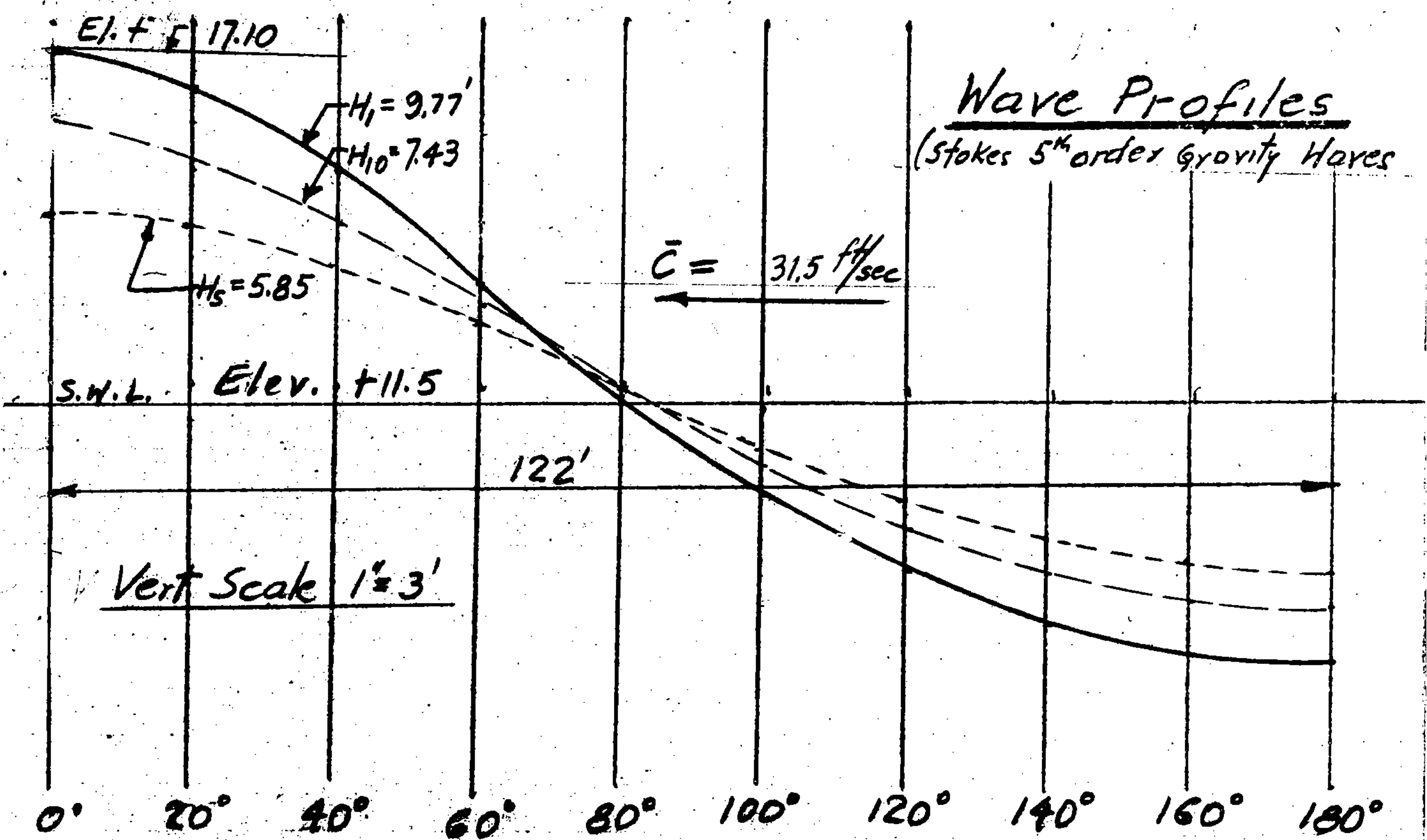

Fig. II - 4

FREDERIC R. HARRIS, INC.
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Subject: Hurricane Barrier Wave Profiles
Computed by: T.K.K. Checked by:
Sheet No. W4 of
Job No. 18-111-02
Date 4/72

Case 4
Waves from Lake Ponchartrain

A Clapotis is formed from a standing wave from the wave impinging on the control structure; height = 2H

S.W.L. = +11.5 $H_o = 6.26'$ $H_s = 5.85'$ $H_{10} = 7.43'$ $H_1 = 9.77'$ $T = 7.75$ sec.
$L_o = 308'$ $d = 30 + 11.5 = 41.5'$ $d/T^2 = 0.69$ $y = 41.5(Y-1)$

Wave Profile Data

H	2H/d	d/T^2	d/L		θ 0°	20°	40°	60°	80°	100°	120°	140°	160°	180°
$2H_s$ 11.70	0.282	0.69	0.167	Y	1.167	1.149	1.105	1.050	0.998	0.956	0.924	0.902	0.889	0.885
				y	6.95	6.17	4.35	2.07	- 0	-1.82	-3.15	-4.06	-4.62	-4.88
$2H_{10}$ 14.86	0.358	0.69	0.164	Y	1.222	1.193	1.128	1.054	0.989	0.939	0.903	0.881	0.867	0.863
				y	9.22	8.00	5.29	2.08	-0.46	-2.52	-4.02	-4.95	-5.52	-5.70
$2H_1$ 19.54	0.470	0.69	0.159	Y	1.306	1.259	1.154	1.050	0.972	0.916	0.878	0.856	0.842	0.835
				y	12.70	10.70	6.40	2.07	-1.16	-3.07	-5.07	-6.00	-6.56	-6.85

Use L = 254' for all waves

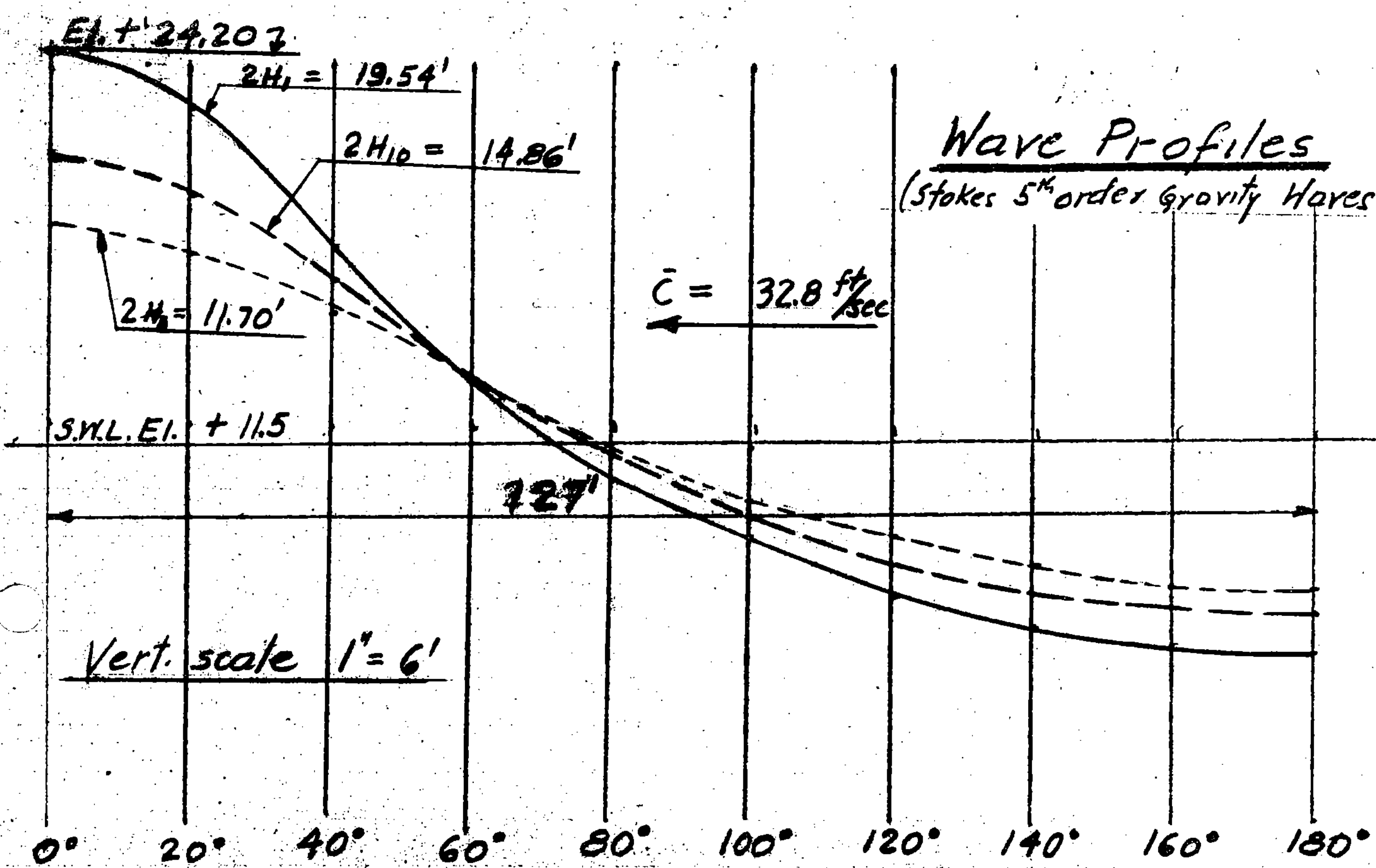


Fig. II - 5

Gulf side Case I

$H_o = 6.08'$ $H_s = 5.8'$ $H_{10} = 5.8 \times 1.27 = 7.37'$ $H_1 = 5.8 \times 1.67 = 9.7'$

SWL = +12.8' (-3.0 Lakeside)

$H_o'/T^2 = 0.[illegible]1\ (1:55)$ T = 5.8 sec. $L_o = 172'$

For depth $d = 30.0 + 12.8' = 42.8'$ @ SWL

$\frac{d}{L_o} = \frac{42.8}{172} = 0.249$; From table D-1;

for $\frac{d}{L_o} = 0.249$ $\frac{d}{L} = 0.267$ $L = \frac{42.8}{0.267} = 160'$

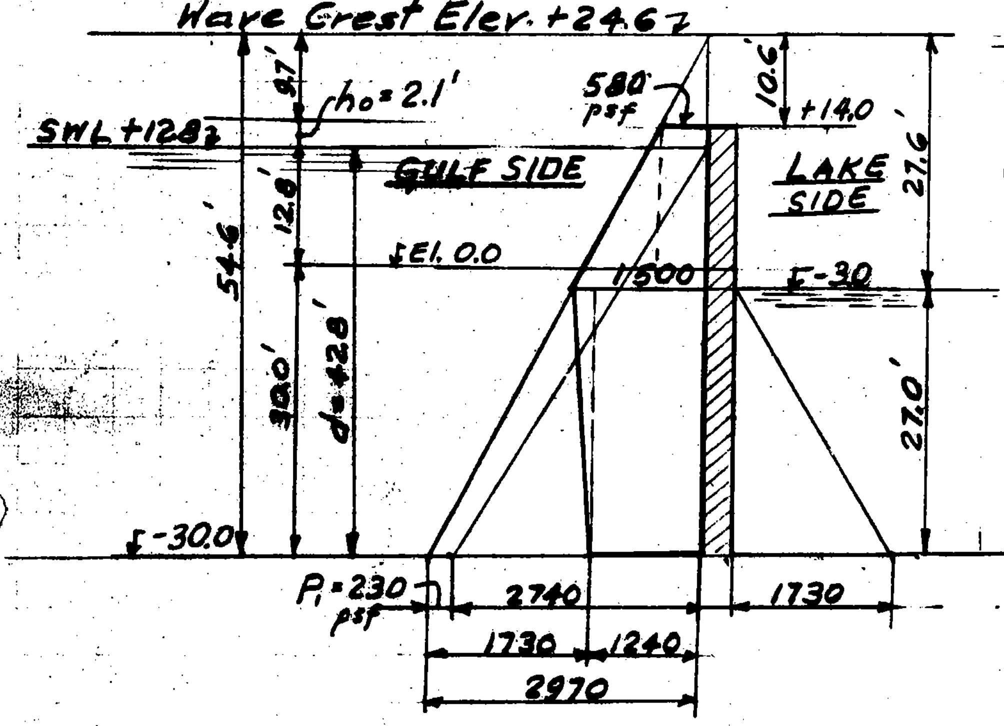
for $\frac{d}{L} = 0.267$ & $H = H_1 = 9.7'$ from Figure 2-13

$L h_o = 330$; $h_o = \frac{330}{160} = 2.06'$

From Figure 2-14 $P_1 = 230$ psf.

$\frac{10.6}{54.6} \times 2970 = 580$

$\frac{27.6}{54.6} \times 2970 = 1500$



580 × 17.0 =	9,900
(1500 - 580) 17.0/2 =	7,800
1240 × 27 =	33,400
(1500 - 1240) 27/2 =	3,500
	54,600

SAINFLOU PRESSURE DIAGRAM Fig. II - 6

Lake side (Case 4)

$H_o' = 6.26'$ $H_s = H_{1/3} = 5.85'$ $H_{10} = 7.43'$; $H_1 = 9.77'$

S.W.L. = +11.5' — (-3.0 Gulf side)

$H_o'/T^2 = 0.104$ (1:96) T = 7.75 sec. $L_o = 308'$

For depth d = 30' + 11.5 = 41.5' at S.W.L.

$\frac{d}{L_o} = \frac{41.5}{308} = 0.135$; From table D-1 $\frac{d}{L} = 0.1708$

$L = 41.5/0.1708 = 243'$ @ d = 41.5'

From Fig. 4-2 & $\frac{d}{L} = 0.171$ & H = 9.77'; $Lh_o = 380$; $h_o = \frac{380}{243} =$

From Fig 4-3 & $\frac{d}{L} = 0.171$ & H = 9.77 $P_1 = 350$ p.s.f.

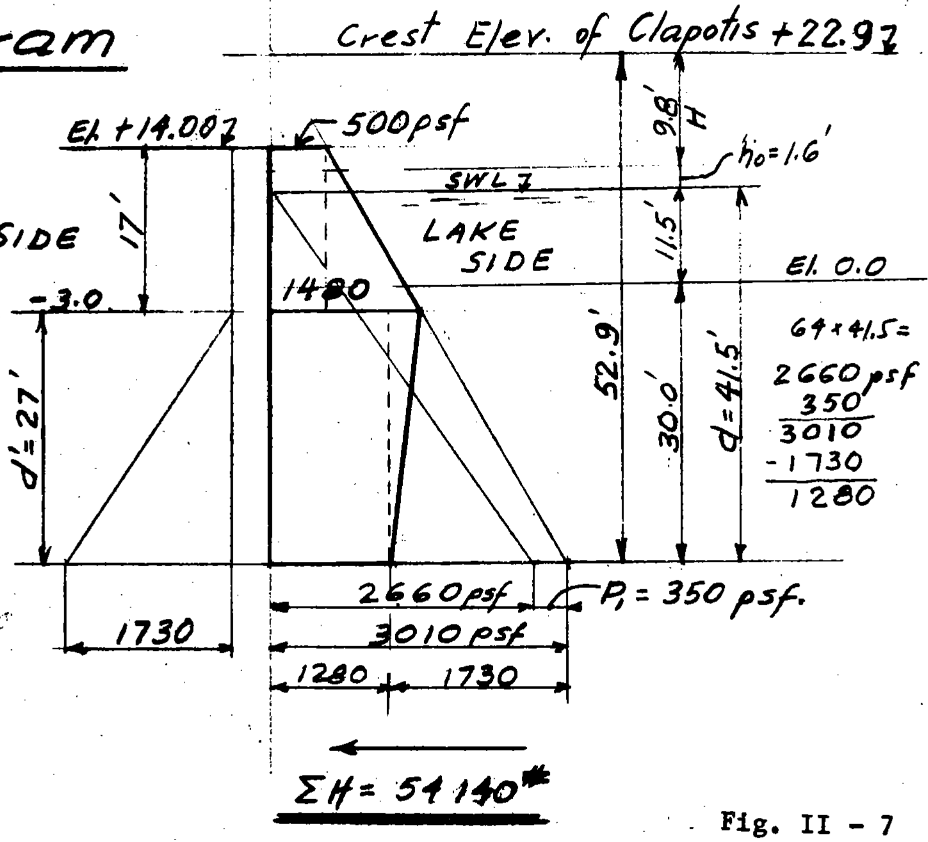
Assume water = 64 pcf.

Sainflou Pressure Diagram

$3010 \cdot \frac{8.9}{52.9} = 500$

$3010 \cdot \frac{25.9}{52.9} = 1480$

$64 \cdot 27 = 1730$ psf



$500 \cdot 17.0 = 8,500$

$(1480 - 500) \cdot \frac{17}{2} = 8,340$

$1280 \times 27.0 = 34,600$

$(1480 - 1280) \cdot \frac{27}{2} = 2,700$

54,140

Fig. II - 7

General Requirements

Determine horizontal and vertical forces resulting from max. height waves acting on the crane and bridge girders.

Method used - The Morison Equation

Reference:
Oceanographic Engineering by R.L. Wiegel.

$$F = \underbrace{\frac{\rho}{2} C_D D |u| u}_{\text{Drag Force}} + \underbrace{\rho C_M V \frac{du}{dt}}_{\text{Inertial Force}}$$

Where:

F = Total wave force on a structural object

ρ = Mass density of water = $\frac{64\ \#/Ft^3}{32.2\ Ft/sec} = 2\ \#\ \frac{Ft^{-4}}{Sec^2}$

D = Projected area of object

V = Volume of displaced water

C_D = Hydrodynamic drag coefficient

C_M = -"- inertia -"-

Determination of C_D and C_M

Laboratory tests with rectangular plates submerged in the wave zone, revealed a relationship between values of $U_{max} \cdot \frac{T}{D}$ and values of C_D and C_M.

The results are presented on graphs shown on page 6, (which is reproduced from "Wiegel")

It is reasonable to assume, that the coefficients will be <u>much lower</u> for circular girders.

The coefficients will be determined from the graphs after the maximum velocities have been calculated.

The thus found values of C_D and C_M will be reduced for horizontal forces by 40% to reflect the circular shape of girders. For vertical force the circular effect is diminished by the grating, therefore the coefficients will be reduced by 10%.

Values of u and $\frac{du}{dt}$

1) u = Horiz. component of orbital velocity of wave particle

$$u = \sigma \frac{H}{2} \cdot \frac{\cosh k(d+y)}{\sinh k(d+y)} \cdot \cos(kx-\sigma t) + \tfrac{3}{4}\sigma k \left(\frac{H}{2}\right)^2 \frac{\cosh 2k(d+y)}{\sinh^4 kd} \cdot \cos 2(kx-\sigma t)$$

1) $\frac{du}{dt}$ = Horiz. component of orbital acceleration of wave particle

$$\frac{du}{dt} = \sigma^2 \frac{H}{2} \frac{\cosh k(d+y)}{\sinh kd} \times \sin(kx-\sigma t) + \tfrac{3}{2}\sigma^2 k \left(\frac{H}{2}\right)^2 \frac{\cosh 2k(d+y)}{\sin^4 kd} \times \sin 2(kx-\sigma t)$$

$\sigma = \frac{2\pi}{T}$; T = wave period ; d = water depth

$k = \frac{2\pi}{L}$; L = wave length

$(kx - \sigma t) = \theta$ = phase angle of wave

y = distance of surface water particle from still water level

$$y = \frac{H}{2}\cos(kx-\sigma t) + \tfrac{1}{2} k \left(\frac{H}{2}\right)^2 \left(1 + \frac{3}{2\sinh^2 kd}\right) \coth kd \cdot \cos(kx-\sigma t)$$

Note

1) For vertical wave forces, vertical velocity and acceleration components v and $\frac{dv}{dt}$, are to be used

$$v = \sigma \frac{H}{2} \frac{\sinh k(d+y)}{\sinh kd} \times \sin(kx-\sigma t) + \tfrac{3}{4}\sigma k \left(\frac{H}{2}\right)^2 \frac{\sinh 2k(d+y)}{\sin^4 kd} \sin 2(kx-\sigma t)$$

$$\frac{dv}{dt} = -\sigma^2 \frac{H}{2} \frac{\sinh k(d+y)}{\sin kd} \cos(kx-\sigma t) - \tfrac{3}{2}\sigma^2 k \frac{\sin 2k(d+y)}{\sin^4 kd} \times \cos 2(kx-\sigma t)$$

Values of D and V

The forces will be computed per linear foot of girder. The projected area of girder

$D = 6.5\ Ft^2/L.Ft.$ see sketch page 17

The volume of displaced water per foot

$V = \frac{3.14 \times 5.0^2}{4} + 1.4 = 21.0\ Ft^3/L.Ft.$

Magnification Factors

a) Dynamic Factor

In addition to the forces determined by the Morison equation, there are additional undetermined effects of waves breaking against the structure.

These include suction due to backdraft, vibration of the structure, cavitation of the entrapped air e.t.c.

In order to compensate for these conditions it is considered advisable to increase the design force by a factor of 1.33

b) Fatigue Factor

Forces produced by waves vary in intensity. A maximum occurs when the crest of the wave passes and a minimum at the trough.

Although the girders are basically exposed to waves from one side only, overtopping waves may act from the opposite side.

This condition may cause reversal of stresses.

It is known that structural materials are adversely affected by repeated loadings. See pages 7 & 8 which are reproduced from a book on struct. dynamics.

It is impossible to predict the number of reversal cycles in the lifetime of the structure. To provide a margin for possible fatigue, an additional factor 1.5 will be used.

R.L. WIEGEL
OCEANOGRAPHICAL
ENGINEERING

7. WAVE FORCES ON SUBMERGED VERTICAL PLATES

Laboratory studies of wave forces on a flat plate have been made for standing waves by Keulegan and Carpenter (1956) and Brater, McNown, and Stair (1958).

The results of Keulegan and Carpenter (1958) are shown in Fig. 11.19. For flat plates, C_M is associated with the displaced volume of a circular cylinder having the same diameter as the width of the plate, provided that the width of the plate is normal to the direction of wave advance. As the plate extended from one wall of the wave channel to the other, the results should be nearly those of a flat plate infinitely long. The plate was mounted horizontally.

Brater, McNown, and Stair (1958) determined values of C_D and C_M for a flat plate 30 in. long, $1\frac{7}{8}$ in. high and $\frac{1}{4}$ in. thick with the 30-in. × $1\frac{7}{8}$-in. face normal to the direction of flow, for waves of the order of 5 ft long in water 1 ft deep. Measurements were obtained with the center of the model $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{3}{4}$ of the water depth below the surface. They determined a type of average C_M and C_D by varying the values until a force-time-history curve was obtained that nearly matched the measured time history curve, with C_M based upon the volume of a circular cylinder $1\frac{7}{8}$ in. in diameter and 30 in. long. It was found that the average C_M was about 1.75 and the average C_D was about 3.5. The values of $u_{max}T/D$ varied from 0.9 to 5.3, the average being 3.1. C_M compares favorably with the curve of Keulegan and Carpenter whereas C_D is lower; however, the C_D obtained by Keulegan and Carpenter varies rapidly in this region.

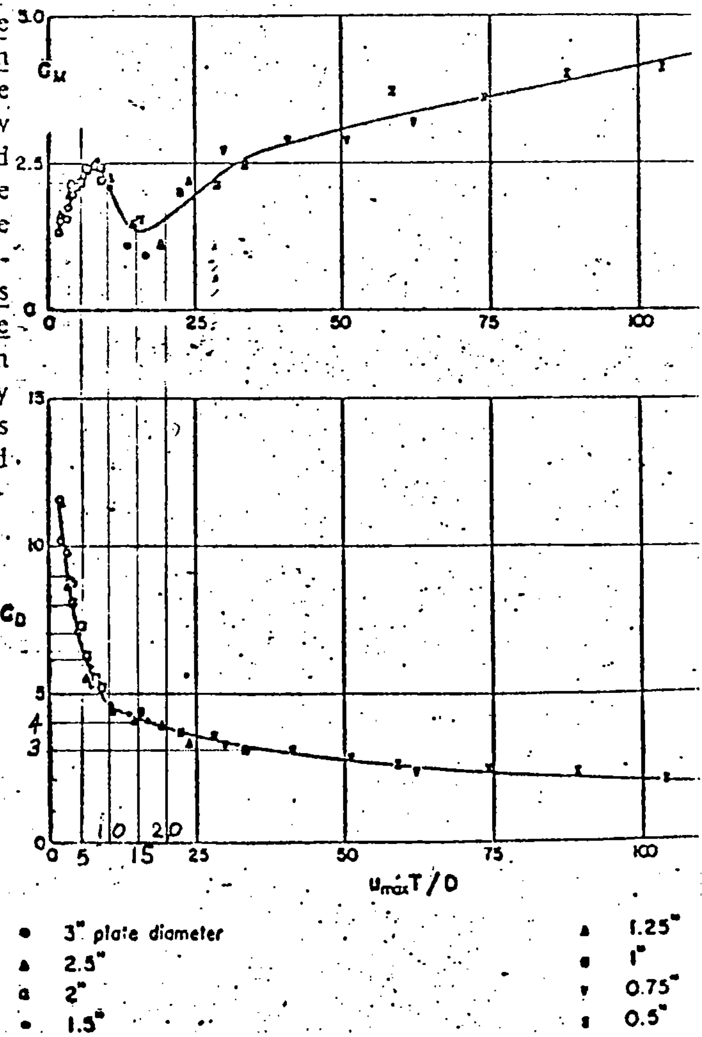


Fig. 11.19. Variation of inertia (top) and drag (bottom) coefficients of flat plates (after Keulegan and Carpenter, 1958)

reinforced concrete in flexure) it appears that the extreme fibers do indeed experience the descending stress-strain relation, to a maximum strain of approximately 0.4 per cent. At this strain, or a little less, complete failure of the concrete is to be expected. Although the descending portion of the stress-strain diagram is shown curved in Fig. 2.1, the actual shape is so uncertain that it could as well be taken as a straight line. Hognestad [2] suggests that the stress at failure be assumed to be 85 per cent of the ultimate strength.

b. Behavior under Repeated Loading. When each cycle ranges from a minimum compression stress that is essentially zero to a maximum compression stress it appears that the maximum stress, repeated for 1 or 2 million cycles, is limited to be between 50 and 65 per cent of the static ultimate strength. For any number of cycles in excess of 1 million it should be safe to assume a range from 0 to 50 per cent of static ultimate in each cycle.

As the minimum stress per cycle is increased, the maximum stress per cycle also is increased, but the tolerable range of stress decreases. For practical purposes, the variation shown in Fig. 2.2 is suggested as a conservative approximation.

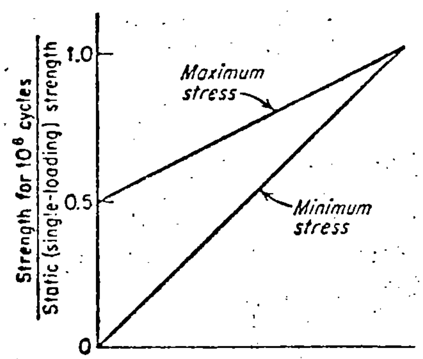


FIG. 2.2. Concrete compressive strength (>1 million cycles) as ratio of static strength.

Although it is certain that the tolerable range of cyclic stress increases with decreasing number of cycles, the available data seem insufficient to warrant proposing any mathematical relationship between these two variables at the present time.

Static tests to failure, after any number of cycles of stress below the endurance limits indicated by Fig. 2.2, show ultimate strengths as great as (or slightly greater than) those obtained from companion specimens not subjected to repeated loading.

c. Behavior under Rapid Straining Rates. Watstein [5] tested 3- by 6-in. cylinders in compression at rates of strain varying from 10^{-6} to 10 in./(in.)(sec). Two concretes of widely different nominal compressive strengths, namely, 2,500 and 6,500 psi, were studied.

Figure 2.3, based on results reported by Watstein [5], is a curve of DIF (*dynamic increase factor* = dynamic f'_c divided by static f'_c) vs. rate of strain. It will be noted that, for very high rates of strain, the dynamic ultimate strength can be very much greater than the static ultimate strength.

The tests referred to above also indicated increasing values of strain

STRUCTURAL DESIGN FOR DYNAM. LOADING
BY NORRIS HANSEN

approximately constant (though steadily decreasing) slope up to the point of yielding. If the member is statically determinate, yielding at the section of maximum moment will convert the member to a mechanism, and the load-deflection curve will be essentially horizontal from the point of yielding to final collapse. If the member is statically indeterminate, yielding at a section will in effect remove one restraint, causing a sharp decrease in the slope of the load-deflection curve. Yielding of successive sections will cause successive reductions in load-deflection shape until the member becomes a mechanism, after which the load-deflection curve will be essentially horizontal to the point of collapse.

c. Flexural Behavior under Repeated Loading. It was pointed out in Sec. 2.2*b* that concrete under repeated compressive stress has an endurance limit which may be as low as 50 per cent of the static ultimate stress. In considering the possible significance of this figure it must be kept in mind that flexural sections subject to repeated loading usually are proportioned for working stresses in extreme compression fibers not to exceed $0.4f'_c$. As one proceeds toward the neutral axis, from the extreme fiber, stress intensities are progressively less. Thus if extreme fibers did, in fact, sustain cumulative strain because of repeated cycles of stress, a redistribution of stress might be expected to increase the share of resistance provided by zones closer to the neutral axis.

In statically indeterminate members the division of moments between different sections may be very different from that assumed in the design, either through an erroneous evaluation of relative stiffnesses, through an unanticipated settlement of supports, or because of some other factor. This might lead to moments on one section much greater (and on another section much less) than assumed and, in consequence, to extreme compression-fiber stresses at one section much greater than the endurance limit. Although cumulative strain at the overloaded section might lead to favorable redistribution of moments, the possibility of fatigue failure under the assumed set of circumstances appears to be real. This suggests that elastic analysis of indeterminate structures to determine internal gross forces may be more important for repeated loading.

A number of investigators [4, 21–25] have tested reinforced-concrete beams under repeated loading. In some cases steel percentages very different from conventional values were used to exaggerate a particular mode of failure. For example, LeCamus [25] used very low steel percentages in beams designed to fail by fatigue of the steel and very high percentages in beams designed to fail by fatigue of the concrete. For these beams he obtained endurance limits of 60 to 65 per cent of static strength. Other investigators [23, 24], using conventional percentages of steel, have obtained higher values; for example, for beams with 0.9 per cent steel Lea obtained a 1 million cycle endurance limit of 80 per

STRUCTURAL DESIGN FOR DYNAM. LOADING
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Horizontal Wave Forces

Gulf-side - Normal Wave

Wave characteristics

$d = 42.8$ ft $\quad H = H_1 = 9.7$ ft $\quad T = 5.8$ sec.

$L_o = 172$ ft $\quad \frac{d}{L_o} = \frac{42.8}{172} = 0.25$

From tables (Appendix 1) $\frac{d}{L} = 0.2679$; $L = \frac{42.8}{0.2679} = 160$ ft.

$\sigma = \frac{2\pi}{T} = \frac{6.28}{5.8} = 1.08$; $\quad k = \frac{2\pi}{L} = \frac{6.28}{160} = 0.0392$

$kd = 0.0392 \times 42.8 = 1.68$

From tables:

$\sinh kd\ (1.68) = 2.590 \qquad \tanh kd = 0.9328$

$\cosh kd = 2.776 \qquad \coth kd = 1.075$

Distance of surface particle y - (equation page 3)

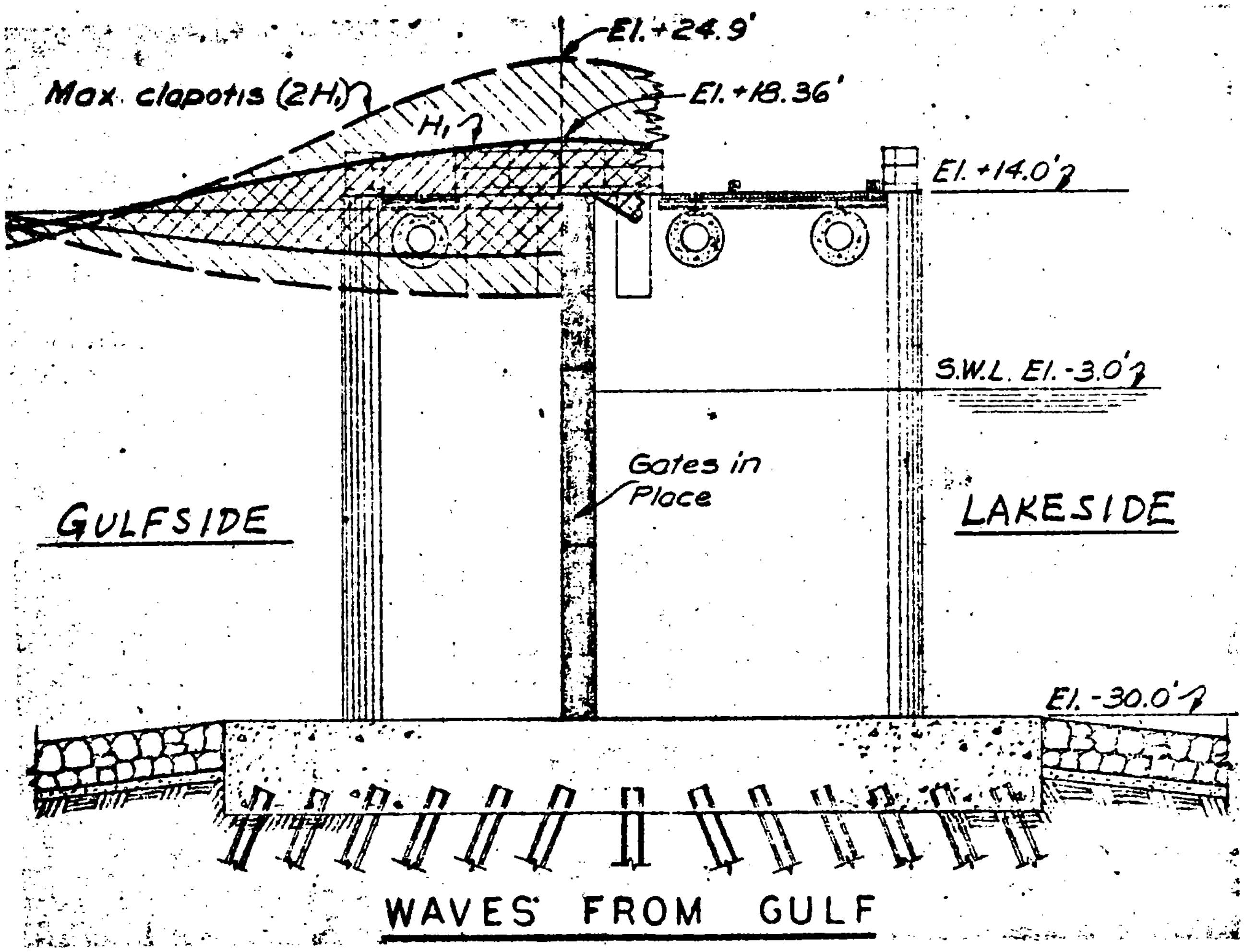
$y = \frac{9.7}{2}\cos\theta + \frac{1}{2}\cdot 0.0392\cdot\left(\frac{9.7}{2}\right)^2\cdot\left(1 + \frac{3}{2\times 2.59^2}\right)\cdot 1.075\cos 2\theta$

$y = 4.85\cos\theta + 0.61\cos 2\theta$

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

Subject: Control Structure
Wave Forces on Girders
Computed by: JLK Checked by:

Sheet No. 10 of
Job No. 18-111-02
Date: May 72



WAVE PROPERTIES

$H_s = 5.80'$ $d = 42.8'$
$L_0 = 172'$ $L_1 = 166'$
$H_1 = 9.70'$ $2H_1 = 19.40'$
$T = 5.8$ Sec. $h_0 = 2.06'$

Fig. II - 17

for $\theta = 0$ (at crest) $\cos\theta = 1.0$ $\cos 2\theta = 1.0$

$\underline{y} = 4.85 + 0.61 = \underline{5.46\ ft}$

$\underline{d+y} = 42.8 + 4.85 = 48.26\ ft.$

$k(d+y) = 0.0392 \times 48.26 = 1.89$; $2k(d+y) = 3.78$

From tables:

$\sinh k(d+y) = 3.235$ $\cosh 2k(d+y) = 21.92$

$\cosh k(d+y) = 3.385$

Velocity u (see equation page 3)

$$u = 1.08 \times 4.85 \times \frac{3.385}{2.590} \times \cos\theta + \frac{3}{4} \times 1.08 \times 0.0392 \times 4.85^2 \times \frac{21.92}{(2.59)^4} \times \cos 2\theta$$

$$u = 6.35 \cos\theta + 0.35 \cos 2\theta$$

for $\theta = 0$ $\cos\theta = 1.0$ $\cos 2\theta = 1.0$

$$u = 6.35 + 0.35 = \underline{\underline{6.70}}\ ft/sec = u_{max}.$$

Acceleration $\frac{du}{dt}$

$$\frac{du}{dt} = 1.08^2 \times 4.85 \times \frac{3.385}{2.590} \times \sin\theta + \frac{3}{2} \times 1.08^2 \times 0.0392 \times 4.85^2 \times \frac{21.92}{(2.59)^4} \times \sin 2\theta$$

$$\frac{du}{dt} = 7.40 \sin\theta + 0.8 \sin 2\theta$$

for $\theta = 0$ $\sin\theta = 0$ $\underline{\frac{du}{dt} = 0}$

Fig. II - 18

Coefficient : C_D

$$U_{max}\frac{T}{D} = 6.70\ \frac{5.8}{6.5} = 6.0$$

From Graph on page 6 find:

$C_D = 6.0$

Use reduced value

$C_D' = 0.6 \times 6.0 = \underline{3.6}$

Values D & V

From page 4 ;

$\underline{D = 6.5\ ft^2/Ft}$ $\underline{V = 21.0\ Ft^3/Ft}$

Wave Force ; (see equation page 1)

$F = \frac{2}{2} \times 3.6 \times 6.5 \times 6.7 \times 6.7 + 0 = \underline{1050}\ \#/Ft.$

Using magnification factors (pages 4 & 5)

$\underline{F} = 1.33 \times 1.5 \times 1050 = \underline{2{,}100}\ \#/Ft$

Horizontal Wave Forces

Gulf-side - Standing Wave (Clapotis)

Wave characteristics

$d = 42.8$ ft $H = H_i = 19.4$ $T = 5.8$ sec

$L_o = 172$ Ft. $L = 160$ Ft.

$\sigma = \frac{2\pi}{T} = 1.08$ $k = \frac{2\pi}{L} = 0.0392$ $kd = 1.68$

$\sinh kd = 2.590$ $\tanh kd = 0.9328$

$\cosh kd = 2.776$ $\coth kd = 1.075$

Distance of surface particle

$$y = \frac{19.4}{2}\cos\theta + \frac{1}{2}\cdot 0.0392\cdot\left(\frac{19.4}{2}\right)^2\left(1+\frac{3}{2\cdot 2.59^2}\right)\times 1.075\cdot\cos 2\theta$$

$$y = 9.7\cos\theta + 2.44\cos 2\theta$$

for $\theta = 0$

$y = 12.14$ ft $d+y = 42.8 + 12.14 = 54.94$

$$k(d+y) = 0.0392 \times 54.94 = 2.20$$

$\cosh k(d+y) = 4.567$ $\cosh 2k(d+y) = 40.72$

$\sinh k(d+y) = 4.466$

Velocity u (see equation page 3)

$$u = 1.08 \times \frac{19.4}{2} \times \frac{4.576}{2.59} \cos\theta + \frac{3}{4} \cdot 1.08 \cdot 0.0392 \cdot \left(\frac{19.4}{2}\right)^2 \frac{40.72}{(2.59)^4} \cos 2\theta$$

$$u = 18.5 \cos\theta + 2.68 \cos 2\theta$$

for $\theta = 0$; $\cos\theta = 1$; $\cos 2\theta = 1$

$u = 18.5 + 2.68 = \underline{21.18}$ Ft/sec

Acceleration $\frac{du}{dt}$

for $\theta = 0$ $\sin\theta = 0$; $\underline{\frac{du}{dt} = 0}$

Coefficient C_D

$$u_{max} \frac{T}{D} = 21.18 \times \frac{5.8}{6.5} = 18.9$$

from graph page 6 ; $C_D = 3.75$

Reduced $C_D' = 0.6 \times 3.75 = \underline{2.25}$

<u>Values D & V</u> see page 4

$D = 6.5\ ft^2/ft$ $V = 21.0\ Ft^3/Ft$

<u>Wave Force</u> (see equation page 1)

$F = 2.25 \times 6.5 \times 21.18 \times 21.18 = 6{,}600\ \#/Ft.$

Using magnification factors pages 4 & 5

$F = 1.33 \times 1.5 \times 6{,}600 = 13200\ \#/Ft.$

Horizontal Wave Forces

Lake side - Normal Wave

Wave characteristics

$d = 41.5\ ft$ $H = H_1 = 9.77'$ Use $H = 9.8'$ $T = 7.75\ sec.$

$L_0 = 308'$ $\frac{d}{L_0} = \frac{41.5}{308} = 0.135$

from tables $\frac{d}{L} = 0.1708$; $L = 244'$

$\sigma = \frac{2\pi}{T} = \frac{6.28}{7.75} = 0.81$ $k = \frac{2\pi}{L} = \frac{6.28}{244} = 0.0258$

$kd = 0.0258 \times 41.5 = 1.073$

from tables:

$\sinh kd = 1.291$ $\tanh kd = 0.7905$

$\cosh kd = 1.633$ $\coth kd = 1.265$

Distance of surface particle y see page 3

$$y = \frac{9.8}{2} \times \cos\theta + \frac{1}{2} \times 0.0258 \times \left(\frac{9.8}{2}\right)^2 \left[1 + \frac{3}{2 \times 1.291^2}\right] \times 1.265 \cdot \cos 2\theta$$

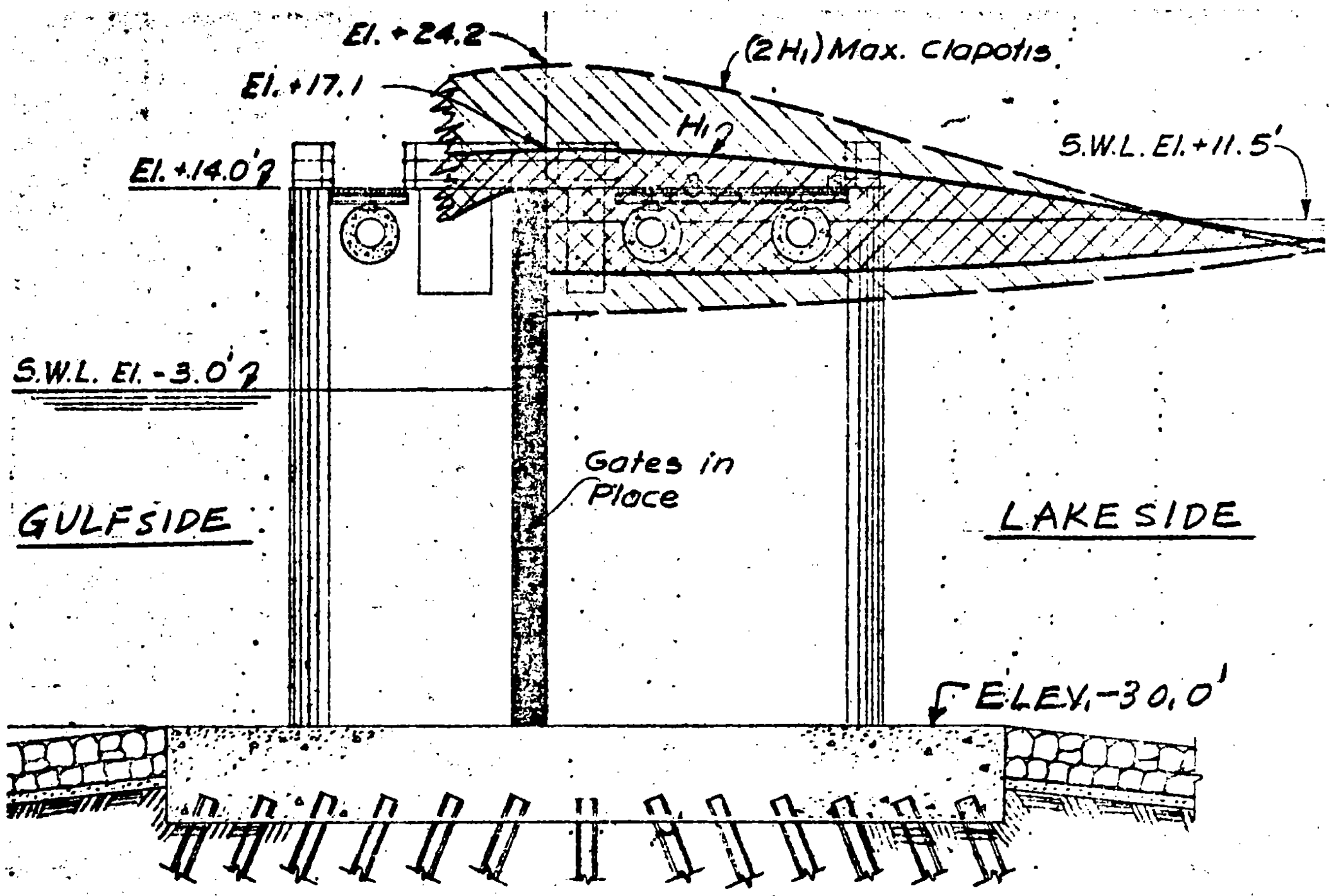
$y = 4.9 \cos\theta + 0.75 \cos 2\theta$

for $\theta = 0$; $\cos\theta = \cos 2\theta = 1$

$y = 4.9 + 0.75 = 5.65'$

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

Subject: Control Structure
Wave Forces on Girders
Computed by: JLK Checked by:
Sheet No. ___ of ___
Job No. 18-111-02
Date: May 72



WAVES FROM LAKE

WAVE PROPERTIES

$H_s = 5.65'$ $d = 41.5'$
$L_o = 308'$ $L_1 = 244'$
$H_1 = 9.77'$ $2H_1 = 19.54'$
$T = 7.75$ Sec. $h_o = 1.6'$

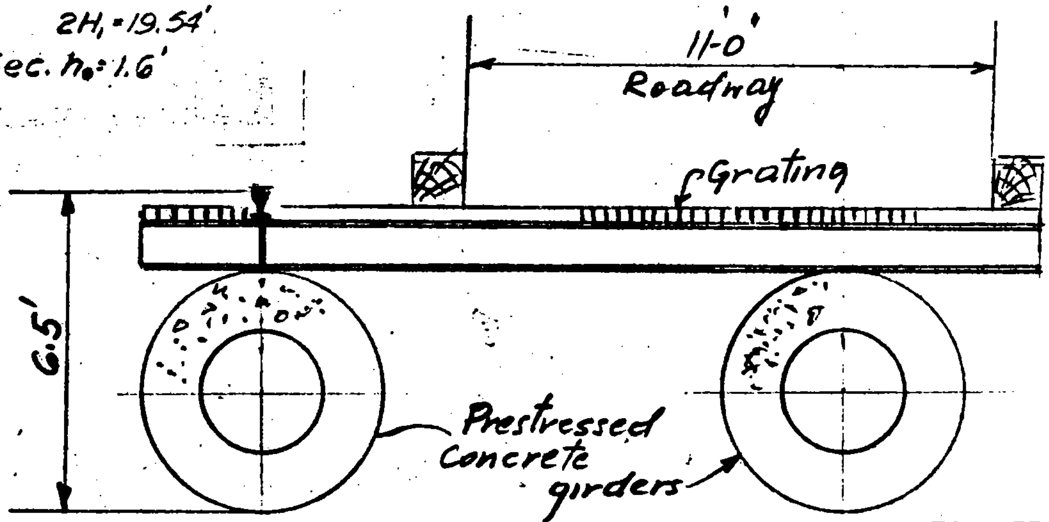


Fig. II - 24

$d+y = 41.5 + 5.65 = 47.15'$

$k(d+y) = 0.0258 \times 47.15 = 1.215$ $\qquad 2k(d+y) = 2.430$

from tables:

$\sinh k(d+y) = 1.536$

$\cosh k(d+y) = 1.833$ $\qquad \cosh 2k(d+y) = 5.725$

Velocity u (see equation page 3)

$$u = 0.81 \times \frac{9.8}{2} \frac{1.833}{1.291} \times \cos\theta + \frac{3}{4} \cdot 0.81 \cdot 0.0258 \cdot \left(\frac{9.8}{2}\right)^2 \frac{5.725}{(1.291)^4} \times \cos 2\theta$$

$$u = 5.6 \cos\theta + 0.73 \cos 2\theta$$

for $\theta = 0$; $\cos\theta = 1$ $\qquad \cos 2\theta = 1$

$u = 5.6 + 0.73 = 6.33$ ft/sec $\qquad$ say 6.4 ft/sec

Acceleration $\frac{du}{dt}$ (see equation page 3)

for $\theta = 0$ $\qquad \sin\theta = 0$

$$\frac{du}{dt} = 0$$

Coefficient C_D

$U_{max} \times \frac{T}{D} = 6.4 \times \frac{7.75}{6.50} = 7.65$

from graph page 6

$C_D = 5.5.$

Reduced $C_D' = 0.6 \times 5.5 = \underline{3.3}$

$D = 6.5\ Ft^2/Ft$

Wave Force

$F = 3.3 \times 6.5 \times 6.4 \times 6.4 = 900\ \#/Ft.$

Using magnification factors (see page 4)

$F = 1.33 \times 1.5 \times 900 = \underline{1,800}\ \#/Ft.$

Horizontal Wave Forces

Lake side - Standing Wave (Clapotis)

Wave characteristics

$d = 41.5$ $H = H_1 = 19.54'$ use $H = 19.6'$ $T = 7.75$ sec

$L_0 = 308'$ $\frac{d}{L_0} = 0.135$; $\frac{d}{L} = 0.1708$ $L = 244'$

$\sigma = 0.81$ $k = 0.0258$ $kd = 1.073$

$\sinh kd = 1.291$

$\coth kd = 1.265$

$\cosh kd = 1.633$

Distance of surface particle (equation page 3)

$$y = \frac{19.6}{2}\cos\theta + \frac{1}{2}\cdot 0.0258\cdot\left(\frac{19.6}{2}\right)^2\left(1+\frac{3}{2\cdot 1.291^2}\right)\cdot 1.265\cdot\cos 2\theta$$

$$y = 9.8\cos\theta + 3.0\cos 2\theta$$

for $\theta = 0$

$y = 12.8'$ $d+y = 41.5+12.8 = 54.3$

$k(d+y) = 0.0258\cdot 54.3 = 1.39$

$\cosh k(d+y) = 2.148$

$\cosh 2k(d+y) = 8.236$

$\sinh k(d+y) = 1.901$

Velocity u

$$u = 0.81 \times 9.8 \times \frac{2.148}{1.291} \times \cos\theta + \frac{3}{4} \times 0.81 \times 0.0258 \times 9.8^{-2} \times \frac{8.236}{(1.291)^4} \cos 2\theta$$

$$u = 13.2 \cos\theta + 4.41 \cos 2\theta$$

for $\theta = 0$ $\cos\theta = \cos 2\theta = 1$

$u = 13.2 + 4.41 = \underline{17.61}$ Ft/sec

Acceleration $\frac{du}{dt}$ see page 3

for $\theta = 0$ $\sin\theta = 0$; $\frac{du}{dt} = 0$

Coefficient C_D

$$u_{max} \frac{T}{D} = 17.61 \times \frac{7.75}{6.50} = 21.0$$

(from graph page 6); $C_D = 3.5$

Reduced $C_D' = 0.6 \times 3.5 = \underline{2.1}$

Wave Force

$F = 2.1 \times 6.5 \times 17.61 \times 17.61 = 4,250$ #/Ft

Using magnification factors - (see pages 4 & 5)

$F = 1.33 \times 1.50 \times 4200 = 8,500$ #/Ft.

Vertical Wave Forces

Gulf Side - Normal Wave

Wave characteristics - see page 9

Distance of surface particle

$y = 4.85 \cos\theta + 0.61 \cos 2\theta$

for $\theta = 0°$ $\quad y = 5.46$

Max. vertical velocity at $\theta = 90°$; $y = 0$

$d + y = d \quad ; \quad k(d+y) = kd$

$\sinh k(d+y) = \sinh kd = 2.59$

$\sinh 2k(d+y) = \sinh 3.36 = 14.38$

Vertical Velocity v
(see equation page 3)

$$v = 1.08 \times 4.85 \cdot \frac{2.59}{2.59} \sin\theta + \frac{3}{4} \cdot 1.08 \cdot 0.0392 (4.85)^2 \frac{14.38}{(2.59)^4} \sin 2\theta$$

$v = 5.25 \sin\theta + 0.22 \sin 2\theta$

for $\theta = 90°$

$v = 5.25$ ft/sec

Vertical acceleration $\frac{dv}{dt}$

(see equation page 3)

$$\frac{dv}{dt} = -(1.08)^2 \, 4.85 \frac{2.59}{2.59} \cos\theta - \tfrac{3}{2} \cdot 0.0392 \cdot 1.08^2 \frac{14.38}{(2.59)^4} \cos 2\theta$$

for $\theta = 90°$

$\frac{dv}{dt} = 0 + 0.022$; can be disregarded

Coefficient C_D: use same graphs as for horiz. force

$v_{max} \frac{T}{D} = 5.25 \cdot \frac{5.8}{6.5} = 4.7$

from graph page 6 - $C_D = 7.0$

Reduced $C_D = 0.9 \cdot 7.0 = 6.3$

Projected area $D = 5.5 \; Ft^2/Ft.$

Vertical Wave Force

$F = \frac{2}{2} \times 6.3 \cdot 5.5 \times 5.25 \times 5.25 = 960 \; \#/Ft.$ (6.3 corrected to 4.3)

Using magnification factors (see page 4)

$F = 1.33 \times 1.5 \times 960 = 1920 \; \#/Ft.$

Vertical Wave Forces

Gulf Side – Standing Wave-(Clapotis)

Wave characteristics - see page 13

$H = 19.4$ ft.

Vertical velocity $v = 1.08 \times \frac{19.4}{2} \cdot \frac{2.59}{2.59} \cdot \sin\theta$

for $\theta = 90°$ $v = 10.5$ ft/sec

Coefficient C_D

$v_{max} \frac{T}{D} = 10.5 \frac{5.8}{7.4} = 8.3$; from graph - $C_D = 4.5$

Reduced $C_D = 0.9 \times 4.5 = 4.05$

Vertical Wave Force

$F = \frac{2}{2} \times 4.05 \times 5.5 \times 10.5 \times 10.5 = 2460$ #/Ft

Using magnification factors

$F = 1.33 \times 1.5 \times 2460 = \underline{4920}$ #/Ft.

Vertical Wave Forces

Lake Side - Normal Wave

Wave characteristics - see page 16

$y = 0$ Vertical velocity; $v = 0.81 \times \frac{9.8}{2} \times \frac{1.291}{1.291} \times \sin\theta$

for $\theta = 90°$; $\sin\theta = 1.0$ $v = 4.0$ Ft/sec.; $\frac{dv}{dt}$ is negligible

Coefficient C_D

$v_{max} \frac{T}{D} = 4.0 \frac{7.75}{7.4} = 4.2$; from graph (page 6) $C_D = 7.0$

Reduced $C_D = 0.9 \times 7.0 = \underline{6.3}$

Projected area: D

Girder solid -	5.0 Ft²
Grating 6 ft × 0.4 (open)	2.4
D =	7.4 Ft²

Vertical Wave Force

$F = \frac{2}{2} \times 6.3 \times 7.4 \times 4.0 \times 4.0 = 750$ #/Ft.

Using magnification factors (page 4)

$F = 1.33 \times 1.5 \times 750 = \underline{1500}$ #/Ft.

Fig. II - 32

Vertical Wave Forces

Lake Side - Standing Wave - (Clapotis)

Wave characteristics: see page 20

$H = 19.6$ ft ; $y = 0$

Vertical velocity $\qquad \frac{dv}{dt}$ is negligible

$v = 0.81 \times \frac{19.6}{2} \times \frac{1.291}{1.291} \sin\theta$; for $\theta = 90°$; $v = 8.0$ ft/sec.

Coefficient C_D

$v_{max} \frac{T}{D} = 8.0 \frac{7.75}{7.4} = 8.4$ from graph (page 6) $C_D = 4.5$

Reduced $C_D = 0.9 \times 4.5 = \underline{4.05}$

Projected area $\qquad D = 7.4$ Ft²/Ft.

Vertical Wave Force

$F = \frac{2}{2} \times 4.05 \times 7.4 \times 8.0 \times 8.0 = 1{,}920$ #/Ft.

Using magnification factors

$F = 1.33 \times 1.5 \times 1920 = \underline{\underline{3840}}$ #/Ft

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT Rigolets Control Structure
Wave Forces on Girders
COMPUTED BY JLK CHECKED BY

SHEET NO. 27 OF
JOB NO. 18-111-02
DATE May 1972

Summary
Wave Forces on Girders

	Unit	Gulf Side		Lake Side	
		Normal Wave	Clapotis	Normal Wave	Clapotis
Wave Height	Ft	9.7	19.4	9.8	19.6
Wave Length	Ft	160	160	244	244
Horiz. Force	Lbs/Ft	2,100	13,200	1,800	8,500
Vert. Force	Lbs/Ft	1,920	4,900	1,500	3,840

Wave Forces
FOR STRUCTURAL DESIGN

LOADING / % OF MAX CLAPOTIS FORCE	Horiz. Load on Girder Lbs/Ft	Vertical Load	
		Girder Lbs/Ft	Grating Lbs./S.F.
60	8,000	2,900	400
100	13,200	4,900	700

Independent check

Horizontal Wave Forces

Max. wave pressure - Empirical method by Molitor (Ref: Quinn - Design & Construction of Ports & Marine Structures)

Max pressure on a vertical wall:

$$p_1 = \frac{k \gamma_w}{2g} (v + v_0)^2 \quad \#/Ft^2$$

where:

k = coefficient = 1.7 for ocean storm waves

γ_w = specific weight of water 64 Lbs/c.F.

g = 32.2 Ft/sec²

v = velocity of wave propagation = $2.26\sqrt{L}$

v_0 = max orbital velocity ft/sec = $7.11 \cdot H/\sqrt{L}$

H = wave height Ft

L = wave length Ft.

For Lake side: $H = 19.54'$ (Clapotis)

$L = 244'$

S.W.L = El. +11.5

$\sqrt{L} = 15.6$

$v = 2.26 \times 15.6 = 35.4$ Ft/sec

$v_o = 7.11 \times 19.54 / 15.6 = 8.9$ Ft/sec

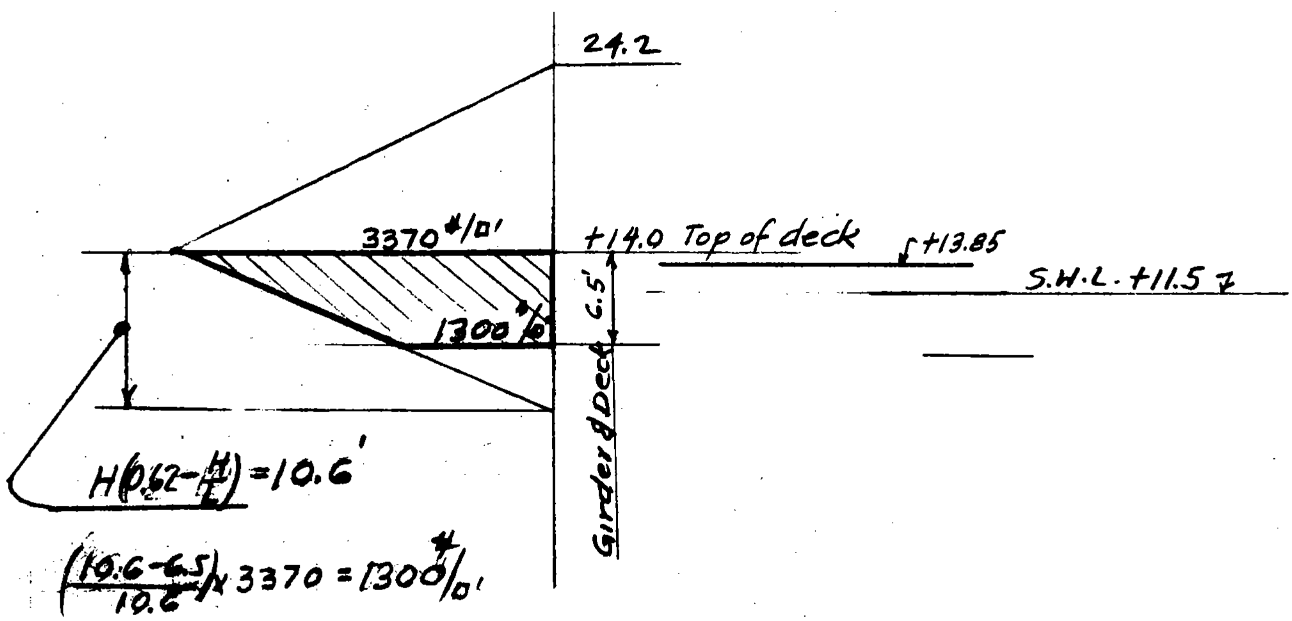
Location of max wave pressure:

$0.12H = 0.12 \times 19.54 = \underline{2.35'}$ - from still water level

+11.50
2.35
13.85

Max. wave pressure:

$$p_1 = \frac{1.7 \times 64.4}{2 \times 32.2} \times (35.4 + 8.9)^2 = \underline{3,370} \ \#/\square'$$



Total pressure on girder

$$P = \tfrac{1}{2}(3370 + 1300) \times 6.5 = \underline{15,000} \ \#/\text{Ft girder}$$

For Gulf side (clapotis)

$H = 19.4$ $L = 160$ ft. $\sqrt{L} = 12.65$ SWL = +12.8

$v = 2.26 \times 12.65 = 28.6$ Ft/sec

$v_0 = 7.11 \times 19.4 / 12.65 = 10.9$ Ft/sec

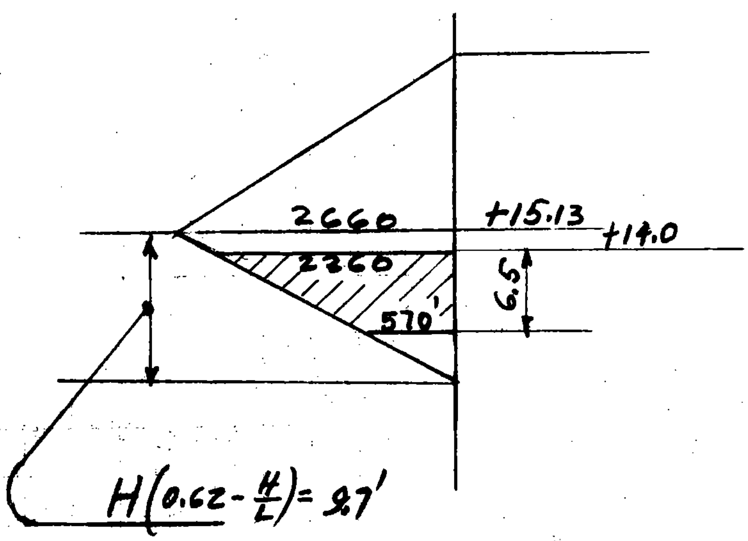
Location max. wave pressure

$0.12 \times 19.4 = \underline{2.33}$ from still water level

12.8
2.33
15.13

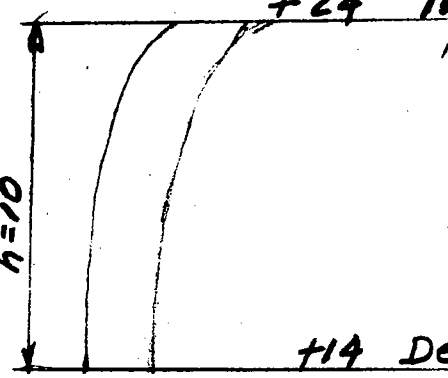
Max. wave pressure:

$$p_1 = \frac{1.7 \times 64.4}{2 \times 32.2}(28.6 + 10.9)^2 = \underline{2660} \ \#/\square'$$



$$P = \tfrac{1}{2}(2360 + 570) \times 6.5 = \underline{\underline{9,500}} \ \#/\text{Ft}$$

Approximate determination of the Vertical Clapotis force on roadway grating.



Assuming the grating is completely blocked (solid surface); the pressure on grating as a function of velocity; $v^2 = 2gh$; $h = \frac{v^2}{2g}$

Energy = pressure ; $p = W\frac{v^2}{2g} = \underline{Wh}$; $W = 64\#/cF.$ $h = 10 ft.$

$p = 64 pcf \times 10 ft = \underline{\underline{640 psf.}}$

Since the grating is approximately 50% open; the pressure should be reduced accordingly. But the friction of water passing through the grating will produce a drag force which will contribute to the pressure.

The drag force can be considered as equivalent to a waterhead loss.

The head loss as used in the design of trash racks; can be determined by the following equation: (Reference Davis - Handbook of Applied Hydraulics see reproduction on page 34.)

$$H_R = \beta \cdot \sin\alpha \cdot \frac{1.33d}{a} \cdot \frac{v^2}{2g}$$

where:

β = flow coefficient, depending on the shape of the grating bars; for common grating $\beta = 2.2$

α = angle between grating and flow of water = 90°

d = maximum bar thickness = 0.375 inches

a = clear space between grating bars = 1.0 inch

$\frac{v^2}{2g} = h$ = water head = 10 ft.

$$H_R = 2.2 \times 1.0 \frac{1.33 \times .375}{1.0} \times 10 = 11.0'$$

It appears from the above, that the

head-loss resulting from the drag is greater than the actual head of 10 ft.

$11 \times 64 \cong 700$ p.s.f.

This pressure $p = 700$ psf. will be used in the design of grating.

[illegible] with 0.5 per cent of reinforcing steel supports the structure. No [illegible] was used. The intake bulkhead wall separating the generator room from the headwater also contains 0.5 per cent of steel in a horizontal direction. The watertightness of the wall has been satisfactory.

Vertical-lift wheeled gates were provided for the Petenwell intake. The gates are of all-welded construction and the wheels are equipped with roller bearings to reduce hoisting effort and to eliminate by-pass or filler valves. The gantry crane is

DESIGN DATA FOR TRASH RACKS

SPACING OF VERTICAL BARS

Type of turbine and location of racks	Space between bars
Impulse turbines	$\frac{1}{5}$ of nozzle diameter
Racks in front of long pipe lines	$\frac{3}{4}$ in. *
Medium size Francis and Kaplan turbines	2 to 3 in.
Large size Kaplan turbines	3 to 6 in.

*For high head installation with pipe line, a mesh rack is recommended instead of bars.

FORMULA FOR HEAD LOSS IN TRASH RACK

$$H_R = \beta \sin \alpha \frac{1.33 d V_0^2}{2ag}$$

where H_R = friction loss, feet
- β = flow coefficient (see table)
- d = maximum width of bar, inches
- a = space between bars, inches
- V_o = water velocity upstream from rack, feet per second
- α = angle between rack and direction of flow, degrees
- g = 32.2, acceleration due to gravity, feet per second²

*β, flow coefficient, for bar shapes shown with a ratio $\frac{l}{d} = 5$, where l = length (in direction of water flow) and d = with (horizontally across direction of flow.)

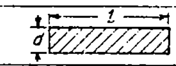
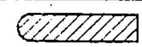
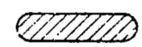
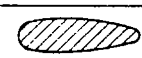
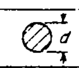
* β	Bar shapes (cross section)
2.42	
1.83	
1.67	
1.03	
0.92	
0.76	
1.79	

FIG. 38.—Design data for trash racks.

capable of lifting the gate against full unbalanced pressure to permit filling the spiral case. The head gates are heavy enough to close with flow through the units, which would be required if the turbine wicket gates became inoperative. In case of emergency, an additional gate slot is provided immediately upstream of the service slot to permit lowering of an extra gate in lieu of stop logs.

The trash racks, of all-welded construction, are built in sections which are self-supporting and can be removed entirely. The trash racks are designed to resist full headwater pressure, in case they become completely blocked by anchor ice or trash. The stress in the steel for this condition is 27,000 psi.

DAVIS. HANDBOOK OF APPLIED HYDRAULICS

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT RIGOLETS COMPLEX
COMPUTED BY JS CHECKED BY

SHEET NO. 1 OF 1
JOB NO. 18-111-02
DATE 12-15-71

FIGURE II-42

FLOW VELOCITIES AT CONTROL STRUCTURE DURING GATE CLOSURE

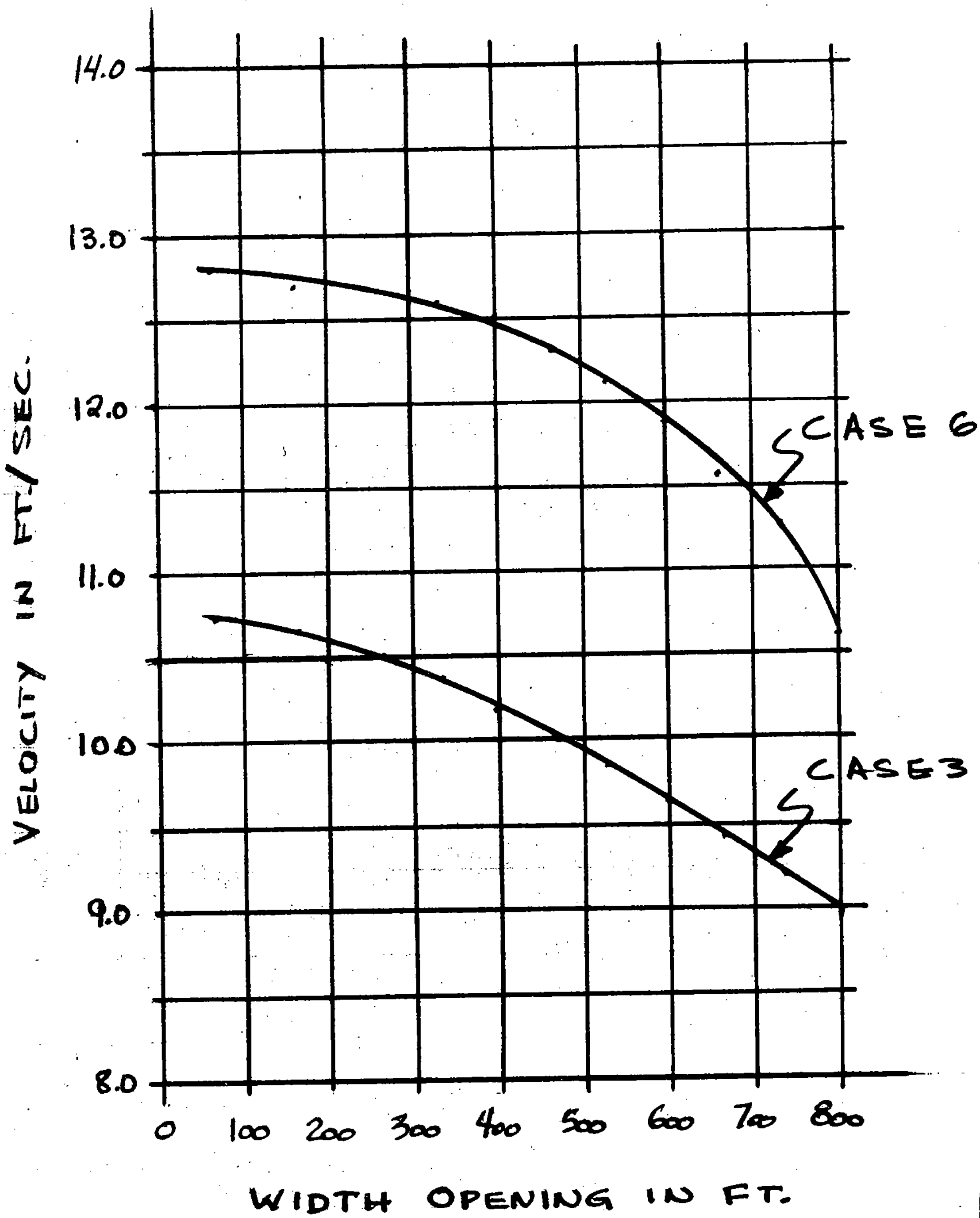


Fig. II - 42

Hydraulics at Control Structure

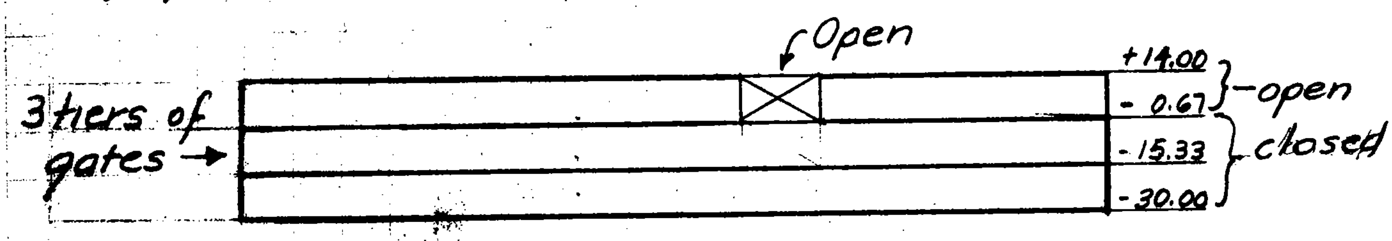
Investigate flow velocities resulting from partial control-gate closures.

Case 3 (See Table 2 - Design Data)

Lake side - Water Elev. +1.0 }
Gulf side - " " +5.0 } Head differential $h = 4.0$ ft.

Case 6

Lake side - water Elev. +2.5 }
Gulf side - " " -3.0 } $h = 5.5$ ft



A) Upper gate(s) open

The flow velocity through the opening will be:

Case 3; $v_1 = C\sqrt{2gh} = {}^{\oplus}0.7\sqrt{2 \cdot 32.2 \cdot 4.0} = \underline{11.2\ fps.}$

Case 6; $v_2 = \quad = 0.7\sqrt{2 \cdot 32.2 \cdot 5.5} = \underline{13.1\ fps.}$

⊕ The coefficient of discharge "C" is given by V.T. Chow in "Open Channel Hydraulics".
For most constructions "C" is less than 0.8; Use 0.7

Flow velocity check by Computer

Computer runs for flow velocities at the Control structure; produced the following results:
(See Computer Run summary pages)

Run 1. – Flow velocities during closure of Contr. Struct.
– **Case 3** (per Table 2 - Design Data)
Water Elevations : Gulf Side +5.0 Ft.
Lake Side +1.0 Ft.

Max. Velocity – **10.7** f.p.s.

Run 2. – Flow velocities during closure of Contr. Struct.
– **Case 6** (per Table 2 - Design Data)
Water Elevations : Gulf Side -3.0 Ft
Lake Side +2.5 Ft.

Max. Velocity **12.8** f.p.s.

Conclusion: A comparison shows that the values of Vmax, resulting from manual and computer calculations are pretty close.

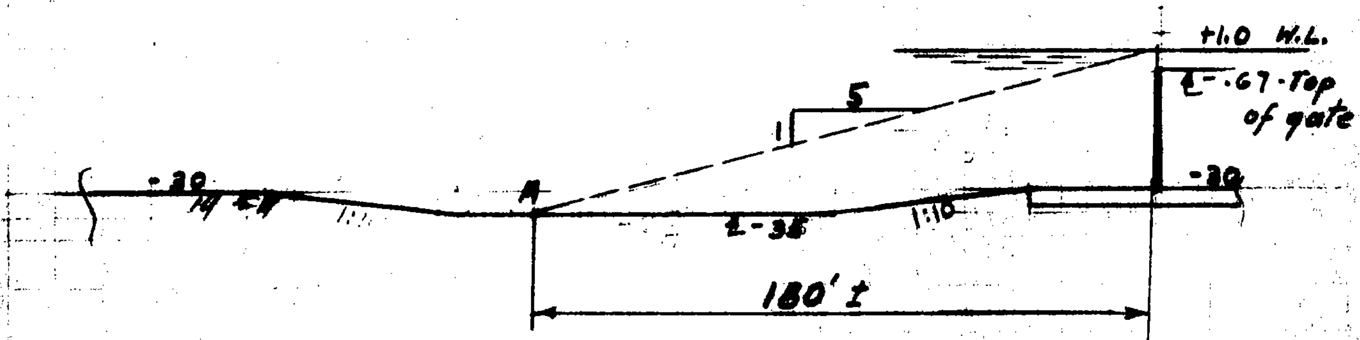
	Manual	Computer
Case 3	11.2 fps	10.7 fps
Case 6	13.1 "	12.8 "

The flow from the top gate will be diffused in both directions; horizontally and vertically.

According to " Rouse, Engineering Hydraulics" - -chapter 25: The Mechanism of Diffusion - The zone of diffusion expands at an angle of about 12°; about 1:5

The resulting flow pattern assumes an approximate shape as shown on the sketch.

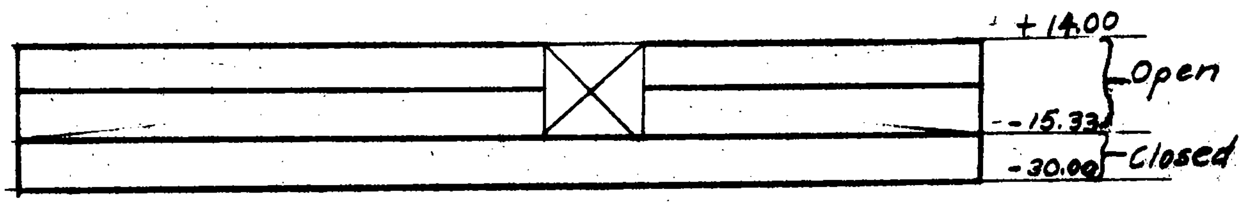
Vertical Diffusion Pattern



At a section beyond point A; the average velocity near the bottom of the channel will be not greater than $V_1' = 11.2 \cdot \frac{1.67}{36.0} = 0.52$ f.p.s. (for Case 3)

The zone of diffusion will end at an approximate distance of $x = \frac{35+1}{\tan 12°} = 180'\pm$

<u>When any number of 2 upper gates are open</u>



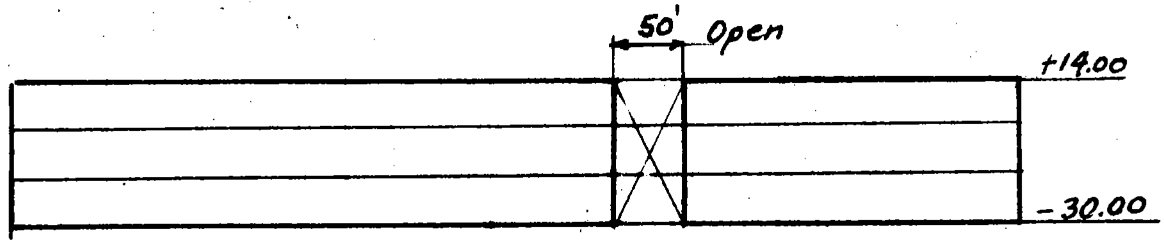
The flow velocity through the opening will be approximately

$$v_2 = 11.2\,\frac{16.33}{31.00} = \underline{5.9\ fps}$$

This flow velocity will be diffused in a similar pattern as for one open tier.

The diffused velocities will be accordingly; much lower.

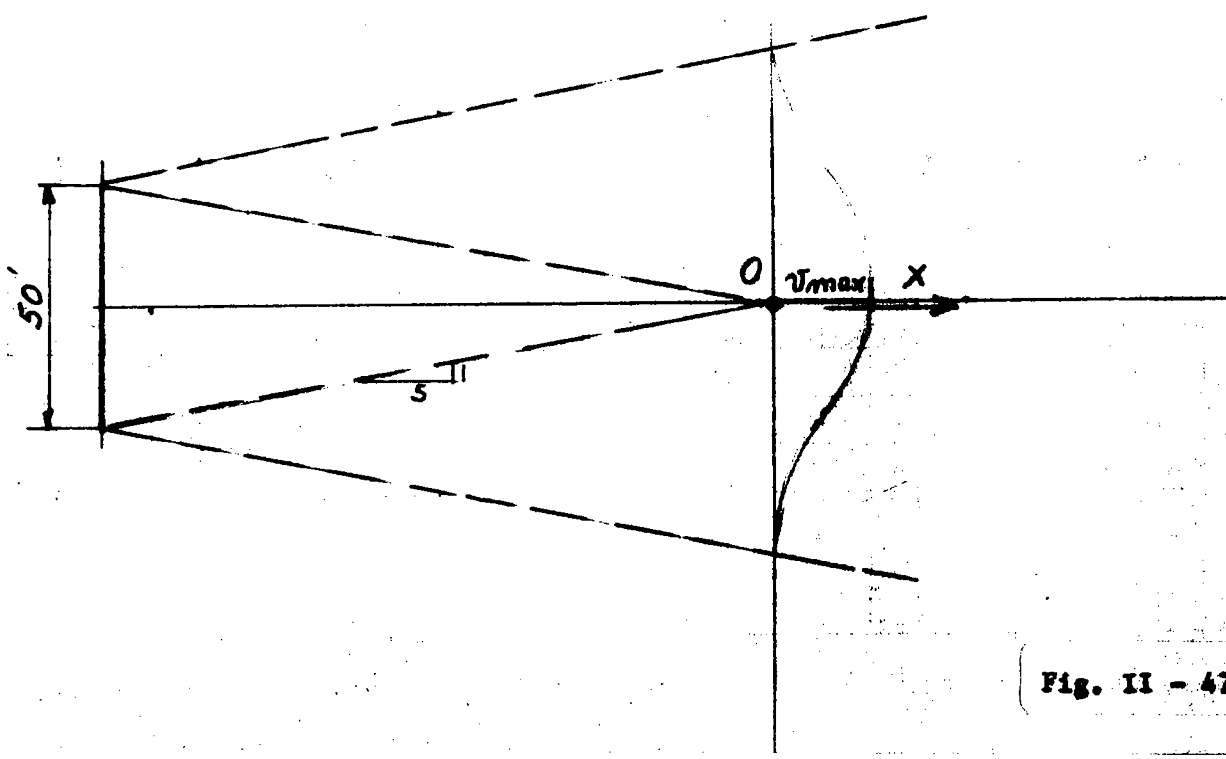
C) When all 3 gates are open in any one bay



The velocity at the channel bottom near the control structure will be the same as for A)

Case 3 $v = 11.2$ f.p.s. | Case 6; $v = 13.1$ f.p.s.

The effect of this velocity will diffuse horizontally and will extend into the channel a distance of about 125 ft, point "O"



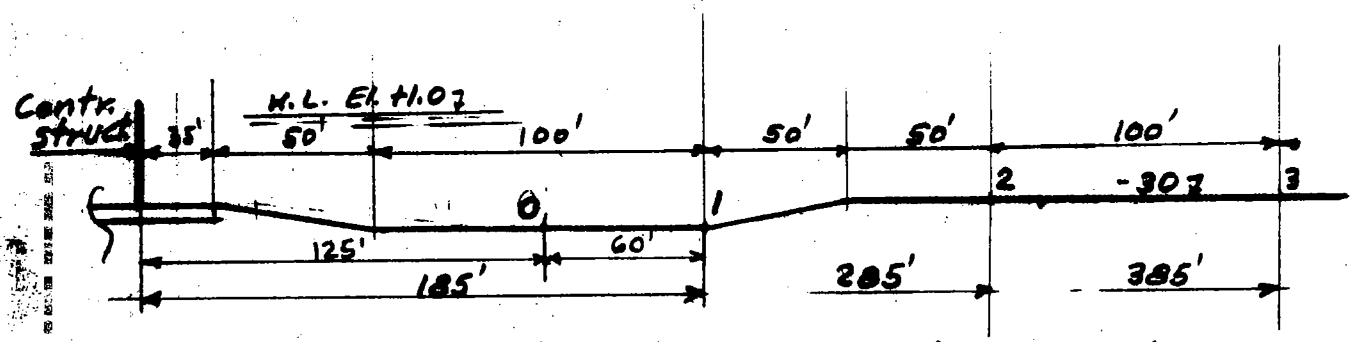
The velocity beyond point "O" will decrease in a linear pattern;

The approximate relationship of the velocity change can be expressed by the following equation:

$$V = V_{max} \cdot \frac{100}{100 + 0.4X} \qquad V_{max.} = 13.1 \text{ f.p.s.} \quad (\text{Case } 6)$$

where X = the distance beyond point "O" (125' from struct.)

(see Reference: Rouse Eng. Hydraulics)



Velocities along channel

Point	Distance (ft) from Control Structure	Distance from "O" X-ft.	$\frac{100}{100+0.4X}$	V f.p.s.	V average f.p.s.
1	185	60	0.8	10.5	5.25
2	285	160	0.61	8.0	4.0
3	385	260	0.49	6.4	3.2

The velocity will also diffuse vertically beyond point 0; at an approximate slope of 1:5.

Theoretically the velocity effect at the channel bottom (El. -30) should disappear at a distance approx. 5×31 = 155', which is at about point "2".

To provide an extra safety margin, the channel lining will be extended another 100 ft to point "3".

For the sake of simplicity, the thickness of channel lining and weight of stones will be determined on the basis of horizontal diffusion only; disregarding the vertical diffusion.

This procedure is on the safe side.

Scour Protection

To prevent scouring of the channel in the area of relatively high flow velocity, stone lining shall be provided for protection.

The weight and size of stones will be determined by a method used in "U.S. Army Coastal Eng Research Center - Technical Report #4 Shore Protection Planning & Des. 3d Edition - 1966"

The relationship of the stone weight to the flow velocity is expressed by equation 4-64 page 303;

$$W = \gamma \frac{V^6 W_s W_w^3}{10^5 (W_s - W_w)^3 (\cos\alpha - \sin\alpha)^3}$$

Where: W = weight of the stable stone in pounds

V = water flow velocity in ft. per sec.

W_s = unit weight of stone ~ 165 #/ft³

W_w = " " of water ~ 63 #/ft³

α = angle of internal friction of material (assume 1:3)

γ = coefficient = $\frac{1}{15.23}$ for embedded stone

= $\frac{1}{2}$ for nonembedded stone

The same relationship is also presented in a graph form shown on figure 4-27, page 305

The weights of stable stones, both: embedded and nonembedded are presented in the following table.

The values are determined from graph 4-27 for a slope 1:10

Distance from Control-structure ft.	Max. Velocity f.p.s.	Embedded Stone		Nonembedded Stone		Required Thickness of Lining ft ⊕
		Weight Lbs	Equivalent Diam. ft.	Weight (W_{50})	Equivalent Diam. ft.	
125	13.1	180	1.3	1,350	2.5	4.0
185	10.5	50	0.8	380	1.6	2.5
285	8.0	10	0.5	75	0.9	1.5
385	6.4	2.5	0.3	20	0.6	1.0

⊕ The thickness is based on nonembedded stone (Riprap.), and is determined by the following equation

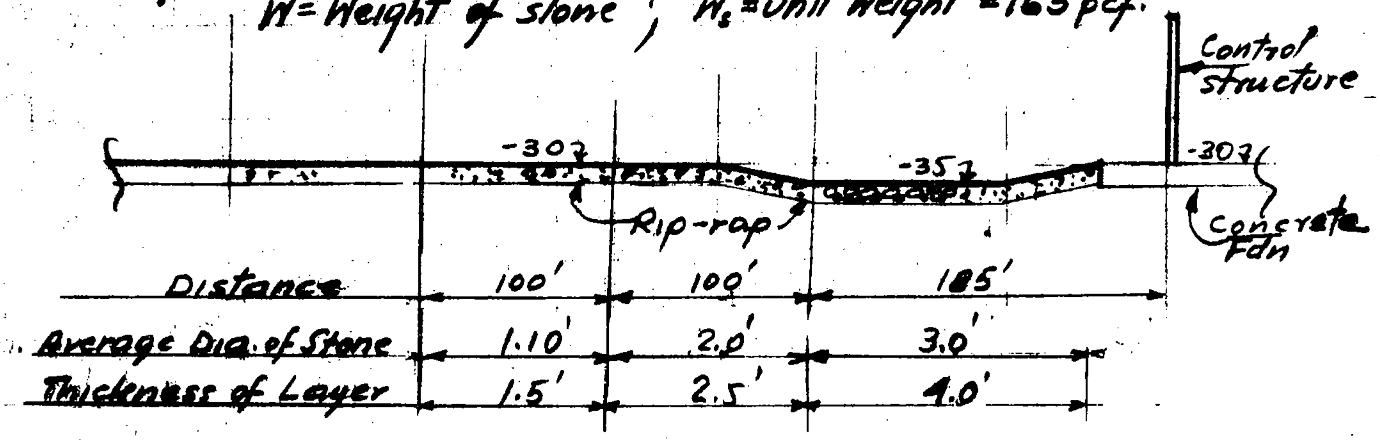
$$r = n K_\Delta \left(\frac{W}{W_r}\right)^{\frac{1}{3}}$$ (Reference CERC-Report #4- Eq. 4-35)

Where: r = thickness
n = number of layers; use 2
K_Δ = layer coefficient = 1.0
W = weight of stone
W_r = unit weight of stone = 165 pcf.

Fig. II - 51

Equivalent stone diameter $d_g = 1.24 \sqrt[3]{W/W_s}$

W = Weight of stone ; W_s = unit weight = 165 pcf.



Distance from Contr. Str. Ft	Thickness Ft	W50		Wmax.		Wmin.		Remarks
		W Lbs	d_g ft	W Lbs	d_g ft	W Lbs	d_g ft	
125	4.0	1,350	2.5	2,700	3.0	300	1.5	
185	2.5	380	1.6	760	2.0	85	1.0	
285	1.5	75	0.9	150	1.1	17	0.6	

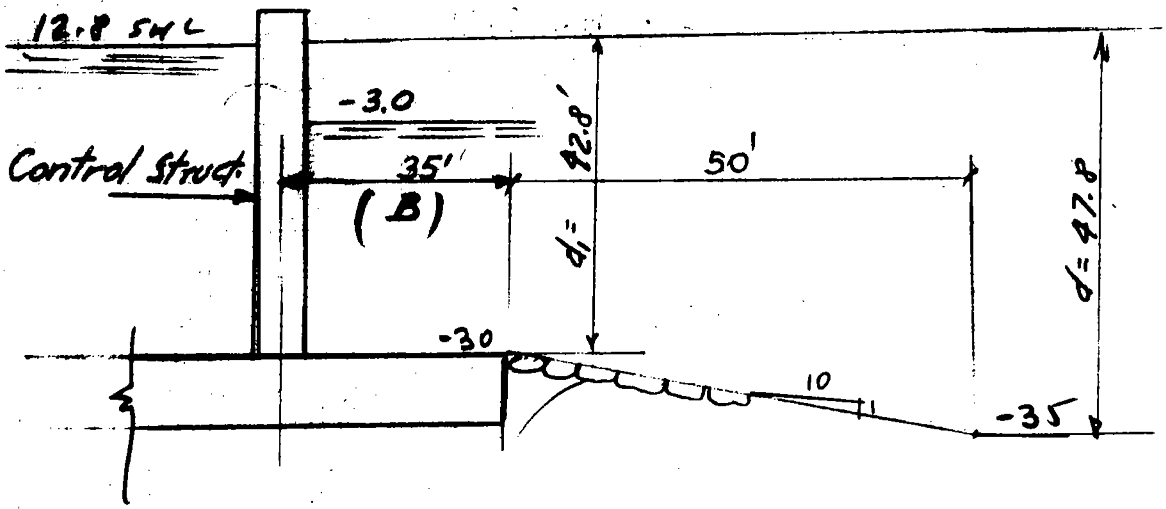
The channel lining shall consist of graded stone. W50 shall constitute 50% of the material. The remaining stone shall range in weight from Wmax = 2.0 W50 to ~~Wmin~~ = 0.22 W50

Fig. II - 52

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT: Control Structure — Fdn. Toe Stability
COMPUTED BY: JCK   CHECKED BY:
SHEET NO. CM#1 OF
JOB NO. 18-111-02
DATE June-1972

Effect of Wave Action on Foundation Toe Stability



Equation 4-37 (page 277) (Reference: U.S. Army. CERC - Technical Report #4)

$$W = \frac{W_r H^3}{N_s^3 (S_r - 1)^3}$$

W = Weight of armor ; W_r = unit weight of rock

H = Design wave height

$S_r = \frac{W_r}{W_w} = \frac{W_r}{W_w} = \frac{150}{64} = 2.34$; $(S_r - 1)^3 = 1.34^3 = 2.23$

N_s = design stability number for toe protection to be determined from Fig. 4-12 (page 279)

Toe stability (cont'd)

Stability Number (N_s^3) - Fig. 4-12

$\frac{d_1}{d} = \frac{42.8}{47.8} = 0.9$

Max (N_s^3) value (for $\frac{d_1}{d}$; 0.75) = 300

Use (N_s^3) = 300 which is on safe side

check min. width of berm B on chart.

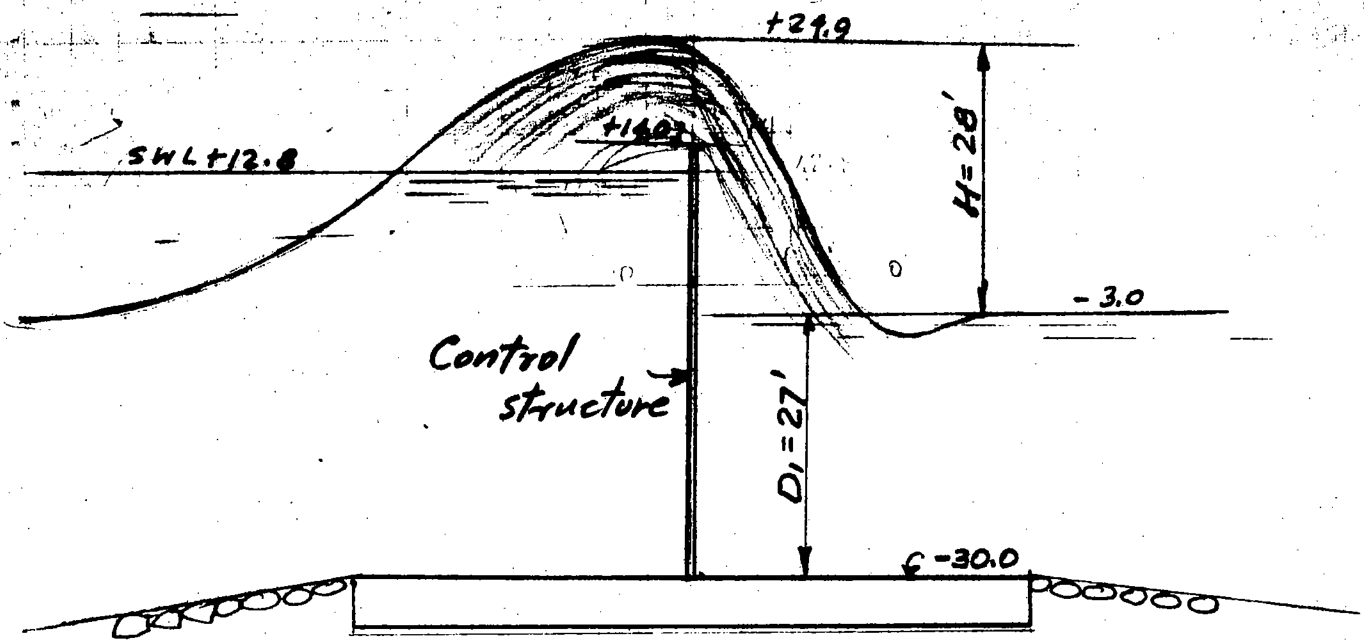
$B = 35'$; $\frac{35}{47.8} = 0.74d > 0.4d$ O.K.

$W = \frac{150 \times 7^3}{300 \times 2.2^3} = 55^{\#}$

Conclusion

The above figures indicate that the weight of stones at the toe is not governed by the stability requirements.

As shown on preceding pages the stones req'd for protection against scour from high flow velocity are much larger. See page A7.



Scouring Effect of Water Overfall

Reference: Handbook of Applied Hydraulics (p. 296) by C. V. Davis - McGraw Hill, 2nd edition

In case of wave overtopping during a hurricane condition, the maximum drop will be $H = 28$ ft.

The depth of tail water $D_1 = 27$ ft.

The dissipation of energy of the falling water is expressed by the following equation:

Energy dissipated $E_D = \frac{6.25\left(\sqrt{1+16\frac{H}{D_1}}-3\right)^3}{\left(1+\frac{H}{D_1}\right)\left(\sqrt{1+16\frac{H}{D_1}}-1\right)}$ (in %)

(page 281)

for $\frac{H}{D_1} = \frac{28}{27} = 1.04$; $E_D = 1.4\%$

(The percentage of dissipated energy; increases with the increase of $\frac{H}{D_1}$.; for $\frac{H}{D_1} = 30$; $E_D = 65\%$)

The low percentage of energy dissipation, indicates that the water fall causes little local disturbance due to the large depth of the tail water.

As a result the flow diffuses slowly; in a pattern similar to sketch shown on page 32.

Consequently, no scouring is likely at the foot of the control gates.

Abutment Slope Protection

The abutment slopes vary from 1:4 to 1:30. They are exposed to wave action from many directions. In addition the slopes may also be affected by the flow velocities generated by the water level differences when some control gates are open.

Wave Action

Slope protection calculations for the closure dam, revealed that for a slope of 4:1 and for a shape factor coefficient $K_D = 1.3$; the required weight of stable armor stone is 1,550 # — (see page C-1, in Design Section for Closure Dam)

Due to the abutment configuration, most of the waves will act at an angle ≠ 90°.

The resulting perpendicular component will be less than the force acting on the closure dam

Abutment slope (cont'd)

To provide an adequate safety margin, for the 1:4 slope, 36" derrick stone will be used.

Flow Velocity

The max flow velocity through the gate openings is 13.1 fps. (see page A2)

From Graph; - Fig. 4-27, page 305 (Technical Report #4)

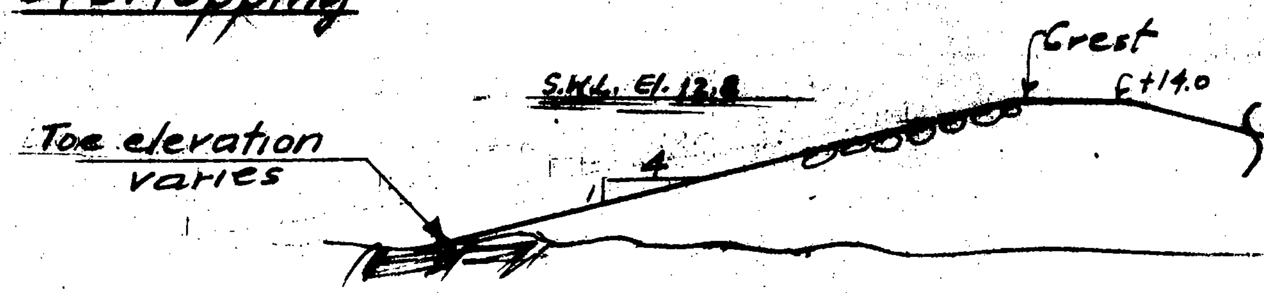
By interpolation:

For nonembedded stone - slope 1:3.4 and for velocity = 13.1 f.p.s.

Required weight of stone is about 1400 Lb.

Conclusion 36 in derrick stones will provide more then 50% safety margin

Overtopping



Gulf side

check for aver. highest 10% of waves

$H_o = H_{10} = 7.37$ ft. $T = 5.8$, $L_o = 172'$

Breaking height & Breaking depth of wave.

Bot. Elevation at toe	-6	-12	-18	-24
Depth from still water level (+12.8) d (ft)	18.8	24.8	30.8	36.8
d/L_o	0.109	0.138	0.179	0.214
Shoaling coefficient $\frac{H}{H_o'}$ (from Table D-1)	0.9263	0.9150	0.9144	0.9215
Design wave $H_o' = H_o\left(\frac{H}{H_o'}\right) = 7.37 \cdot \left(\frac{H}{H_o'}\right)$	6.85	6.75	6.70	6.80
H_o'/L_o	0.040	0.0393	0.0389	0.0386
From Fig. 1:82 H_b/H_o'	1.0	1.02	1.03	1.04
Height of breaking wave H_b ft.	6.85	6.90	6.90	7.1

Fig. II - 58

Crest Elevation

Use $H_0' = 6.8$ ft. average

$$\frac{H_0'}{T^2} = \frac{6.8}{5.8^2} = 0.203$$

Water depth – d	18.8	24.8	30.8	36.8
$\frac{d}{H_0'}$	2.76	3.63	4.51	5.40
R/H_0' – from Fig 3-1 & 3-2 for slope 1:6	0.86	0.9	0.9	0.9

Runup $R = 0.9 \times H_0' = 0.9 \times 6.8 = 6.1'$

The theoretical top of levee required, to prevent overtopping; would be Elev. 12.8+6.1 = +18.9

The crest elevation was set at elev. +14.0; see "General Design Memorandum No 2" - Plate 7.

It is obvious that overtopping will occur and will cause a water build-up on the opposite side of the levee.

This condition was anticipated. (See paragr. 17 same reference)

6. Hydraulic Computations.

MATHEMATICAL MODEL FOR TWO-DIMENSIONAL FLOW IN OPEN CHANNELS AND VERTICALLY MIXED IN FLOW IN TIDAL ESTUARIES AND RESERVOIRS

(with special provision for flow through constricted openings and flow over submerged closure dams and highways).

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INTRODUCTION

This is a general description of a mathematical model for computing depth, velocity and rate of flow in two dimensions. It is based on the theoretical motion of unsteady open channel flow. The theory is applied by decomposing the area of interest into segemnts as illustrated in Figure 1. The resulting model is a conceptual or hypothetical model of the real system. It is a deterministic model because all of the input data are given functions of time or are related uniquely to the physical properties of the catchment.

A general system for creating a two-dimensional model of any flow system has been created as a FORTRAN program. This program requires data input which describes the physical features of the real flow system and which describes the occurence of changing water levels at some locations, the input of flow hydrographs at other locations, and the filling-in of closure dams or operation of control structures to regulate the flow between pairs of segments. The program prints out the water levels, velocities and rate of flow at selected points throughout the modeled area.

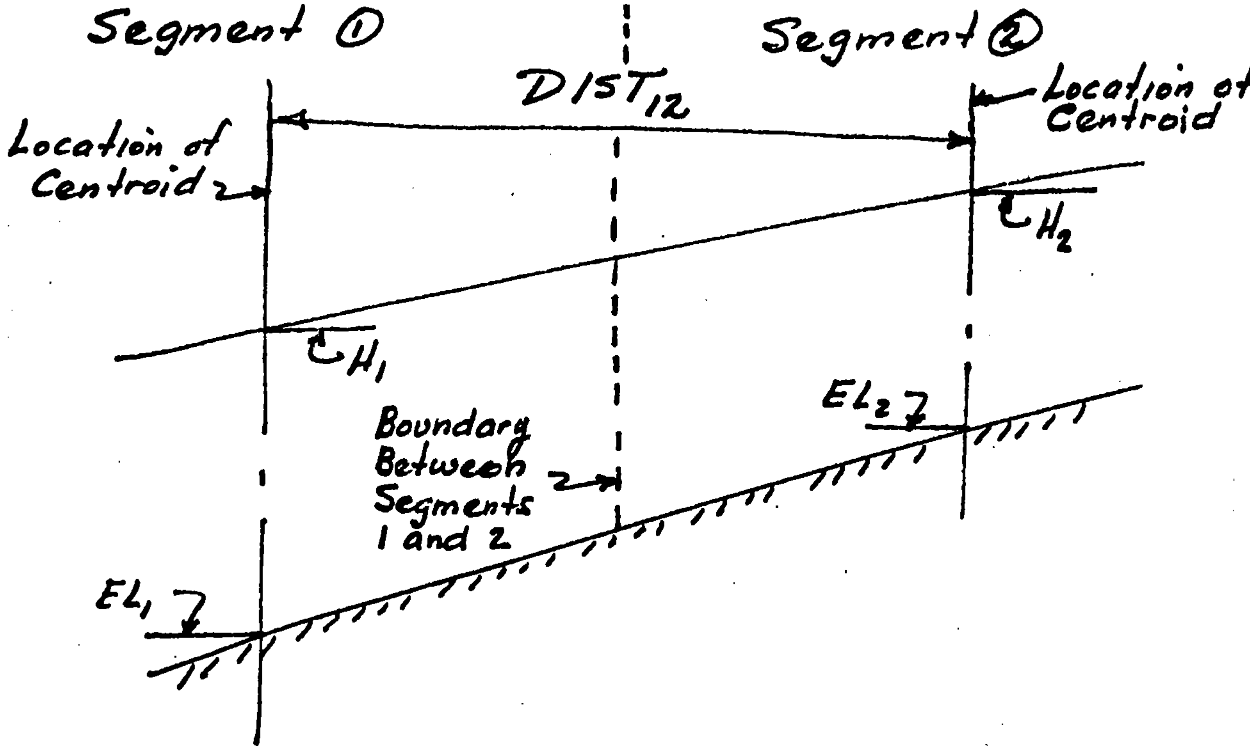
Extensive field data are not required to use this model although such data as are available may be used to establish appropriate boundary conditions or to determine how well the mathematical model is able to simulate the observed operation of the system. All parameters can be estimated on the basis of physical features alone, although field data can be used to adjust model perameter values if it is desired to improve the 'fit' of the model to the real system.

DETAILED MODEL DESCRIPTION

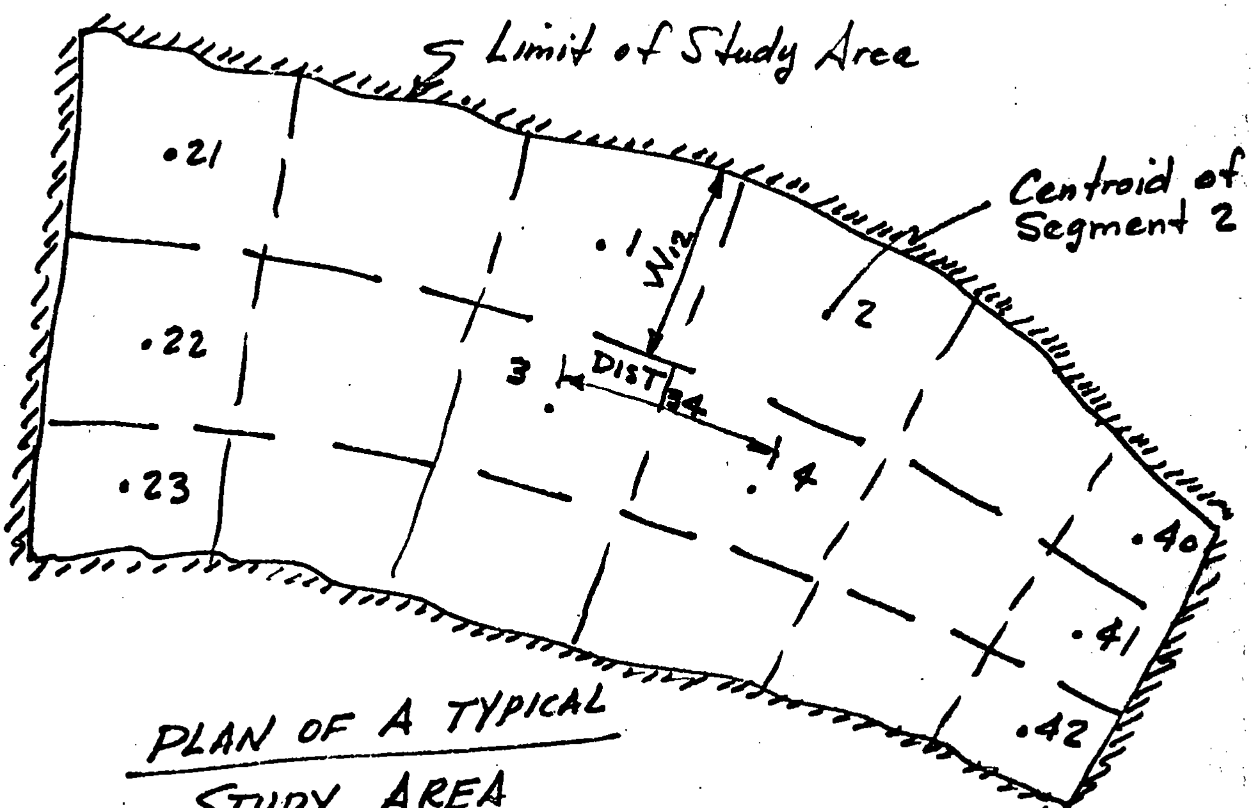
Data Input The input data supplies the program with information about the physical features of the catchment and the spatial and temporal properties of so-called boundary conditions such as water level changes at given locations, inflow hydrographs, and changes in the physical configuration of the boundaries between adjacent segemnts. The organization of the data deck is shown in Figure 2.

Before discussing how the theory of unsteady flow in open channels has been used to develop this model, the data required by the model will be presented.

The total input data set is divided into several parts as shown in Figure 2. Following is an explanation of the data in each part. The FORMAT specification for the data cards in each part are also given. (Note, the user should always punch a decimal point for all REAL variables so that the user's decimal point over-rides the decimal point location given in the FORMAT specifications.)



SECTION THROUGH SEGMENTS 1 AND 2



PLAN OF A TYPICAL STUDY AREA

Figure 1 Decomposition of a Study Area into Segments

(Revision to control card for Flow 2)

1) The following variable have been added

IH = 0 if computation are to begin from steady-state, given the initial boundary conditions

= 1 if computations are to begin from a transient condition, give the initial water levels and initial boundary conditions

NHP = 0 if the final water levels are not to be punched

= N if copies final water levels are to be punched

2) The format of HINIT has been changed to 10F8.0

3) The bridge data cards have been revised

Two cards now are required for each bridge

a) IBR (I), NBRSEG (I), (IBRSEG (I.J.K), J=1, NBRSEG (1))

(215, 1813)

b) (WOPEN (I.J), J=1, NBRSEG (I))

(95.0)

where

IBR (I) = external number of the I=th constriction

NBRSEG (I) = # of segment boundaries along the I - th constriction (9)

IBRSEG (I,J,K) = external number of the K-th segment (K=1,2) adjacent to the J-th segment boundary along the I-th constriction opening

LET NBR = # of constricted openings

NBRSEG = # of segment boundaries along the I-th constricted opening

IBRSEG (I,J,K) = external # of K-th segment (K=1,2) adjacent to the J-th segment boundary along the I-th constricted opening

IIB (I,J) = 0, if the J-th segment adjacent to segment I is not along a constricted opening

= L, if along the L-th constricted opening

KBRSEG (I,J,K) = internal index corresponding to IBRSEG (I,J,K)

JBRSEG (I,J,K) = value of I corresponding to the J the segment adjacent to segment KBRSEG (I,J,K)

Fig. II - 65

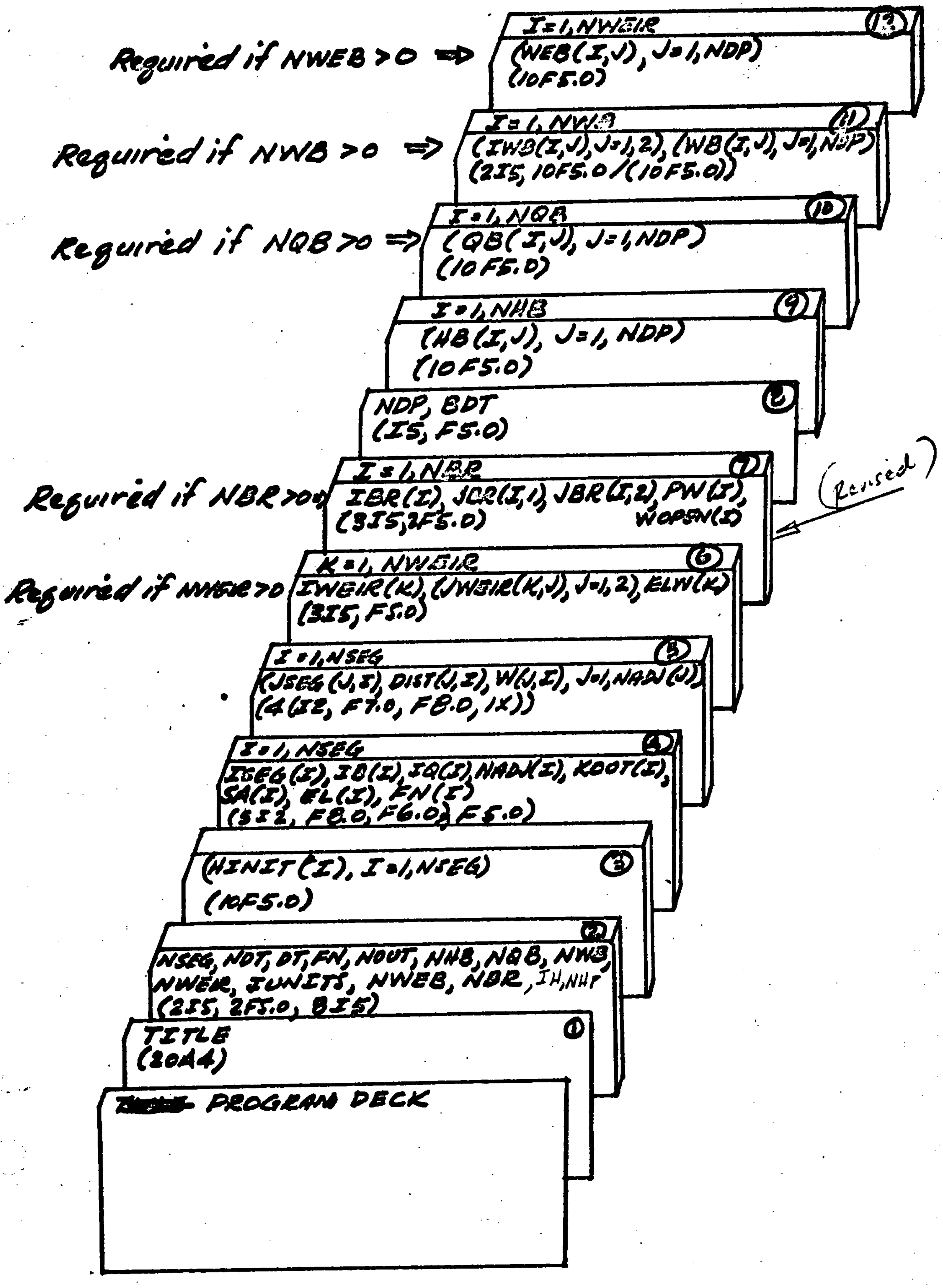


FIGURE 2 ORGANIZATION OF INPUT DATA CARDS

Data Input - Part 1 One card (20A4)

TITLE = 80-character heading to be printed at the top of each page

Data Input - Part 2 One card (2I5, 2F5.0, 8I5)

NSEG = total number of segments that the total area has been decomposed into, including boundary segments where water levels are to be given and boundary segments that receive inflow hydrographs.

NDT = total numbers of time steps to be executed

DT = duration of a time step, seconds

FN = Manning's "n", assumed constant for all seg nts

NOUT = number of time steps between times when results are printed out

NHB = number of segments at which water surface elevations are to be given as a boundary condition (1≤NHB≤5)

NQB = number of inflow hydrographs to be given as a boundary condition (0≤NQB≤5)

NWB = number of locations where the width of the flow area between adjacent segments is to vary as a boundary condition, with time. This permits control structures to be opened or closed and for closures to be made by filling across the opening between adjacent segments. (0≤NWB≤6)

NWEIR = number of locations where there is a submerged, broad-crested weir to be represented across the opening between two segments. This permits submerged highways and closures made by filling upward to the water surface to be represented. (0≤NWEIR≤20)

IUNITS = 0 if the withs and distances on the segment data cards are in feet
= 1 if in thousands of feet (this also applies to boundary condition cards)

NWEB = number of locations where the elevation of the top of the submerged weir is to vary, as a boundary condition, with time. (0≤NWEB≤20)

NBR = number of locations where the opening between two segments is to be modeled as a constriction such as a bridge opening. The width of this constricted opening may be caused to vary with time as a boundary condition. (0≤NRB≤20)

Data Input - Part 3 One or more cards (10F5.0)

(HINIT(I),I=1,NSEG) = initial water surface elevation at each of the segments. These values must be given in the same order as the segment data cards which follow. These values are used to obtain initial estimates of the parameters used by the program to recompute these initial water levels. The initial water levels given on these cards need not be very accurate, but extremely poor initial guesses could lead to slow convergence (or even failure to converge) of the initial water level calculations.

Data Input - Part 4 One card for each segment, I=1,NSEG (5I2,F8.0,F6.0,F5.0)

ISEG(I) = segment number. Any two digit number will suffice. Segments do not have to be numbered in order. This is an external number defined by the user. Internally, the program remembers the order in which the user arranges these segment data cards and sets up an internal indexing system that is completely separate and not dependent on the external numbering system used by the user.

IB(I) = 0 if this segment is not a boundary segment where water level boundary conditions are to be given.
= number (i.e.index) to identify the water level boundary condition associated with this segment. The boundary condition data will be given subsequently in the array HB(K,J) where J=1,NOP and IB(I)=K. If NHB=1, then IB(I)=0 for all segments but one. For that segment IB(I)=1.

IQ(I) = 0 if this segment does not recieve as input an inflow hydrograph.
= number (i.e.index) to identify the inflow hydrograph boundary condition associated with this segment. The boundary condition data will be given subsequently in the array QB(K,J) where J=1,NOP and IQ(I)=K.

NADJ(I) number of segments adjacent to this one.

KOUT(I) = 0 if results for this segment are not to be included among the output
= 1 if the results for this segment are to be printed

SA(I) = surface area of this segment in millions of square feet

EL(I) = elevation of the bottom at the centroid of this segment

FN(I) = value of Manning's "n" for this segment if a different value than given as FN in part 2 is to be used for this segment If the part 2 value of FN applies to this segment, leave FN(I) blank.

Data Input - Part 5 One card for each segment I=1,NSEG (4(I2,F7.0,F8.0,1X))

JSEG(J,I) = external (user specified) number of the J-th segment adjacent to segment ISEG(I).

DIST(J,I) = distance from the centroid of segment ISEG(I) to the centroid of JSEG(J,I)

W(J,I) = width of the opening from ISEG(I) to JSEG(J,I). This may subsequently be changed with time as a boundary condition, if desired.

Data Input - Part 6 NWEIR cards required, one for each weir. If NWEIR = 0, no cards are required (3I5,F5.0)

IWEIR(K) = external number of the K-th weir. Any 5-digit number will suffice.

JWEIR(K,J) = external number of the J-th segment adjacent to the K-th weir, J=1,2.

ELW(K) = elevation of the top of the K-th weir.

Data Input - Part 7 NBR cards required, one for each bridge opening If NBR=0, no cards are required. (3I5,2F5.0)

IBR(I) = external number of the I-th bridge opening.

JBR(I,J) = external number of the J-th segment adjacent to the I-th bridge opening, J=1,2.

PW(I) = constriction ratio defined as ratio of unconstricted flow through bridge location to total unconstricted flow rate.

WOPEN(I) = width of the constricted opening at time t=0. (note W(J,I) for this location is not used). This may subsequently be changed with time as a boundary condition.

Data Input - Part 8 One card required. This card gives the number of data points on subsequent boundary condition data cards, NDP, and the time internal, BDT, in seconds, between these data points. The first point occurs at time t=0. If the total length of the computation (i.e. NDT*DT) should exceed the length of time given by the boundary conditions (i.e. NDP*BDT), the boundary conditions are assumed to apply cyclically. In that case, the value of the boundary condition at the first point is assumed to apply,again, immediately after time NDP*BDT (or any multiple of NDP*BDT) has passed. Thus, if NDT*DT is exactly equal to N*NDP*BDT, the program will completely cycle through the boundary conditions a total of N times. Also, if a boundary condition is a continuous function, the first value and the NDP-th value must be the same. (I5,F5.0)

NDT = number of data points on the boundary condition function.

BDT = time internal, seconds between boundary condition data points.

Data Input - Part 9 NHB sets of cards required (I=1,NHB) (10F5.0)

(HB(I,J),J=1,NDP) = water level boundary conditions at the I-th water level boundary segment. The point for J=1 occurs at t=0. Other points occur at time (J-1)*BDT seconds.

Data Input - Part 10 NQB sets of cards required (I=1,NQB). If NQB=0, no cards are required. (10F5.0)

(QB(I,J),J=1,NDP) = inflow rate, thousands of cfs, at the I-th inflow boundary segment at time (J-1)*BDT seconds. Inflow hydrographs may not be supplied to water level boundary segments.

Data Input - Part 11 NWB sets of cards are required, I=1,NWB. If NWB=0, no cards are required. (2I5,10F5.0/(10F5.0)).

(IWB(I,J),J=1,2) = external numbers of the segments adjacent to the I-th opening which is to vary with time as a boundary condition.

(WB(I,J),J=1,NDP) = width of the opening between segments IWB(I,1) and IWB(I,2) at time (J-1)*BDT seconds. The units of WB(I,J) are the same as the units of W(J,I) in Part 5, and these units depend on the value of IUNITS in Part 2.

Data Input - Part 12 NWEIR sets of card are required, I=1,NWEIR. If NWEIR=0, no cards are required. (10F5.0)

(WEB(I,J),J=1,NDP) = elevation of the J-th weir at time (J-1)*BDT seconds.

APPLICATION STRATEGY

The area to be studied may be a river, an estuary, a lake, etc. The total area to be modeled is represented as a set of segments, of any arbitrary shape, that jointly span the total area. Certain segments must be designated as boundary segments. There must be at least one segment where the water surface elevations, as a function of time are given, although more than one segment may be designated as a water level boundary segment. If there are tributary inflows to be represented, these may be accounted for by designating certain segments as receiving inflow from a given inflow boundary condition.

After the study area has been partitioned into segments, a segment data card must be prepared for each segment. The sequence of these cards is not important, but the segment sequence for Parts 4 and 5 of the input data must be the same.

When laying out the segments, the boundaries between adjacent segments should follow as much as possible the general flow pattern so that velocity vectors generally lie either parallel or perpendicular to the segment boundaries.

There is wide flexibility to the approach one can take in dividing a study area into segments for unsteady flow computation. At one extreme, the maximum number of segments is limited by storage availability for the arrays of data that are generated for each segment. The maximun number of segments is also limited by the large amount of computer time needed to execute jobs with many segments. This time increases in proportion to the square of the number of segments. As the storage and time requirements are permitted to increase, the resolution and accuracy of the model also improve, but there is a limit to the ultimate accuracy. There must be some rational strategy to guide this decision process, but none has been developed so far. Since the segments may take on any shape, certain areas of particular interest may be modeled in more detail than other more remote areas. At present, it remains an art to characterize a study area in terms of a number of segments that account for the essential properties of the study area.

OUTLINE OF THE COMPUTER MODEL

The sequence of operations performed by the model are illustrated in the flow chart given in Figure 3. The main calculations are performed first to determinethe initial steady-state water levels and then to determine the flow conditions at time T from those at T-DT and from the boundary conditions.

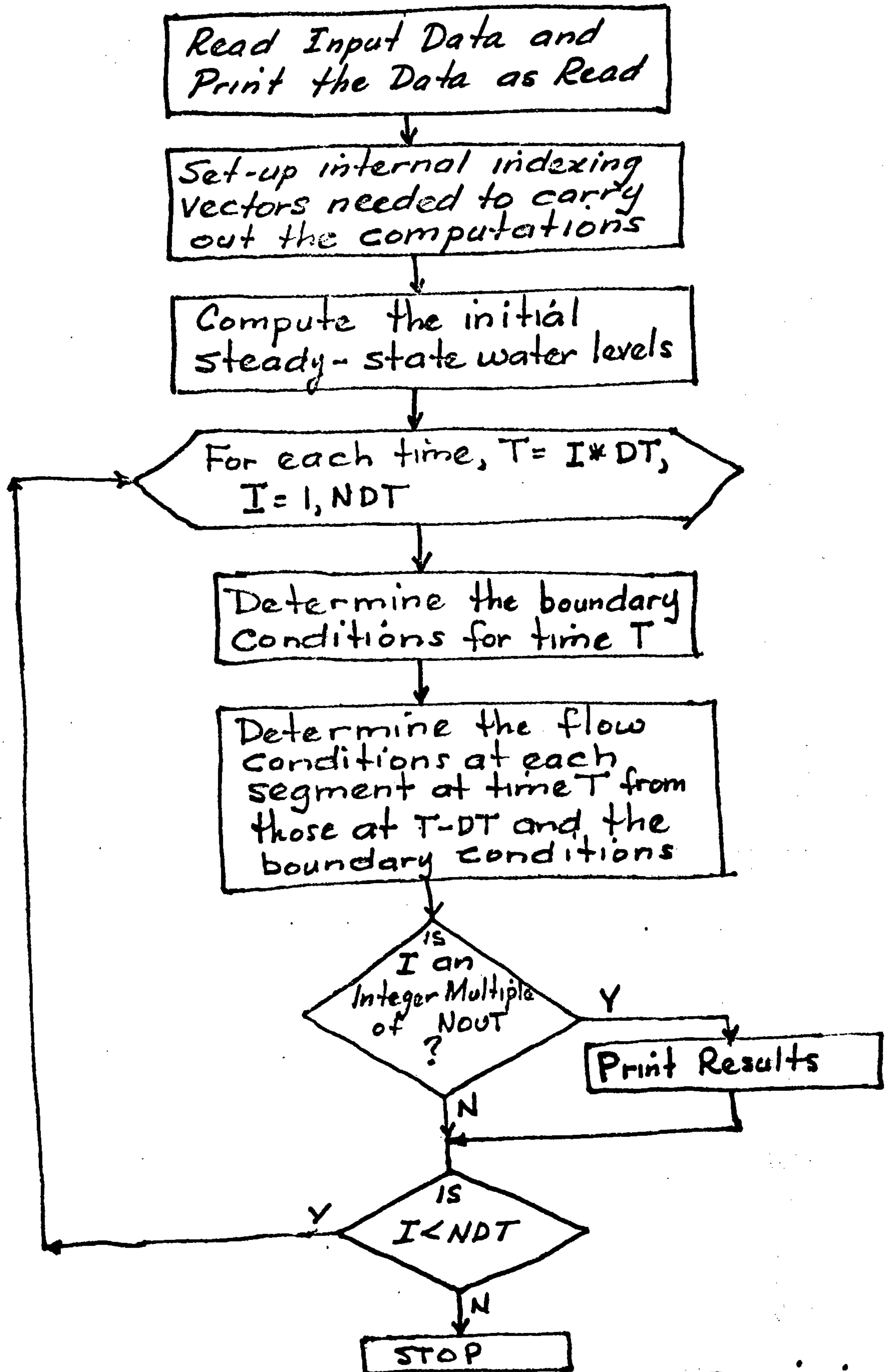


Figure 3 - Flow Chart of Two-Dimensional Flow Model.

APPLICATION OF UNSTEADY FLOW THEORY

Consider a grid of rectangularly shaped elements as shown in Figure 3, and assume that this grid is superimposed on a two-dimensional study area. Flow would be observed to pass from one cell of the grid to another. The mechanics of this flow may be described mathematically in terms of a system of partial differential equations. These equations may be used to devise a set of approximate relations for the unsteady flow through finite elements composing the rectangular grid. Additionally, the approximate relations may be used with somewhat less accuracy for irregularly shaped finite elements.

One of the unsteady flow partial differential equations is a so-called continuity equation. Application of this equation to a finite element in Figure 4 leads to the statement that the instantaneous net rate of inflow to a segment must be balanced by an equivalent instantaneous rate of change of storage in that segment. Assuming now that the segment boundaries are located so that the rate of change of volume in the segment is almost exactly equal to the surface area, SA, times the rate of change in water level dH/dt, the approximate continuity relation for segment i is

$$SA_i \frac{dH_i}{dt} = \text{net instataneous inflow rate to i.} \qquad (1)$$

If segment j is adjacent to i, let Qij be the flow rate from j to i. Then, Equation becomes

$$SA_i \frac{dH_i}{dt} = \sum_{j \in J_i} Q_{ij} \qquad (2)$$

where J_1 is the set of segments adjacent to i.

The remaining unsteady flow partial differential equations are so-called momentum equations because they describe the balance of momentum flux across the boundaries of an elemental control volume. Assuming that certain terms in this equation are of little significance for some practical problems (this must be shown to be true in any use of this model), the momentum equations may be simplified by neglecting the unimportant terms. In particular, omitting the inertial term $\partial v/\partial t$ and the connective velocity term $v\partial v/\partial x$ leads to the simplified equation

$$\frac{\partial H}{\partial x} = S_f \qquad (3)$$

where $\partial H/\partial x$ is the local slope of the water surface and S_f is the local friction term. Assuming the Manning equation applies,

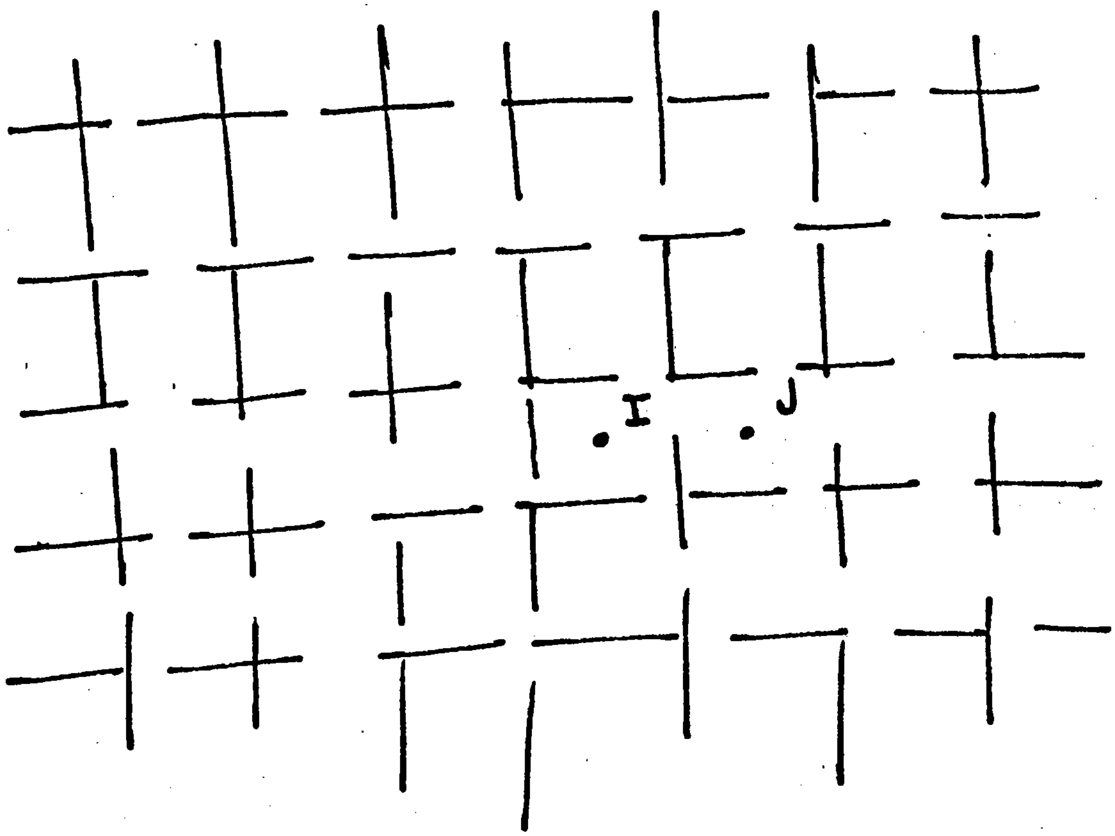


Figure 4 Rectangular Segments

$$S_f = \frac{n^2 V^2}{1.49^2 h^{4/3}} \qquad (4)$$

where V is the local velocity and h is the local depth.

If the segments are laid out so that the slope of the water surface, $\partial H / \partial x$, at the boundary between i and j is equal to $(H_j - H_i)/DIST_{ij}$, the approximate relation

$$\frac{H_j - H_i}{DIST_{ij}} = \frac{n^2 V_{ij}^2}{1.49^2 h_{ij}^{4/3}} \qquad (5)$$

would apply at the boundary between segments i and j.

Let the width of this boundary be Wij and the flow area equal to $h_{ij}W_{ij}$. Then, the rate of flow Qij may be computed by the relation

$$Q_{ij} = k_{ij} (H_j - H_i)^{1/2} \qquad (6)$$

where

$$k_{ij} = \frac{1.49\, h_{ij}^{5/3}}{n\, DIST_{ij}} \qquad (7)$$

Substituting Equation 6 into Equation 2 leads to a system of non-linear equations that completely, but approximately, describe the flow regime. The system is

$$SA_i \frac{dH_i}{dt} = \sum_{j \in J_i} k_{ij} (H_j - H_i)^{1/2} \qquad (8)$$

In summary, the assumptions required to derive equation 8 are
(1) Volume change in segment i are proportional to water level changes at the centroid.

(2) $\partial V/\partial t$ inertial terms are not significant

(3) $V\partial V/\partial x$ connective velocity terms are not significant

(4) Slope of water surface at the segment boundary is equal to the average slope in the water surface between the centroids of the two segments.

Equation 8 does not consider possible tributary inflows to segment i. If these exist, the tributary flow rate must be added to the right hand side of Equation 8.

NUMERICAL SOLUTION OF THE NON-LINEAR DIFFERENTIAL EQUATIONS

Equation 8 defines a system of non-linear differential equations, one equation for each segment, except for water level boundary segments. The numerical procedure for solving these equations is a follows. First, define

$$K_{ij} = \frac{k_{ij}}{(|H_j - H_i|)^{1/2}} \qquad (9)$$

so that Equation 8 may be re-written

$$SA_i \frac{dH_i}{dt} = \sum_{j \in J_i} K_{ij}(H_j - H_i) + Q_{in_i} \qquad (10)$$

which appears to be a linear equation if K_{ij} is regarded as a constant. The tributary inflow term Q_{in_i} also appears now on the right-hand side of Equation 10.

The procedure for solving Equation 10 is to use an iterative procedure in which initial guesses of H_i are used to compute K_{ij} and then Eq 10 is used to get improved solutions. The program begins with the assumption that the system is in a steady state of equilibrium with the initial values of the boundary conditions. In other words dHi/dt=0 everywhere.

The values of H_i @ t=o are computed from the relation

$$A_o H_o = B_o \qquad (11)$$

where A_o is an NA*NA matrix and Ho and Bo are NAx1 vectors. NA is the number of segments excluding water level boundary segments.

The i-th element of Bo is

$$b_i = K_{ij} H_{j_0} + Q_{in_i} \qquad (12)$$

where Hj is the initial water level boundary condition at segment j, a water level boundary segment adjacent to segment i. Since most segment will not be adjacent to water level boundary segments and since most segments will not have tributary inflows, most elements, b_i, will be equal to zero. Only those segments adjacent to water level boundary segments or receiving tributary inflows will have non-zero elements, b_i.

The i-th row of A_o contains in the i-th column the term

$$\sum_{j \in J_i} K_{ij} \qquad (13)$$

and the j-th column, $j \in J_i$, contains the entries

$$-K_{ij} \qquad (14)$$

for all j not denoting water level boundary segments. Since the elements of A_o are functions of H_o, Equation 11 may be written as

$$A_o(H_o^{(k)}) H_o^{(k+1)} = B_o \qquad (15)$$

where the superscript, k, denotes the k-th iteration. The present program carries out 3 iterations of Equation 15 and assumes that the resulting values of H_o are the initial solution to Equation 10

In order to advance the solution with time, the differential dHi/dt is replaced by the term

$$\frac{H_i^{(t+\Delta t)} - H_i^{(t)}}{\Delta t} \qquad (16)$$

and the values of $H_i^{(t+\Delta t)}$ are computed from

$$A^{(t)} H^{(t+\Delta t)} = B^{(t)} \qquad (17)$$

The i-th element of $B^{(t)}$ is

$$b_i^{(t)} = K_{ij} H_j^{(t+\Delta t)} + Q_{in_i}^{(t+\Delta t)} + \frac{SA_i}{\Delta t} H_i^{(t)} \qquad (18)$$

where $H_j^{(t+\Delta t)}$ and $Q_{in_i}^{(t+\Delta t)}$ are boundary conditions applicable to segment i.

The i-th row of $A^{(t)}$ contains in the i-th column the term

$$\sum_{j \in J_i} K_{ij} + \frac{SA_i}{\Delta t} \qquad (19)$$

and the j-th column, $j \in J_i$, contains the entry

$$-K_{ij} \qquad (20)$$

All other elements of the i-th row of $A^{(t)}$ are zero.

Although it would be possible to carry out an iterative procedure for computing very accurate values of $H^{(t+\Delta t)}$, the program does not iterate. It is assumed that the change from $H^{(t)}$ to $H^{(t+\Delta t)}$ is slow enough that the improvement in the accuracy is not worth the cost.

Since the opening between two segments may be constricted by a bridge embankment or by a submerged weir, Equations 7 and 9 do not apply. Special procedures have been programed to compute appropriate values of K_{ij} where bridges and weirs are present.

FLOW THROUGH CONSTRICTED OPENINGS

The following procedure for computing flow through bridge openings is based on the U.S. Bureau of Public Roads report "Hydraulics of Bridge Waterways", Hydraulic Design Series #1, Aug. 1960. Only a special case has been incorporated in this

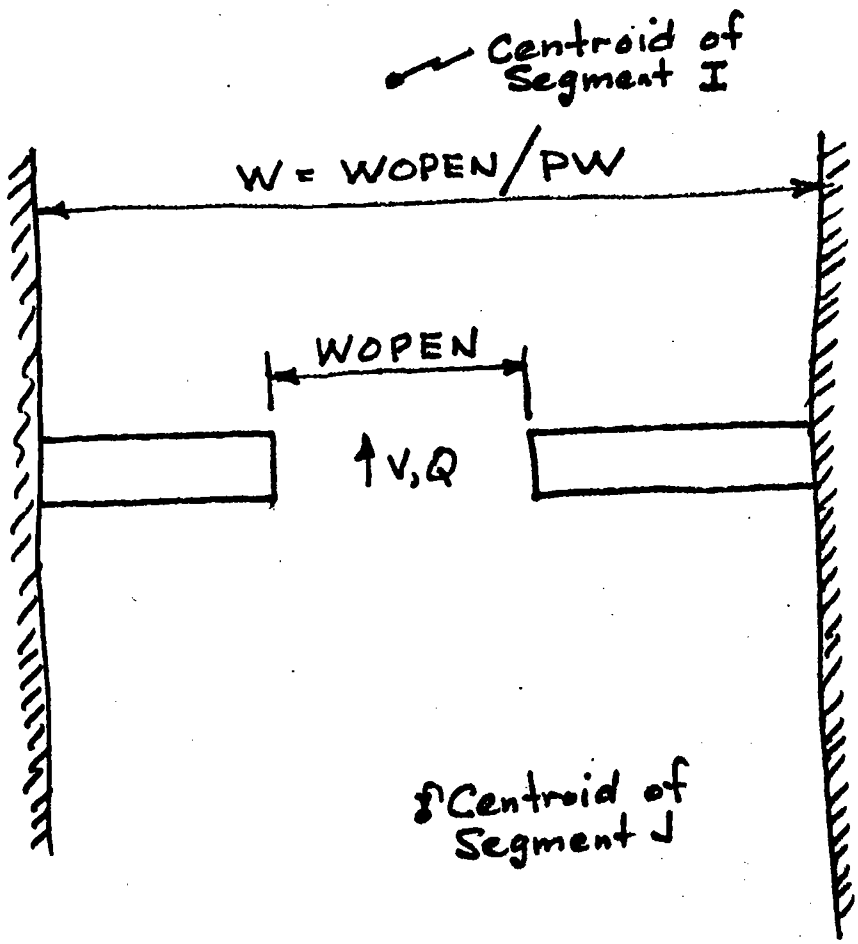


Figure 5 Constricted Opening

program of the complete procedures described in that report.

The head loss through a constricted opening is approximately equal to

$$h_c = k_b \frac{V^2}{2g} \qquad (21)$$

where h_c is the backwater rise above normal stage on the upstream side of the bridge and V is the average velocity in the constriction for flow at normal stage. If the flow rate through the opening is Q and if the area of he opening below normal stage is a, then

$$V = Q/a \qquad (22)$$

The coefficient k_c depends on the degree of constriction. The notation used to determine k_c is illustrated in Figure 5. Let the width of the opening through the constriction be WOPEN and let the unconstricted, normal width of flow be W. Then, the relation for k_b is

$$k_b = 2.25\,(1 - WOPEN/W) \qquad (23)$$

Equations 21 and 22 may be combined to give

$$h_c = k_c \left(\frac{Q^2}{2ga^2}\right) \qquad (24)$$

This accounts only for the head loss through the opening. Since the centroids of the two segments may be a considerable distance apart, the friction loss between the centroids must be included as well. The additional head loss due to friction will be

$$h_f = \frac{DIST}{\left(\frac{1.49}{n}\right)^2 a^2\, AVGY^{4/3}}\, Q^2 \qquad (25)$$

The total head loss, DH, will be

$$DH = h_f + h_c \tag{26}$$

The term Q^2 appears in the expressions for h_f and h_c, and it may be factored out. Equation 26 may be rewritten as

$$DH = (K_b + K_f)\frac{Q^2}{AVGY^2} \tag{27}$$

which may be further written as

$$Q = \frac{AVGY}{\sqrt{DH(K_b + K_f}} DH \tag{28}$$

where

$$K_b = \frac{2.25(1 - W/WOPEN)}{2g\ WOPEN^2} \tag{29}$$

and

$$K_f = \frac{DIST}{AVGY^{4/3}\left(\frac{1.49}{n}\right) W^2} \tag{30}$$

If follows from Equation 28 that the value of K_{ij} to be used in Equation 10 is

$$K_{ij} = \frac{AVGY}{\sqrt{|H_i - H_j|(K_b + K_f)}} \tag{31}$$

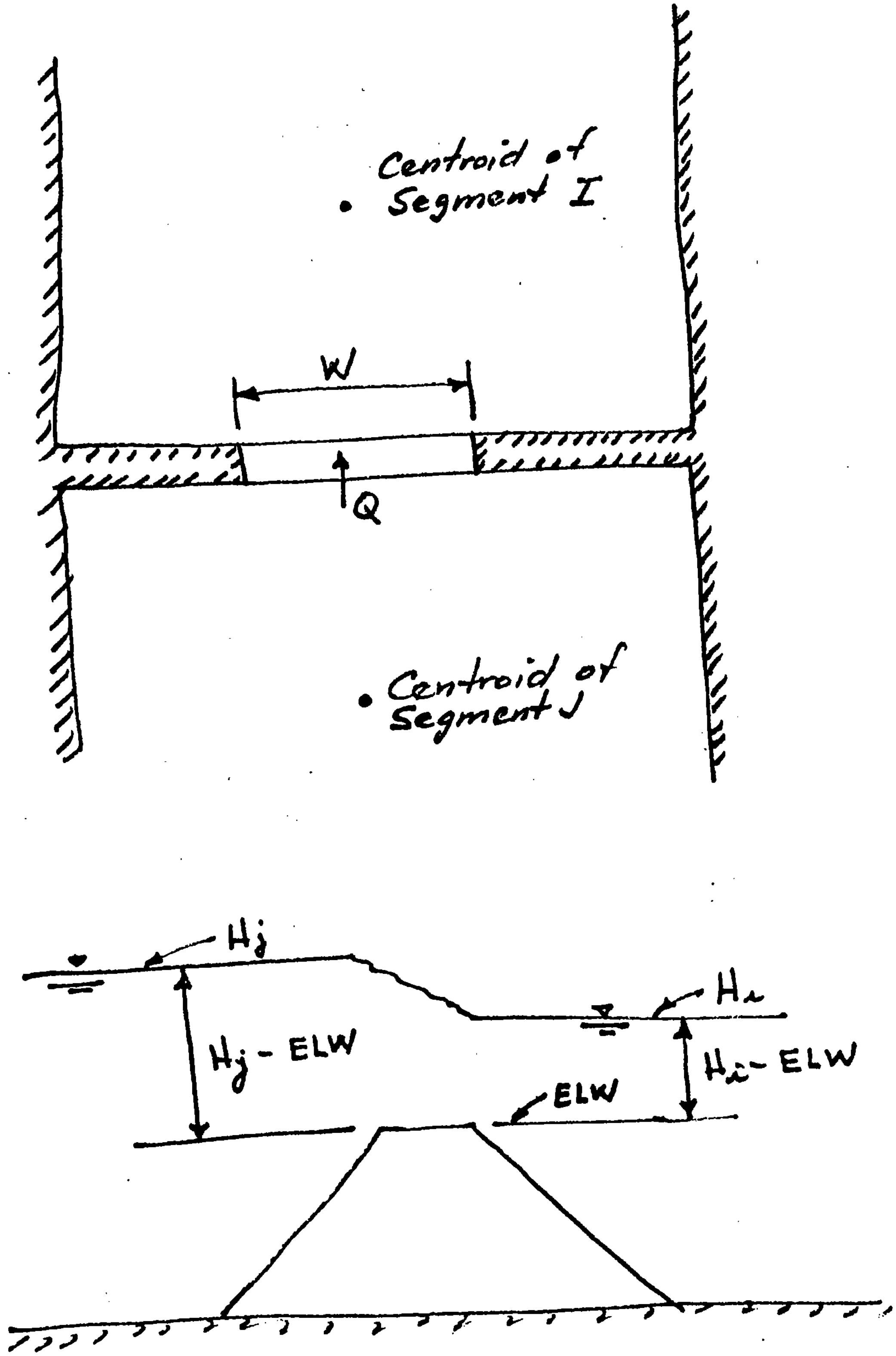


Figure 6 - Flow Over Embankment-Shaped Weirs

FLOW OVER A SUBMERGED EMBANKMENT

If flow over an embankment occurs from segment J to segment I as shown in Figure 6, the equation for the flow rate is

$$Q = (W)(C)(H_j - ELW)^{3/2} \qquad (32)$$

If the flow is not submerged (i.e. if $(H_i-ELW)/(H_j-ELW) \leq 0.83$) the coefficient C is

$$C = 3.09 \qquad (33)$$

However, if $(H_i-ELW)/(H_j-ELW) > 0.83$, the coefficient C is

$$C = 3.09 - 17.5\left[(H_i - ELW)/(H_j - ELW) - 0.83\right]^{2.5} \qquad (34)$$

Equations 33 and 34 were derived from Kindsvater, C.E., "Discharge Characteristics of Embankment - Shaped Weirs", U.S. Geological Water-Supply Paper 1617-A, 1964.

Equation 32 may be rewritten

$$Q = \left[(W)(C)(H_j - ELW)^{1.5}\right](H_j - ELW) \qquad (35)$$

or

$$Q = K_w (H_j - ELW) \qquad (36)$$

In the program, K_w is computed on the basis of H_j at time T-DT; appropriate modifications are made to the A matrix and the term $K_w ELW$ is added to the appropriate elements of the B vector. The friction loss between adjacent segments connected by a weir is neglected.

SECTION III - FOUNDATION INVESTIGATION

1. Previous Investigations

a. General Design Memorandum No. 2, Supplement No. 1, dated March 1970, includes preliminary soils and foundation investigations and studies, as well as a description of the geology of the area. In the area of the project site, six 5-inch diameter undisturbed borings (B-13 thru B-18) were made along the closure alignment. Twenty-seven general type borings were made in the vicinity of the Rigolets Channel. Ten of the 27 borings (C-9 thru C-18) were exploratory in quest of suitable borrow materials. These borings were performed during the period of February 1967 to May 1968.

b. The boring's depth varied from 40 to 150 feet. The associated laboratory tests were made on soil samples obtained from different soil stratification in each boring location. (Refer to GDM No. 2 Plates 3 thru 7 for boring location and on Plates 26 thru 69 for test results.)

2. Field Exploration

a. Fifteen additional soil borings were made from April to August 1971 for the detailed design of this project. Three 5-inch diameter undisturbed borings (X-1U, X-3U and X-5U) and 3 general type borings (X-2, X-4 and X-6), were located approximately along the center line of the Closure Dam. The remaining nine borings were 5-inch diameter undisturbed borings. Seven borings (X-7U thru X-13U) were laid out at about the center line and four corners of the Control Structure Site.

Two borings (X-14U and X-15U) were made at both east and west ends of the closure alignment near the contract limit lines. The boring's depth varied from 65 to 205 feet. Three structure borings (X-7U, X-12U and X-13U) extended to a depth of approximately 170 feet below the channel bottom to El. - 195 feet. The standard split spoon sampler was used in the sand stratum in all borings. The locations of borings are shown on Plate III-1. The logs of borings are shown on Plates A-1 thru A-17.

b. In addition, a field pumping test was performed at the proposed project site in April 1969. The location of the Test Well (2-W) is shown on Plate III-1. An 8-inch I.D. Test Well was extended down to elevation -64.4 feet. The Pumping Test data and results are presented in the Appendix.

3. Laboratory Tests

a. Visual classification and water content determinations were made on all undisturbed soil samples obtained from the borings.

b. Unconfined Compression (UC), unconsolidated-undrained (Q), consolidated-undrained (R), and consolidated-drained (S) shear test and consolidation Tests (C) were performed on samples of representative soils from the undisturbed borings.

c. Liquid and plastic limits were determined for all undisturbed cohesive soil samples tested.

d. Grain size gradations were made on cohesionless soil samples obtained from the borings.

e. The locations of the soil samples tested and the test data are shown on the Soil Boring Drawings, Plates A-1 thru A-17 the consolidation test data which are presented in Plates A-18 thru A-29, and the shear tests which are presented in Fig. A-1 to A-118.

4. Foundation Conditions

a. The project site comprises the low land and water areas in the Rigolets Channel as shown on Plate III-1. The project consists of three major components: the Tie-in Levee, the Control Structure, and the Closure Dam. The Control Structure is located in the deepest portion of the existing water channel. Closure of the complete Channel is effected by the Tie-in Levee on the West and the Closure Dam on the East.

b. The subsurface conditions as encountered along the axis of the proposed structures is presented on the Generalized Soil Profile presented in Plate III-2. Each stratum is identified and the physical properties also presented in Plate III-2. The conditions for each of the project components are discussed separately.

c. Tie-in Levee

(1) The Tie-in Levee is located between Stations 22 + 21 and 32 + 04. The subsurface soil stratification along the Levee axis is indicated on the Generalized Soil Profile, Plate III-2. Cross-sections developed at Station 26 + 64 and Station 30 + 14 are presented in Plates III-3 and III-4 respectively.

(2) The existing ground surface slopes gently in an eastward direction from El. 0 to El. -8 at approximately Station 30 + 00 and then more steeply down to El. -30 at the Control Structure.

(3) In the shallow areas the stratification consists generally of alternating layers of clay, silt and sand. Very soft clays, Strata II, IV and VII and silty sand Strata II and V are encountered down to the top of the Pleistocene Prairie formation. The Pleistocene formation consists of a stiff to hard overconsolidated clay (CH) having increasing shear strength with greater depth. This formation is generally encountered in this area at El. -75.

(4) In the deep water areas the stratification is similar with the exception that Strata II, III and IV are not encountered.

d. Control Structure

(1) The Control Structure is located between Stations 32 + 04 and Stations 43 + 04. The subsurface soil stratification along the axis of the Control Structure is indicated on the Generalized Soil Profile, Plate III-2. A typical cross-section indicating the soil stratification is presented on Plate III-23.

(2) The Control Structure is located in the reach of the Rigolets which contains the deepest channel. The existing channel bottom varies from El. -22 to El. -33.

(3) Between the East and West banks, the Rigolets Channel has eroded and redeposited channel fill consisting of fine sand, silt, or clays. In this portion of the Rigolets, the channel fill primarily consists of a 40 foot thick deposit of fine sand (SP), F, Stratum V. Contained within this sand stratum is an extensive pocket of soft, fat

clays (CH). Stratum VI. This pocket extends from approximately Station 40 + 25 in the western portion of the Control Structure to approximately Station 56 + 45 beneath the Closure Dam and varies up to 17 feet in thickness. A second more recent deposit of very soft, fat clay (CH), Stratum VI is encountered as the surface deposit in the deepest portion of the channel. This stratum varies up to 8 feet in thickness. Below the bottom of the sand, Stratum V, El. -60, the soil stratification is similar to that encountered beneath the Tie-in Levee.

e. Closure Dam

(1) The Closure Dam is located between Station 43 + 04 and Station 82 + 27, the limit of contract. The sub-surface soil stratification along the axis of the Closure Dam is indicated on the Generalized Soil Profile, Plate III-2. Cross-sections developed at Station 47 + 84, Station 61 + 74, Station 73 + 44, Station 77 + 54, and Station 80 + 54 are presented on Plates III-5, III-6, III-7, III-8 and III-9 respectively.

(2) The existing channel bottom is at about El. -25 from the junction with Control Structure at Station 43 + 04 to approximately Station 67 + 50; thereafter, the channel bottom rises gradually to El. +3 at Station 80 + 50.

(3) Throughout the extent of the Closure Dam, the soil stratification below El. -60 is similar to that encountered for both the Tie-in Levee and Control Structure. The conditions from existing ground surface or channel bottom down to El. -60 vary along the axis of the structure in the following manner:

(a) From the Control Structure to Station 56 + 45, a 17 foot thick layer of fat clay (CH), Stratum VI, is sandwiched within the fine sand (SP), F, Stratum V, as discussed in paragraph 4d (3), foundation conditions for the Control Structure.

(b) From Station 56 + 45 to Station 72 + 75 the subsurface stratification consists of the fine sand (SP) F, Stratum V, underlain by Strata VII and the Pleistocene.

(c) From Station 72 + 75 to the project limits the subsurface conditions are similar to that encountered beneath the Tie-in Levee, with the exception that the existing ground surface at the extreme end of the project is at about El. +3 rather than El. 0. The stratification in this area consists of very soft clay with organics (CHO), Stratum I, approximately 6 feet thick which overlies the soft clay-sand-clay layers as for the Tie-in Levee.

5. <u>Foundation Design - Tie-in Levee</u>

a. General. The Tie-in Levee provides the continuity of closure of the Rigolets from the juncture with the existing Connecting Levee (completed by others) to the western abutment of the Control Structure. The levee cross-section provides for a 20 foot wide top. The top of the levee joins the Connecting Levee at El. +9 and ascends in grade to El. +14 at the Control Structure. The levee is constructed using hydraulic sand fill with side slopes provided with slope protection. The Tie-in Levee cross-section and the foundation treatment are related to the problems of foundation stability and construction procedure.

b. Stability Analysis

(1) Shear Strength Parameters used in the analysis are as indicated on Plate III-2, Generalized Soil Profile. The soil properties were established by analysis of all pertinent field and laboratory test data foe each stratum.

(2) Method of Analysis. The stability analysis was performed using the Method of Planes. The failure surface is assumed to consist of an inclined plane in the active zone, a horizontal plane along the base of the failure mass, and an inclined plane in the passive zone. Successive trial wedges and failure surfaces are assumed and analyzed until the minimum factor of safety for a given loading condition is established. This method of analysis is presented in detail on Plates III-10 and III-11.

(3) Design Conditions

(a) End of Construction (Q Case)

Maximum water level, El. + 12.8 feet -- Gulf side.

Minimum water level El. -3.0 feet -- Lake side.

FS = 1.3 (minimum)

Soil Shear Strength - Q Case

(b) Long Term (S Case)

Water Level Conditions as for End of Construction.

F.S. = 1.3 (Minimum)

Soil Shear Strength - S Case

(c) Earthquake condition

Seismic coefficient = 0.05 for computing horizontal force due to earthquake.

Due to the remote possibility of an earthquake and hurricane occuring at the same time, normal water levels are assumed on both the Gulf and the Lake side at El. + 1.5

FS = 1.0 (minimum)

(4) Results of Analysis

(a) Stability analyses were performed at Station 26 + 64 and at Station 30 + 14. The results of these analysis are presented on Plate III-3 and III-4 respectively.

(b) Consideration was given initally to constructing the levee directly on top of the existing soft materials. This configuration resulted in a factor of safety less than unity.

(c) The stability was improved by the method of removing the unsuitable materials and replacing with hydraulic sand fill. Analyses were performed in sufficient number and scope to determine the necessary depth of excavation of the unsuitable material and the lateral width of the excavation which would result in a stable configuration.

(d) The recommended cross-section consists of a levee with a 20 foot wide top, side slope of 4 horizontal and 1 vertical and removal of all unsuitables down to the top of Stratum V (Approximate El. -20).

(e) At Station 26 + 64 the bottom width of this excavation should be a minimum of 40 feet to either side of the center line and the side slopes in the excavation would be 3 horizontal and 1 vertical.

(f) The factors of safety at Station 26 + 64 are 1.35 for the Q Case, and 1.60 for the S Case. The factor of safety for the earthquake loading is 1.40 and 1.64 for the Q Case and S Case, respectively.

(g) At Station 30 + 14 the cross-section is as indicated previously for Station 26 + 64 with the exception that the bottom width of the excavation should extend to a minimum of 60 feet on either side of the center line of the levee. The factors of safety at this section for the end of construction Q Case is 1.30 and for the S Case is 1.49. With earthquake loading the factor of safety is 1.44 and 1.64 for the Q Case and S Case, respectively.

c. Settlement.

(1) The problem of settlement for the Tie-in Levee is substantially eliminated by the removal of the high compressible unsuitable soils Strata II and IV.

(2) Some settlement will be induced by consolidations of Stratum VII and the deep lying Pleistocene clay stratum as a result of the superimposed load.

(3) The magnitude of settlement which can be anticipated should be less than 6 inches at the abutment and in the range of several inches in the shallow inshore areas. The settlement will occur over a prolonged period of time.

(4) Effects of stttlement can be compensated for by the regrading of the top surface of the Tie-in Levee during normal maintenance operations.

d. Site Preparation.

(1) The recommended cross-section necessitates the removal of unsuitable soft materials to approximately El. -20, the top of Stratum V. This operation is best accomplished by the use of a hydraulic dredging procedure and should be conducted during the initial stages of work in the area.

(2) The dredging operation would remove all unsuitable materials within the limits designated as areas II on Plate III-12. This material will be pumped to designated spoil areas.

(3) The excavation will then be filled by underwater placement of hydraulic sand fill pumped from the designated borrow areas. The filling operation would continue until the excavation has been filled.

(4) The subsequent procedures for final construction of the Tie-in Levee will be coordinated with the construction sequence for the Control Structure and are discussed under the appropriate paragraphs in Section III 6, Foundation Design-Control Structure.

6. <u>Foundation Design - Control Structure</u>

a. General.

(1) The Control Structure is 1,100 feet long and 50 feet wide at the top. The top is at El. +14.0 and the sill is at El. -30.0. The structure is founded on a reinforced concrete slab 70 feet wide and 8 feet deep with a bottom at El. -38.0. The slab is supported by steel H-piles driven into the underlying Pleistocene Formation.

(2) Approach Channels are provided to either side of the Control Structure and vary in width from 800 feet at the structure sill to a maximum width of 1,590 on the Gulf Side and 1,040 feet on the Lake Side. The channel bottom on either side of the Control Structure will drop away from the sill at El. -30 to El. -35 along a 1 on 10 slope, then remain level for 100 feet, thence slope upwards along a 1 on 10 slope to El. -30 and continue at this elevation for approximately 2,900 feet on the Gulf Side and 2,300 feet on the Lake Side, then slope upwards on a 1 on 10 slope to the existing channel bottom. This Approach Channel will be paved with rip-rap ranging from 1 to 4 feet in thickness, as erosion protection.

(3) A two-year construction period will be required for completion of the Control Structure and Approach Channel. A cofferdam and dewatering system will be required to maintain the excavation in a dry condition.

b. Site Preparation

(1) The proposed design will require excavation of the overall area to El. -30 feet, with a deeper excavation at the Control Structure to El. -38 feet. In the center portion of the Control Structure, the foundation conditions encountered at this level consist of Sand, Stratum V and areas of the very low strength, high compressibility soft clays, Strata VI. The soft clays, when exposed, will not be capable of supporting the construction activities and must be over-excavated and replaced with a granular material.

(2) The proposed construction in the eastern abutment area provides for a pile supported abutment enclosed within the earth. The soft clays serving as a foundation material for the embankment will result in differential settlement between the embankment and the relatively rigid pile supported abutment.

(3) The Strata VI deposits will be removed prior to the initiation of the Control Structure construction and replaced with hydraulic sand fill.

(4) The limits of the dredging shall be as shown on Plate III-12 and designated as Area I. The deepest dredging will be to approximately El.-50. The unsuitable soils will be deposited in the spoil areas shown on Plate 16 in GDM No. 2. Any dredged sands which can be readily separated from the soft clays will be deposited in temporary storage areas adjacent to the work site for future use.

(5) Upon completion of the removal of all unsuitable material, the dredged area will be back-filled to El. -30

with hydraulic sand fill obtained from the borrow areas.

c. Cofferdam

(1) A Cofferdam and dewatering system is required to permit construction of the Control Structure and the approached channel all in the dry.

(2) The area to be enclosed is as indicated on Plate III-15 and consists approximately of a rectangle measuring 1,450 feet long by 950 feet wide in the deep water connected to the shore by a 650 feet long by 250 foot wide area. The deep water portion of the Cofferdam consists of 62 circular cells, 63.8 feet in diameter with connecting arcs. The closure to the existing shore line at the Connecting Levee is achieved by the use of "Z" type steel sheet piling. The cells and steel sheet piling will be installed after site preparation has been completed and all unsuitable soft soils in the area have been removed and replaced with hydraulic fill.

(3) The top of Cofferdam has been established as El. +6. The frequency of occurrence of the High Water Level of the Rigolets to a stage of +6 is 7 occurrences per 100 years. With a two year construction period, the possibility of over-topping is approximately 14 per cent, within the acceptable limits of risk for construction.

(4) The design of the cells is controlled by the criteria for bearing capacity and interlock tension. In the deep water portion, the Cofferdam must be extended down to Stratum IX at El. -80

and an interior berm to El. -15 feet must be provided to achieve the minimum allowable factor of safety of 3.0. See Plates III-15 thru III-21.

(5) The inshore cells can be terminated at a shallower depth, El. -65 feet. The final tip elevation for these cells and the "Z" sheet pile bulkhead is also controlled by the cut-off requirements as discussed in paragraph 3.6d. The analysis for the cells and sheet pile are presented on Figures III-1 through III-13 and III-14 through III-17, respectively.

(6) The Saturation line within the cell sand fill is critical to the stability. Should the fill dry out so that the saturation line is reduced to approximately El. -15, the factor of safety for bearing capacity would be inadequate. This condition is avoided by providing a series of 2 inch diameter holes at El. -2 on the outboard face of the cell to maintain the desirable saturation line within the cell. Drainage of the cell is facilitated by the use of 4 inch diameter deep holes located at El. -13 on the inboard side of the cells. (See Plate III-20).

(7) Five of the cells, Cell Nos. 29, 30, 31, 32 and 33, will remain as part of the permanent installation. The remainder of the cells will be removed and the sheet piling reused for the cells making up the core of the Closure Dam, see Plate III-19, IV-1 and IV-2 and the Sequence of Construction presented in Section IV.

d. Construction Dewatering System

(1) The construction dewatering system is designed to provide a dry excavation for all Control Structure and Approach Channel work within the Cofferdam. The system provides for handling seepage through the Cofferdam and beneath the Cofferdam as well as surface runoff due to rainfall. The system contains three major components: Cofferdam cell drainage; deep well pumping; and well points.

(2) Seepage. The estimate of seepage flow through the Cofferdam is based upon the assumption that flow can take place through the interlocks in the sheet pile, and through the 2 inch diameter holes provided in the sheet piling at El. -2 to prevent drying of the fill. The total seepage flow into the construction area by the entire periphery of the Cofferdam is estimated at 1,630 gpm. The calculations for this analysis are presented on Figures III-18 and III-19.

(3) Runoff. The runoff resulting from a storm frequency of once in two years is equal to a peak intensity of 19,600 gpm. A significant proportion of this quantity will flow directly into the porous sand and replenish the ground water drawn down by the dewatering system. In the event of high intensity runoff at times when the ground water table is also high, surface drainage would be provided and sump pumps used to discharge runoff out of the Cofferdam excavation. Calculations for the estimation of runoff are presented in Figures III-20 and IV-21.

(4) Field Pumping Tests. A field pumping test

was conducted to establish the permeability characteristics of the foundation sand stratum. The test well was pumped at two different discharges and draw downs recorded. The results of this pumping test indicate the following properties of Stratum V, the sand layer;

coefficient of horizontal permeability, $K_h = 109 \times 10^{-4}$ cm/sec

specific yield = 4 gals. per minute per foot of drawdown.

The range of grain size for the 10 percent finer by weight, 0.08 to 0.168 millimeters.

(5) Hydraulic Design Conditions. The dewatering system must do the following:

(a) maintain the saturation line within the cells from the external water surface to El. -13 at the inboard face.

(b) Provide for lowering of the water tables within the general excavation area to El. -32, approximately two ft. below the general excavation level of El. -30.

(c) Provide for drawdown to a point 5 ft. below the elevation of construction excavation. At the control structure this would correspond to lowering the water level to El. -43. Along the approach channels somewhat higher ground water table elevations could be tolerated due to lesser depth of excavation required to accommodate the rip rap erosion protection construction.

(d) The design drawdown condition for the dewatering system is presented on Plate III-23.

(6) Cell Drainage System. 4 inch diameter weep holes are provided at El -13 along the inboard face of the circular cells. The water seeping out of the weep holes will be permitted to discharge directly onto the berm. A stone fill drained by a 12 inch diameter porous concrete drain pipe will permit infiltration of the weep water and collection by means of the drain pipe. The water would then be carried by gravity flow to concrete sump pits and then would be pumped out of the excavation area. Details of this system are presented on Plates III-20. The location of the sump pits are indicated on Plate III-22.

(7) Deepwells.

(a) Deepwells are provided as the major component of the dewatering system. A total of 26 wells are located on the berm and are apaced at 150 ft. intervals about the excavation. A 7 inch submersible deepwell turbin pump capable of pumping 450 gpm with a head of 53 ft. is provided in each well. The pumps will be installed immediately after completion of the Cofferdam and berm and after water within the excavation has been lowered to below a minimum El. -20 ft. by the use of large capacity surface pumps.

(b) The deepwell system has been designed to lower the water table betwee n the pumps to El. -32 ft. The well will be gravel packed and provided with a 5 foot stainless steel well screen having a #100 slot. A 5 inch diameter discharge pipeline shall lead the pump water beneath the berm through a protective corrugated pipe sleeve and then up the side of the Cofferdam, across the top of the Cofferdam to discharge. The location of the deepwell on the berm and details of the installation are presented on Plate III-20. The layout of the deepwell pumps is presented on Plate III-22 and the details of the deepwell are presented on Plate III-24. The design calculations are presented in Figures III-22 through III-27 inclusive.

(8) Well Points. The third component of the dewatering system consists of a wellpoint installation. Well points 2½ feet long and 2½ inches in diameter with stainless steel screen of No. 10 slot will be located throughout the excavation area as necessary to lower the water table in the immediate proximity of the construction activity. In general, the well point system will be located parallel to the longitudinal axis of the Control Structure as indicated on Plate III-22. The well points will depress the water table to a point 5 ft. below the base excavation. The estimated drawdown range for the entire system of weep holes, deep wells and well points is presented on Plate III-23. A detail of the well points is presented on Plate III-24. The calculations for the well point design are presented on Figure III-28.

(9) Piezometers. Piezometers are provided throughout the excavation area to permit checking the behavior of the deepwell and well point systems. The piezometer consists of a well point tip and a riser pipe. The locations of the piezometers are

presented on Plate II-22 and details of the piezometer are presented on Plate III-25.

(10) Automatic Water Level Control. Automatic water level control devices are provided to control the operations of the deepwell pumping system. The automatic water level controls are located in a cased hole so that each automatic control can control the operation of four deepwell pumps in the immediate area of the control. The control device permits adjustment of the "start pumping" and "stop pumping" levels. These levels will be adjusted in the field based upon experience in operating the system. Once properly set, the system should provide for the automatic starting and stopping of the deep well pumps to guarantee achieving the design ground water drawdown. The location of the automatic control devices are presented on Plate III-22 and a detail of the device is presented on Plate III-25.

(11) Power Distribution System. A power distribution system is required for operation of the deepwell pumps and automatic water control system as well as to provide area lighting. The system provides for a transformer rated at 500 KVA with 13.2 KV/ 480-277V. A standby diesel generator having capacity of 750 KVA with 480/277 volts is also provided. Area lighting consisting of 1,000 watt mercury lamps is provided on the berm about the periphery of the construction area. The contractor will be required to provide such additional lighting as he deems necessary within the immediate area

of work on the Control Structure. A plan of the power distribution system is presented on Plate III-26.

e. Pile Foundations

(1) The recommended scheme for foundation design of the Control Structure provides for the use of a battered pile system to take the vertical and horizontal loads imposed on the structure. For this system to function, the piles will act in both tension and compression.

(2) The soil stratification indicates the existence of alternating layers of sands and clays. Without the existence of a significant bearing strata, all piling driven in this area will act primarily as friction piles. It is, essential that adequate penetration be achieved to permit development of the necessary pile capacities.

(3) Pile types considered included both displacement and non-displacement piles. Due to the large depths of penetration required to achieve the pile design capacity and the relative stiffness of the layers to be penetrated, non-displacement piles have been selected. These piles will permit easier penetration with lower driving energy requirements while the displacement pile may not be able to achieve the design tip elevations without resorting to preboring or other similar methods of construction.

(4) Steel "H" piles are specified. The potential for corrosion at the site is neglible because of the complete submergence and burial of the piling. No reduction in steel area is

necessary and the full cross section of the "H" pile can be used. Based on the AISI Building Code, the allowable structural strength of steel piles is specified as no more than 35 percent of the minimum yield strength (Fg) not exceeding 36,000 psi. The allowable unit stress of the steel "H" pile for use in design is 12,600 psi.

(5) Soil Design Parameters. The shear strength of the soil assumed for design are as follows:

(a) Channel sand and hydraulic sand fill.

Shear Strength:

$\phi = 30^{o}$, C = 0 psf

Friction Factor;

tan S = 0.25

Coefficient of Earth Pressure

K_r = 0.7

(b) Clay

Cohesion from the "Q" case or adhesion whichever is less, but not to exceed a maximum of 750 psf

(6) Design Parameters.

(a) Friction shall be assumed to exist about the full perimeter of the pile section.

(b) End bearing is assumed negligible.

(c) The factor of safety with respect to shear strength

Compression piles, FS = 1.75

Tension piles, FS = 2.0

(7) Drag Forces. The construction of the high embankment fill at both the east and west abutments of the Control Structure will result in consolidation of the underlying clay Stratum VII. The Pleistocene clay encountered below El. -75 are over-consolidated, and will not consolidate significantly under the imposed load. The consolidation of Stratum VII will cause the overlying sand and embankment fill to apply a drag force on the piles in that area. The drag forces will vary depending upon the height of embankment fill surcharge overlying the piles, with the maximum drag force developed at Bay No. 1 and Bay No. 23 and the minimum drag force for the abutment piles developed at Bays No. 4 and No. 20. The influence of drag forces will be dissipated approximately 150 ft. from the abutment. Within the abutment area the drag forces will range from a maximum of 100 tons per pile to a minimum of 45 tons per pile. The piles will have the ability to support this load in addition to that required for the design loading of the structure. Since the ability of the pile to transfer load to the soil is a function of the perimeter area, it will be necessary to achieve greater depth of penetration in the Pleistocene clays.

(8) Pile Design Capacities. Curves of pile design capacity versus penetration depth have been developed for the different Bay locations and for various pile sizes. The design curves include the effects of drag as well as the factor of safety for tension and compression. The calculations and pile design

curves are presented in Figures III-30 through III-44, inclusive.

(9) Pile Load Tests. Pile load tests will be required to confirm the pile design capacity and the pile lengths. The pile load testing will be performed prior to the driving of production piles. Load tests should be performed both in tension and compression.

f. Erosion. The foundation soils in the approach channels consist of Stratum V, silty sand or the hydraulic sand fill material used to replace the unsuitable clays. The sands are susceptible to scour when the current velocities in the 30 feet deep channel exceeds 3.5 feet per second, see Fig. IV-3. The anticipated current velocities for normal design conditions are in excess of 4.5 feet per second, therefore erosion protection is required for both approach channels.

g. Sheet Pile Cutoff. A sheet pile cutoff is not required beneath the Control Structure. The maximum differential head under which seepage could occur or piping could be initiated is limited to approximately 15 ft. and this only for the extreme hurricane condition. With the base length of the structure and the differential head, the gradient would not cause piping even should the steady flow state be able to be maintained. In addition the erosion protection both upstream and downstream of the Control Structure has been provided with a filter which will prevent migration of the sand through the erosion protection.

h. Settlements. The Control Structure and abutments are supported on pile foundations. Therefore, settlement of these structures will be negligible. The embankment fills at the abutments, however, will settle due to the consolidation of the compressible Stratus VII. This settlement will be in a magnitude of several inches and should progress slowly with time. No special measures are required to compensate for settlement.

i. Structural Backfill. The structure backfill required about the abutment will be placed in the dry prior to removal of the Cofferdam. The backfill will consist of sand fill obtained from the hydraulic borrow operations. The fill will be shaped and erosion protection provided in accordance with the design criteria established for erosion protection presented in Section V.

7. Foundation Design - Closure Dam

a. General. The Closure Dam completes the closure of the Rigolets from the eastern abutment of the Control Structure to the east bank of the Rigolets. The Dam is approximately 4,000 ft. long and provides a 64 ft. wide top at El. +14 and utilizes a center core of circular sheet pile cells with connecting arcs to facilitate construction. The cells are enclosed within a hydraulic sand fill and erosion protection is provided on the outer slopes. The required slopes and cross-section are related to the stability of the foundation soils and the construction procedures as discussed in the following paragraphs.

b. Stability Analysis

(1) The shear strength parameters used in the analysis are as indicated on Plate III-2 Generalized Soil Profile.

The soil properties were established by analysis of all pertinent field and laboratory test data for each Strata.

(2) Method of Analysis. The stability analyses were performed using the Method of Planes as discussed in paragraph 3.5 b (2). This method of analysis is presented in detail on Plates III-10 and III-11.

(3) Design Conditions

(a) End of Construction (Q Case)

Maximum water level El. + 12.8 feet -- Gulf side.

Minimum water level El. -3.0 feet -- Lake side.

FS = 1.3 (minimum)

Soil Shear Strength - Q Case

(b) Long Term (S Case)

Water Level Conditions as for End of Construction.

F.S. = 1.3 (Minimum)

Soil Shear Strength - S Case

(c) Earthquake condition

Seismic coefficient = 0.05 for computing horizontal force due to earthquake.

Due to the remote possibility of an earthquake and hurricane occuring at the same time, normal water levels are assumed on both the Gulf and the Lake side at El. + 1.5

FS = 1.0 (minimum)

(d) A cellular cofferdam is used as a construction device for achieving closure and the placement of the sand. This structure is assumed to provide no assistance in the stability considerations.

(4) Results of Analyses. Stability analyses were performed at five sections along the length of the Closure Dam. The results of the analyses at each section are discussed individually.

(a) Station 47 + 84

The subsurface conditions in this area indicate the existence of a soft clay stratum identified as Stratum VI. Assuming that this stratum is not to be removed, stability analyses were performed for a Closure Dam having a 64 ft. crest width and side slopes of 4 horizontal on one vertical. The preliminary results of this analysis indicated a factor of safety less than 1.0. Flatter slopes were successively investigated until a stable cross section was developed. The stable cross section is as indicated on Plate III-5 and consists of slopes of 4 horizontal on one vertical from El. +14 to El. +5 and a 10 horizontal on one vertical slope below El. +5. The minimum factor of safety is 1.36 for the Q condition and 1.99 for the S condition. For the earthquake condition the factor of safety reduces to 1.14 and 1.72 for the Q and S conditions respectively.

This section is referred to as the Alternate Scheme, since consideration was given to the use of a single cross-section with 4 horizontal on one vertical slopes for the entire length of the Closure Dam, providing that foundation conditions were suitable. Removal of the unsuitable low strength Stratum VI and replacement with hydraulic sand fill would result in foundation conditions similar to that encountered at Station 61 + 74. The results of the analysis at Station 61 + 74 are applicable at this station if the unsuitable clay is removed and replaced with sand, therefore no additional analyses of the standard section were performed at this station.

(b) Station 61 + 74. This is the typical cross-section and foundation condition for the Closure Dam for the major extent of the Rigolets. The Dam has a top width of 64 ft. and side slopes of 4 horizontal on one vertical. The foundation is a fine sand Stratum V. The analyses at this station is presented on Plate III-6. The factors of safety for the Q case is 1.55 and for the S case 1.78. The factor of safety for the earthquake condition is 1.49 and 1.85 for the Q and S cases respectively.

(c) Station 73 + 44. This station occurs on the east bank of the Rigolets where the channel bottom is shallower. At this point Stratum IV soft clay is encountered. The presence of the cellular Coffer Dam in the dam section has a stabilizing effect for the end of construction conditions but the stability of the outer slopes, is controlled by the existence of the clay strata. In order to achieve a stable embankment with the four horizontal on one vertical slope, it is necessary to excavate the unsuitable soft material and replace it with hydraulic fill. The minimum required width of excavation at the interface between Stratum IV and V corresponds to 82 ft. as measured from the center line of the dam. The results of the stability analyses and the excavation limits are presented on Plate III-7. The minimum factors of safety for the Q case is 1.35 and for the S case is 1.58. For the earthquake conditions the factor of safety is 1.22 and 1.58 for the Q and S cases respectively.

(d) Station 77 + 54. Similar to Station 73 + 44, soft clay material .is encountered in increasing thickness as a foundation soil. The bottom level of this stratum is at approximately El. -20 throughout the area. At this station the minimum width of excavation of soft clay required at the interface of Strata IV and V is 82 ft. as measured from the center line of the dam. The results of the stability analysis and the limits of excavation are presented on Plate III-8. The minimum factors of safety for the Q case are 1.45 and for the S case 1.35. For the earthquake conditions the minimum factors of safety are 1.41 and 1.22 for the Q and S cases, respectively.

(e). Station 80 + 54. The conditions encountered at this extreme eastern end of the structure are similar to that discussed in the preceding two stations with the soft compressible low strength clay strata existing to a depth below El. -20.

The unsuitable soft material will be excavated and replaced with hydraulic sand fill. The minimum width of excavation required at the interface between Strata IV and V is 52 ft. as measured from the center line of the dam. The results of the stability analysis and the configuration of the excavation are presented on Plate III-9. The minimum factors of safety for the Q case is 1.43 and for the S case 2.13. For the earthquake condition the minimum factor of safety is 1.24 and 1.99 for the Q and S cases respectively.

c. Settlement

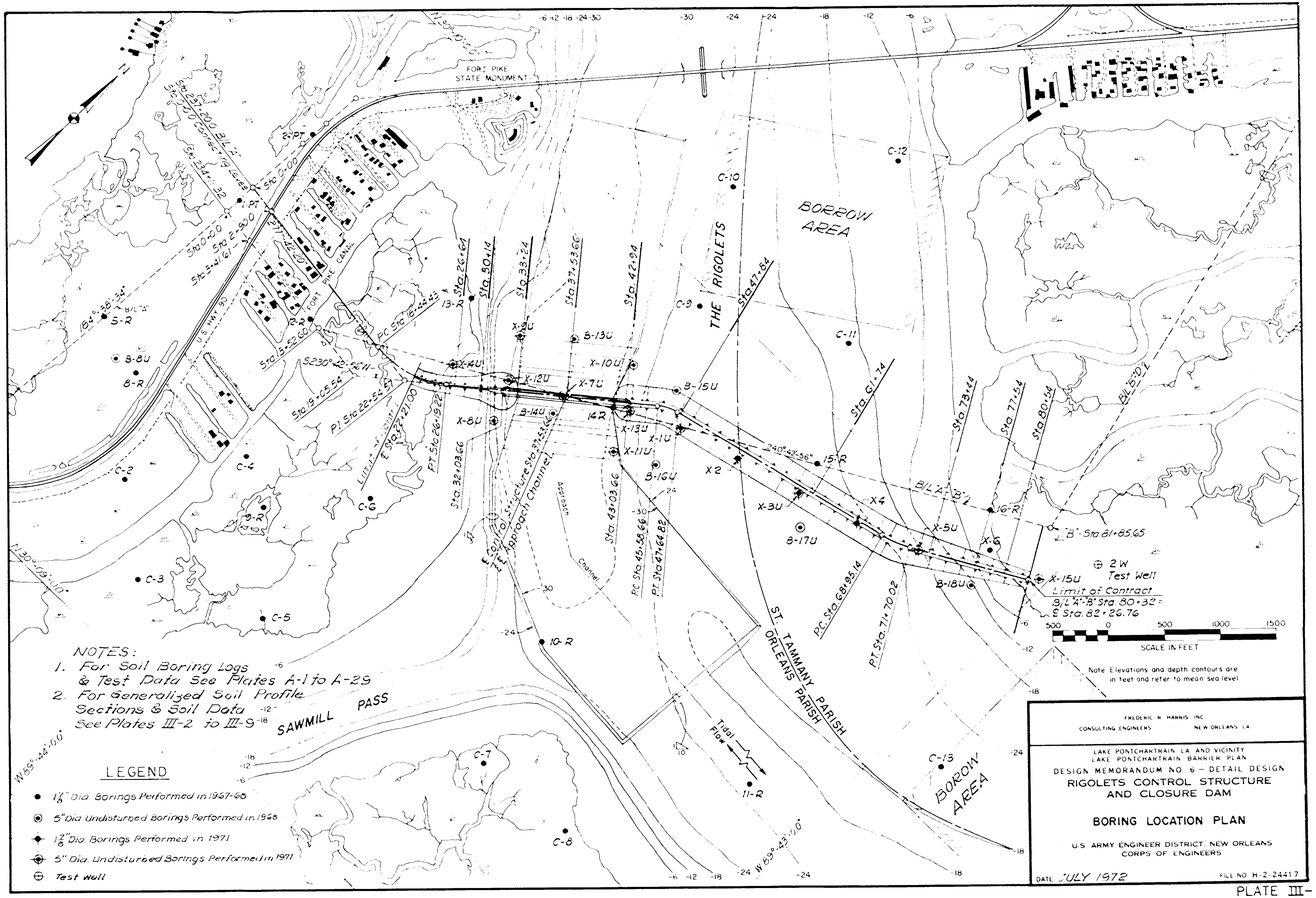
(1) The construction of the Closure Dam to El. +14 will impose a loading on the underlying clay strata such that consolidation of these strata will occur with resulting settlement of the dam. Settlement analyses have been performed for the typical cross-section. The analyses indicate that the major compression would occur in the highly compressible Stratum VI with approximately 49 inches of compression. The total compression for all of the other underlying strata amounts to a total of 10 inches. The calculations are presented in Figures III-45 through III-49.

(2) Compression of Stratum VI in the vicinity of the Control Structure would result in significant differential settlement between the portion of the Closure Dam founded on this compressible soil and that portion of the Closure Dam tied into the west abutment where the soft compressible strata has been removed. Removal of the soft clay stratum and replacement with hydraulic sand

fill will result in a uniform foundation condition beneath both Control Structure and Closure Dam and through the major extent of the Closure Dam.

(3) The time required for consolidation of the underlying clay strata indicates several inches of settlement to occur during the construction period with up to 7 inches taking place in the subsequent 10 year period. Camber, to compensate for the settlement, is not required for this Dam since the crest elevation can be raised in conjunction with the normal maintenance program as the slow process of settlement takes place.

d. Site Preparation. The recommended Closure Dam foundation treatment necessitates the removal of the unsuitable soft materials both overlying and sandwiched in Stratum V. This operation is best accomplished by the use of a hydraulic dredge and should be conducted in conjunction with the other dredging operations required during initial site preparation for the Tie-in Levee and Control Structure. The dredging operation would remove all unsuitable materials within the limits designated as areas II and III on Plate III-12. The material will be pumped to designated spoil areas. The excavation will then be filled by underwater placement of hydraulic sand fill pumped from the designated borrow areas. Filling operations would continue until the excavation has been filled. The subsequent procedures for construction of the Closure Dam are discussed under the appropriate paragraphs in Section IV Closure Dam Design.



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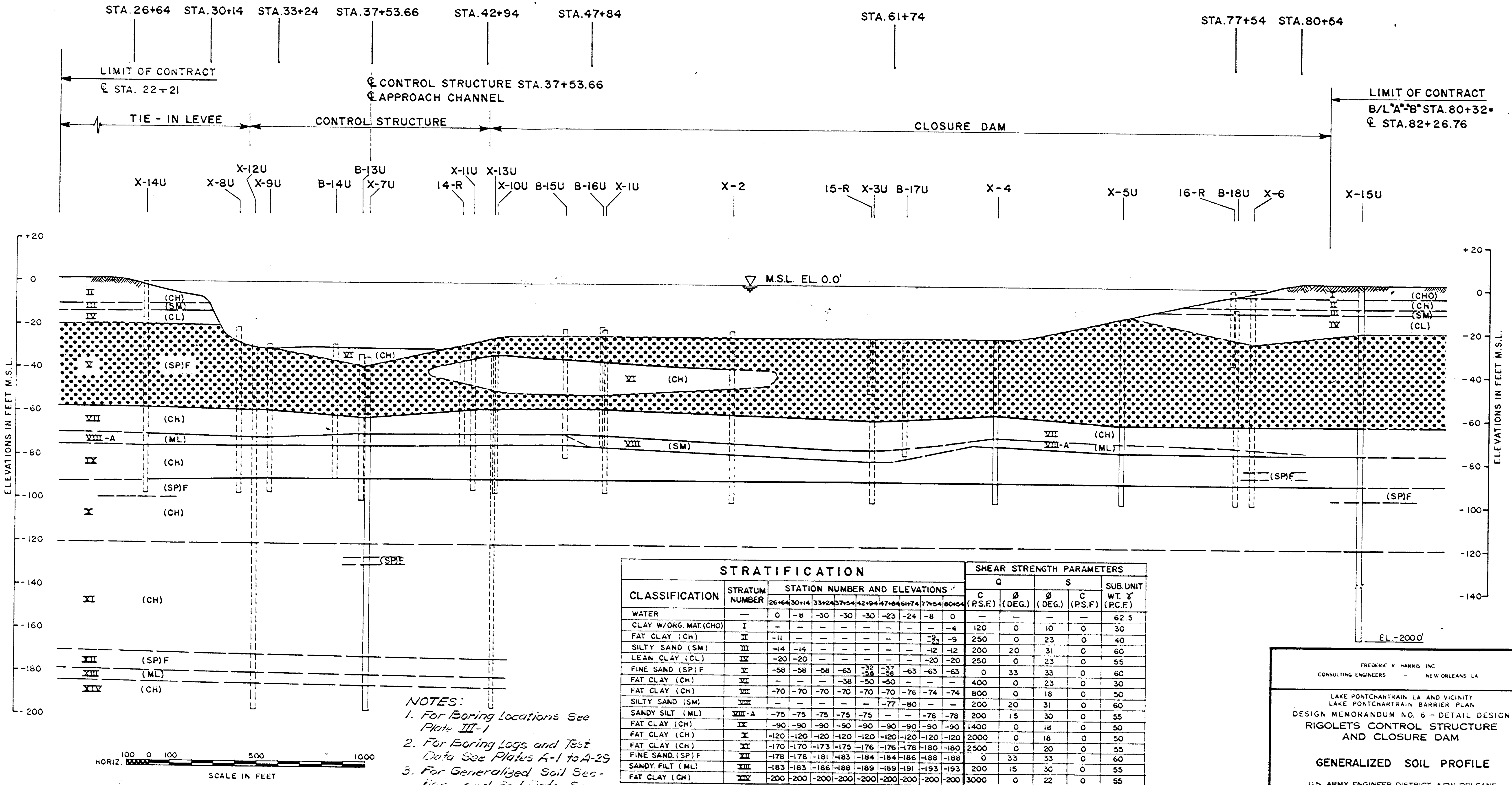
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN

RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

BORING LOCATION PLAN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417



NOTES:

1. *For Boring Locations See Plate III-1*
2. *For Boring Logs and Test Data See Plates A-1 to A-29*
3. *For Generalized Soil Sections and Soil Data See Plates III-3 to III-9*

STRATIFICATION											SHEAR STRENGTH PARAMETERS				
CLASSIFICATION	STRATUM NUMBER	STATION NUMBER AND ELEVATIONS									Q		S		SUB. UNIT WT. γ (P.C.F.)
		26+64	30+14	33+24	37+54	42+94	47+84	61+74	77+54	80+64	C (P.S.F.)	Ø (DEG.)	Ø (DEG.)	C (P.S.F.)	
WATER	—	0	-8	-30	-30	-30	-23	-24	-8	0	—	—	—	—	62.5
CLAY W/ORG. MAT. (CHO)	I	—	—	—	—	—	—	—	—	-4	120	0	10	0	30
FAT CLAY (CH)	II	-11	—	—	—	—	—	—	-9 / -23	-9	250	0	23	0	40
SILTY SAND (SM)	III	-14	-14	—	—	—	—	—	-12	-12	200	20	31	0	60
LEAN CLAY (CL)	IV	-20	-20	—	—	—	—	—	-20	-20	250	0	23	0	55
FINE SAND (SP) F	V	-58	-58	-58	-63	-32 / -58	-37 / -58	-63	-63	-63	0	33	33	0	60
FAT CLAY (CH)	VI	—	—	—	-38	-50	-60	—	—	—	400	0	23	0	30
FAT CLAY (CH)	VII	-70	-70	-70	-70	-70	-70	-76	-74	-74	800	0	18	0	50
SILTY SAND (SM)	VIII	—	—	—	—	—	-77	-80	—	—	200	20	31	0	60
SANDY SILT (ML)	VIII-A	-75	-75	-75	-75	-75	—	—	-78	-78	200	15	30	0	55
FAT CLAY (CH)	IX	-90	-90	-90	-90	-90	-90	-90	-90	-90	1400	0	18	0	50
FAT CLAY (CH)	X	-120	-120	-120	-120	-120	-120	-120	-120	-120	2000	0	18	0	50
FAT CLAY (CH)	XI	-170	-170	-173	-175	-176	-176	-178	-180	-180	2500	0	20	0	55
FINE SAND. (SP) F	XII	-178	-178	-181	-183	-184	-184	-186	-188	-188	0	33	33	0	60
SANDY. FILT (ML)	XIII	-183	-183	-186	-188	-189	-189	-191	-193	-193	200	15	30	0	55
FAT CLAY (CH)	XIV	-200	-200	-200	-200	-200	-200	-200	-200	-200	3000	0	22	0	55

DESIGN PARAMETERS

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

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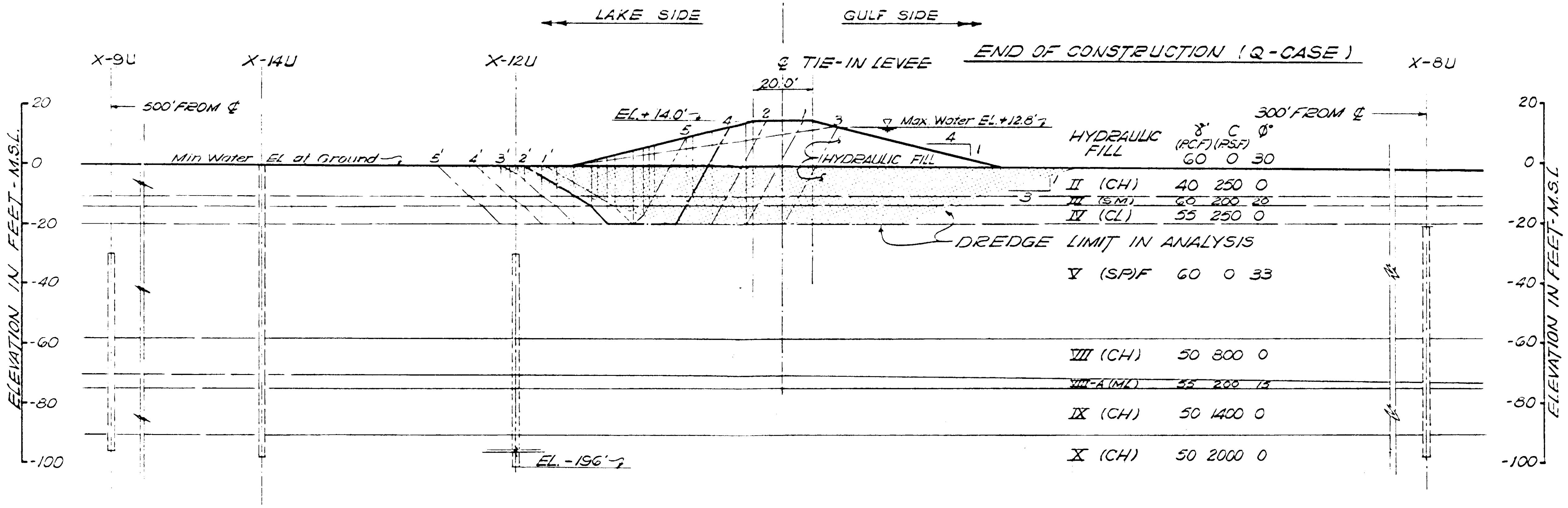
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

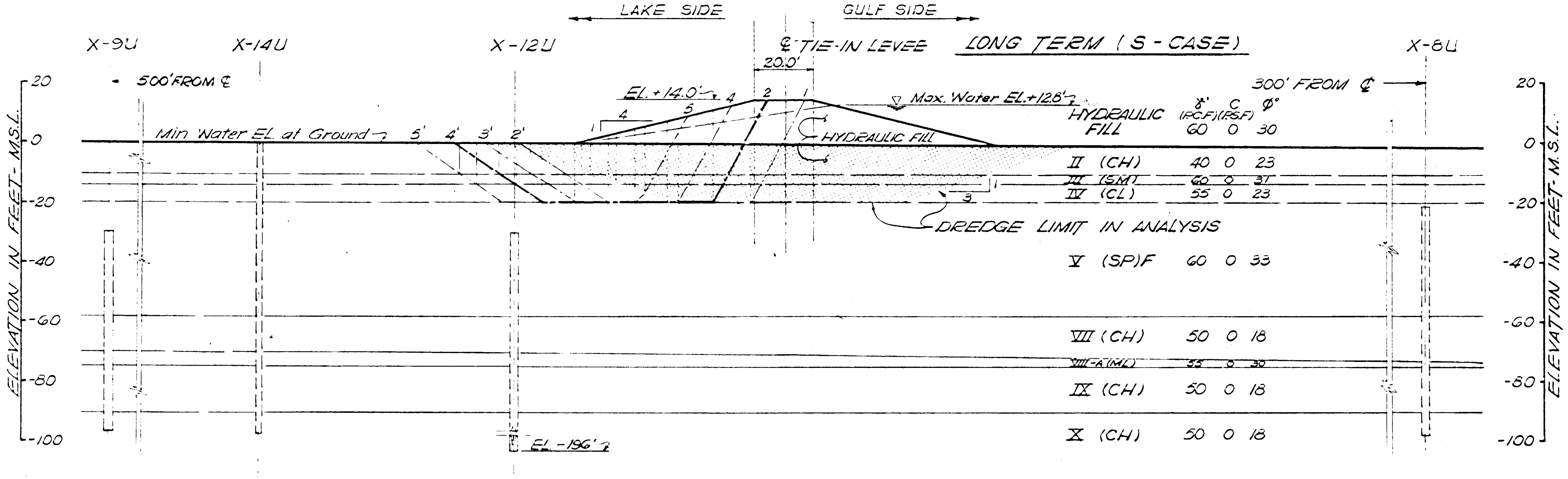
GENERALIZED SOIL PROFILE

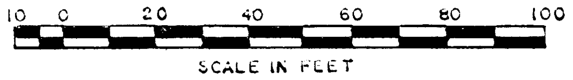
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE *JULY 1972*

FILE NO. H-2-24417







SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-20.0	40876	14631	15600	41845	27250	34750	15992	77992	1.86
12'	-20.0	40876	11450	15600	45026	27250	37000	11793	76044	1.69
13'	-20.0	40876	11370	15600	45106	27250	39750	11542	78943	1.75
14'	-20.0	40876	9950	15600	46526	27250	42500	12671	82422	1.77
15'	-20.0	40876	8014	15600	48462	27250	45750	11167	84168	1.74
21'	-20.0	45414	14631	12862	43645	30276	19700	15992	65968	1.51
22'	-20.0	45414	11450	12862	46825	30276	22000	11793	63670	1.36
23'	-20.0	45414	11370	12862	46906	30276	24700	11542	66519	1.42
24'	-20.0	45414	9950	12862	48326	30276	27700	12671	70647	1.46
25'	-20.0	45414	8014	12862	50262	30276	30953	11167	72393	1.44
31'	-20.0	42781	14631	21086	49236	28521	56000	15992	100513	2.04
32'	-20.0	42781	11450	21086	52417	28521	58500	11793	98815	1.90
33'	-20.0	42781	11370	21086	52497	28521	61250	11542	101296	1.93
34'	-20.0	42781	9950	21086	53917	28521	64000	12671	105192	1.95
35'	-20.0	42781	8014	21086	55853	28521	67250	11167	154438	2.76
41'	-20.0	36234	14631	8611	30214	24156	6800	15992	46948	1.55
42'	-20.0	36234	11450	8611	33395	24156	9300	11793	45250	*1.35
43'	-20.0	36234	11370	8611	33475	24156	12050	11542	47749	1.43
44'	-20.0	36234	9950	8611	34895	24156	14800	12671	51604	1.48
45'	-20.0	36234	8414	8611	36396	24156	18050	11167	53350	1.46
52'	-20.0	25284	11450	7020	20853	15624	2250	11793	29668	1.42
53'	-20.0	25284	11370	7020	20934	15624	5000	11542	32167	1.53
54'	-20.0	25284	9950	7020	22353	15624	7750	12671	36095	1.61
55'	-20.0	25284	8414	7020	23890	15624	11250	11167	38041	1.59

Critical Wedge 42' F.S = 1.35
With Earthquake Loading +DH = .05W

				+DH						
42'	-20.0	36234	11450	7260	32044	24156	9300	11793	45250	1.40

LONG TERM STABILITY

22'	-20.0	45414	11750	12862	46526	30276	28000	17936	76212	1.64
23'	-20.0	45414	10072	12862	48204	30276	33900	14632	78808	1.62
24'	-20.0	45414	7891	12862	50385	30276	39500	11359	81135	*1.60
25'	-20.0	45414	7441	12862	50835	30276	45000	10782	86058	1.70
42'	-20.0	36234	11750	8611	33095	24156	15200	17936	57292	1.74
43'	-20.0	36234	10072	8611	34773	24156	21100	14632	59888	1.72
44'	-20.0	36234	7891	8611	36954	24156	26700	11359	62215	1.69
45'	-20.0	36234	7441	8611	37404	24156	32200	10782	67138	1.80
52'	-20.0	27583	11750	7020	22852	17676	5500	17936	41112	1.81
53'	-20.0	27583	10072	7020	24531	17676	11400	14632	43708	1.79
54'	-20.0	27583	7891	7020	26712	17676	21000	11359	56035	2.10
55'	-20.0	27583	7441	7020	27162	17676	32500	10782	60958	2.24
12'	-20.0	40876	11750	15600	44725	27250	46200	17936	91388	2.05
13'	-20.0	40876	10072	15600	46404	27250	52800	14632	94684	2.03
14'	-20.0	40876	7891	15600	48485	27250	58400	11359	97010	2.00
15'	-20.0	40876	7441	15600	49035	27250	63900	10782	101933	2.09

Critical Wedge 24' F.S. = 1.60
With Earthquake Loading +DH = .05W

				+DH						
24'	-20.0	45414	7891	11900	49423	30276	39500	11359	81135	1.64

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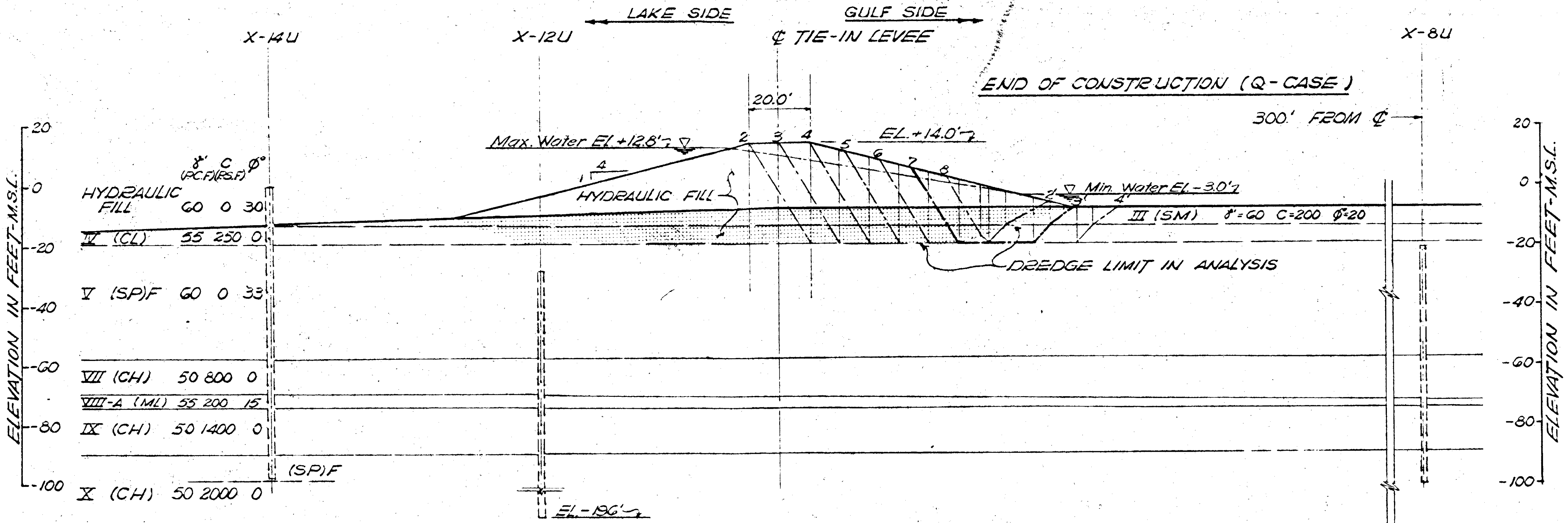
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

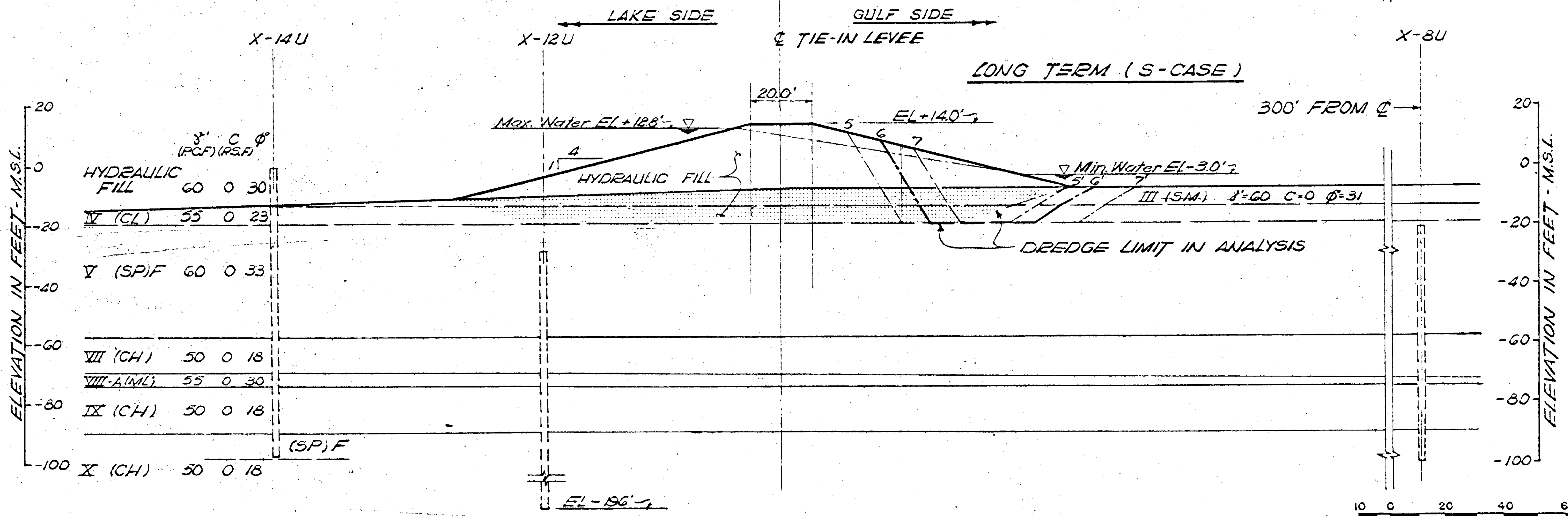
STABILITY ANALYSIS
TIE-IN LEVEE
STA. 26+64

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III-3





SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	+RA	RESISTING +RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
22'	-20.0	42400	8818	20966	54548	28267	55500	9306	93073	1.71
23'	-20.0	42600	5024	20966	58342	28267	59250	7502	95019	1.63
24'	-20.0	42600	4284	20966	59082	28267	63000	7611	98878	1.67
32'	-20.0	45899	8818	17222	54302	30599	42000	9306	81905	1.51
33'	-20.0	45899	5026	17222	58097	30599	45750	7502	83851	1.44
34'	-20.0	45899	4284	17222	58837	30599	49500	7611	87710	1.49
42'	-20.0	38191	8818	13728	43100	25461	30700	9306	65467	1.52
43'	-20.0	38191	5024	13728	46895	25461	34450	7502	67413	1.44
44'	-20.0	38191	4284	13728	47635	25461	38200	7611	71272	1.50
52'	-20.0	29877	8818	12074	33133	19918	19500	9306	48724	1.47
53'	-20.0	29877	5026	12074	36927	19918	23250	7502	50670	1.37
54'	-20.0	29877	4284	12074	37667	19918	27000	7611	54529	1.45
62'	-20.0	24941	8818	8954	25076	16627	10950	9306	36883	1.47
63'	-20.0	24941	5024	8954	28871	16627	14700	7502	38829	1.34
64'	-20.0	24941	4284	8954	29611	16627	18450	7611	42688	1.44
72'	-20.0	19693	8818	6084	16958	13128	2500	9306	24934	1.47
73'	-20.0	19693	5024	6084	20753	13128	6250	7502	26880	*1.30
74'	-20.0	19693	4284	6084	21493	13128	10000	7611	30739	1.43
83'	-20.0	14703	5024	3463	13141	9416	3750	7502	20668	1.57
84'	-20.0	14703	4284	3463	13881	9416	7500	7611	24527	1.77

Critical Wedge 73' F.S = 1.30
With Earthquake Loading, +DH = 0.05W

				+DH						
73'	-20.0	19693	5024	4050	18719	13128	6250	7502	26880	1.44

LONG TERM STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	+RA	RESISTING +RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
65'	-20.0	24941	7975	8954	25920	16627	15700	11771	44098	1.70
66'	-20.0	24941	4920	8954	28975	16627	19400	7254	43281	*1.49
67'	-20.0	24941	4231	8954	29664	16627	24800	6369	47796	1.61
75'	-20.0	19693	7975	6084	17802	13128	7400	11771	32399	1.82
76'	-20.0	19693	4920	6084	20856	13128	11400	7254	31783	1.52
77'	-20.0	19693	4231	6084	21546	13128	16800	6369	36297	1.68
55'	-20.0	29877	7975	12074	33976	19918	25400	11771	57090	1.68
56'	-20.0	29877	4920	12074	37031	19918	29100	7254	56272	1.52
57'	-20.0	29877	4231	12074	37721	19918	34500	6369	60788	1.60

Critical Wedge 66' F.S. = 1.49
With Earthquake Loading +DH = .05W

				+DH						
66'	-20.0	24941	4920	6200	26221	16627	19400	7254	43281	1.64

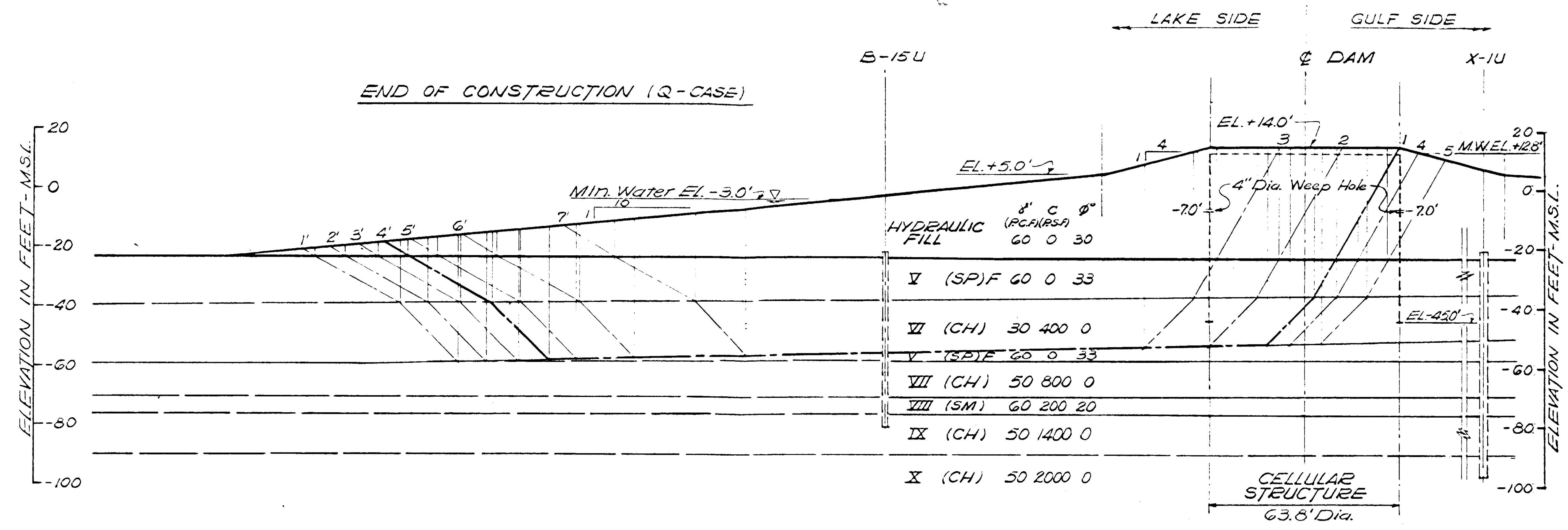
Note: For simplicity of analysis, it was conservatively assumed that the maximum water level on the Lake side is equal to that on the Gulf side, ie. +12.8 feet in lieu of +11.5 feet.

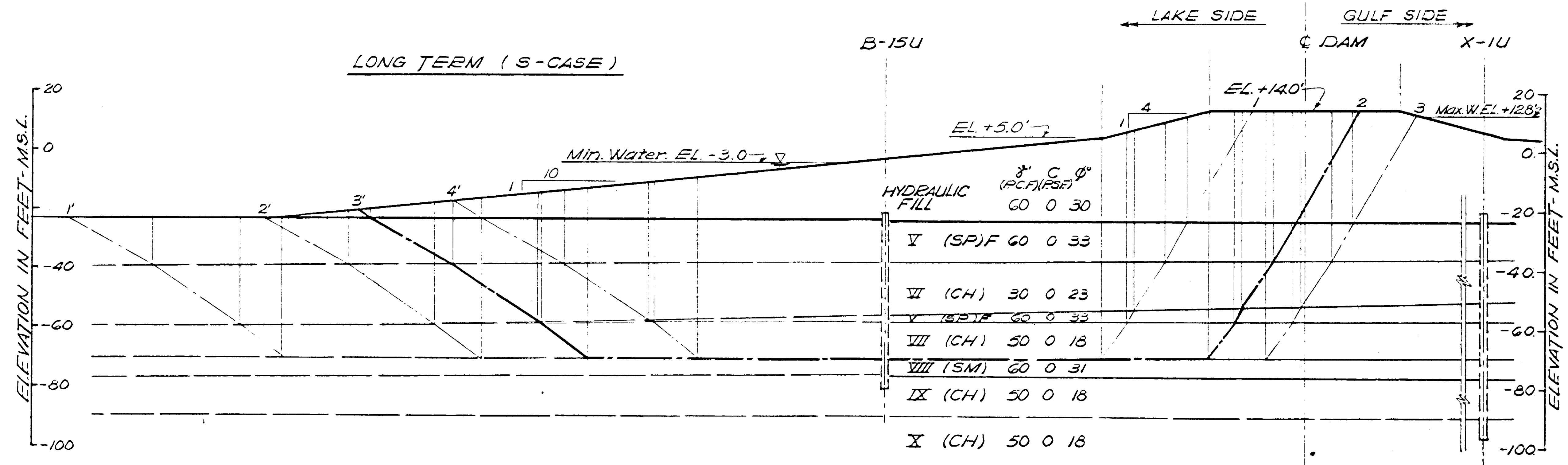
HORIZ SCALE IN FEET: 10 0 20 40 60 80 100

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
STABILITY ANALYSIS
TIE-IN LEVEE
STA. 30+14
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417





NOTE:
Alternate Closure Dam Scheme Assumes no Removal of Unsuitable Material

SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-59.0	146757	43627	59436	162565	73448	109200	42646	225295	1.38
12'	-59.0	146757	43847	59436	162346	73448	105200	43114	221763	1.36
13'	-59.0	146757	46153	59436	160039	73448	101200	45686	220334	1.37
14'	-59.0	146757	46877	59436	159315	73448	97200	47345	217994	*1.36
15'	-58.0	146757	48200	58500	157057	73448	93200	49017	215665	1.37
16'	-58.0	146757	52810	58500	152446	73448	85200	54368	213016	1.39
17'	-58.0	146757	65204	58500	140052	73448	70400	65960	209808	1.49
21'	-59.0	152528	43627	50700	159600	76660	101200	42646	220506	1.38
22'	-59.0	152528	43847	50700	159381	76660	97200	43114	216975	1.36
23'	-59.0	152528	46153	50700	157074	76660	93200	45686	215546	1.37
24'	-59.0	152528	46877	50700	156350	76660	89200	47345	213205	1.36
25'	-58.0	152528	48200	49888	154217	76660	85200	49017	210877	1.36
26'	-58.0	152528	52810	49888	149606	76660	77200	54368	208228	1.39
27'	-58.0	152528	65204	49888	137212	76660	62400	65960	205020	1.49
31'	-59.0	149199	43627	42213	147784	78648	93600	42646	214895	1.45
32'	-59.0	149199	43847	42213	147565	78648	88800	43114	210563	1.42
33'	-59.0	149199	46153	42213	145259	78648	84800	45686	209134	1.43
34'	-59.0	149199	46877	42213	144535	78648	80800	47345	206794	1.43
35'	-58.0	149199	48200	41527	142526	78648	76800	49017	204465	1.43
36'	-58.0	149199	52810	41527	137915	78648	68800	54368	201816	1.46
37'	-58.0	149199	65204	41527	125521	78648	54000	65960	198608	1.58
41'	-59.0	142653	43627	63897	162923	71363	112000	42646	226010	1.38
42'	-59.0	142653	43847	63897	162703	71363	108000	43114	222478	1.36
43'	-59.0	142653	46153	63897	160397	71363	104000	45686	221049	1.37
44'	-59.0	142653	46877	63897	159673	71363	100000	47345	218709	1.36
45'	-58.0	142653	48200	62899	157352	71363	96000	49017	216380	1.37
46'	-58.0	142653	52810	62899	152741	71363	88000	54368	213732	1.39
47'	-58.0	142653	65204	62899	140347	71363	73200	65960	210523	1.50
52'	-59.0	129087	43847	63897	149138	65450	112000	43114	220565	1.47
53'	-59.0	129087	46153	63897	146831	65450	108000	45686	219137	1.49
54'	-59.0	129087	46877	63897	146108	65450	104000	47345	216796	1.48
55'	-58.0	129087	48200	62899	143787	65450	100000	49017	214468	1.49
56'	-58.0	129087	52810	62899	139176	65450	92000	54368	211819	1.52

Critical Wedge 14', F.S = 1.36
With Earthquake Loading, +DH = 0.05W

14'	-59.0	146757	46877	90936 +DH	190816	73448	97200	47345	217994	1.14

LONG TERM STABILITY

11'	-70.0	206803	51033	44928	200697	125463	265522	66806	457792	2.28
12'	-70.0	206803	77512	44928	174219	125463	216976	93395	435835	2.50
13'	-70.0	206803	66568	44928	185163	125463	186555	86913	398933	2.15
14'	-70.0	206803	80283	44928	171448	125463	152233	111632	389329	2.27
21'	-70.0	235195	51033	64646	248808	141504	316822	66806	525132	2.11
22'	-70.0	235195	77512	64646	222329	141504	268275	93395	503175	2.26
23'	-70.0	235195	66568	64646	233274	141504	237855	86913	466273	*1.99
24'	-70.0	235195	80283	64646	219558	141504	203532	111632	456669	2.07
31'	-70.0	217810	51033	74880	241656	131001	347211	66806	545019	2.25
32'	-70.0	217810	77512	74880	215178	131001	298664	93395	523061	2.43
33'	-70.0	217810	66568	74880	226122	131001	268244	86913	486159	2.14
34'	-70.0	217810	80283	74880	212407	131001	233921	111632	476555	2.24

Critical wedge 23', F.S = 1.99
With Earthquake Loading, +DH = 0.05W

23'	-70.0	235195	66568	103120 +DH	271747	141504	237855	86913	466273	1.72

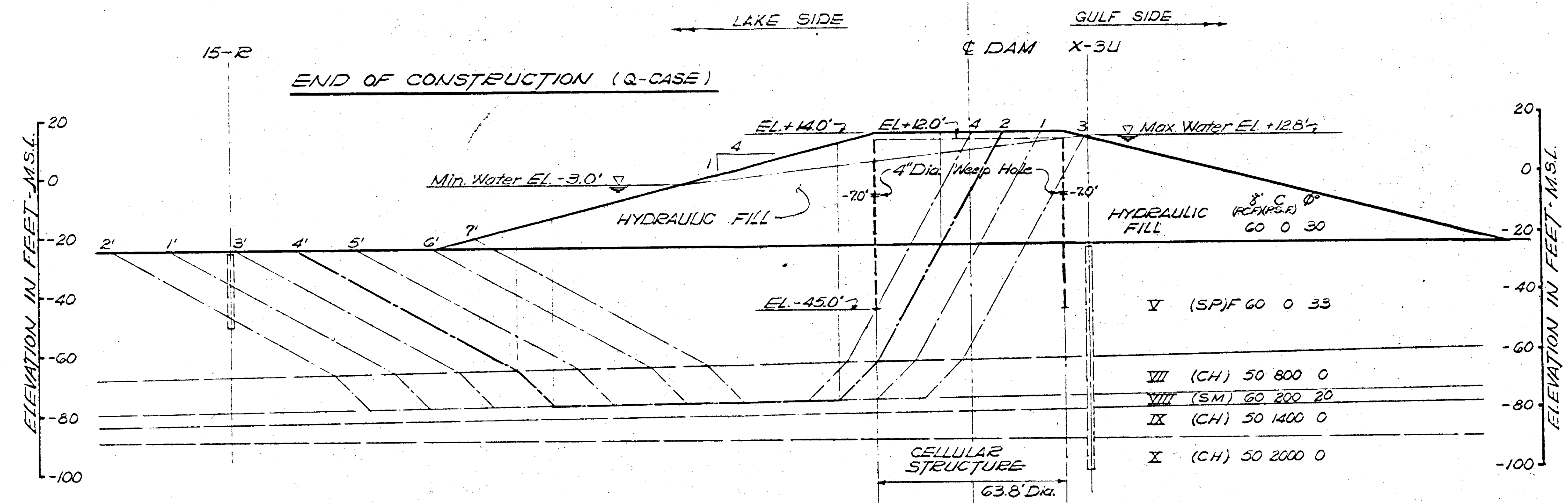
FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA.

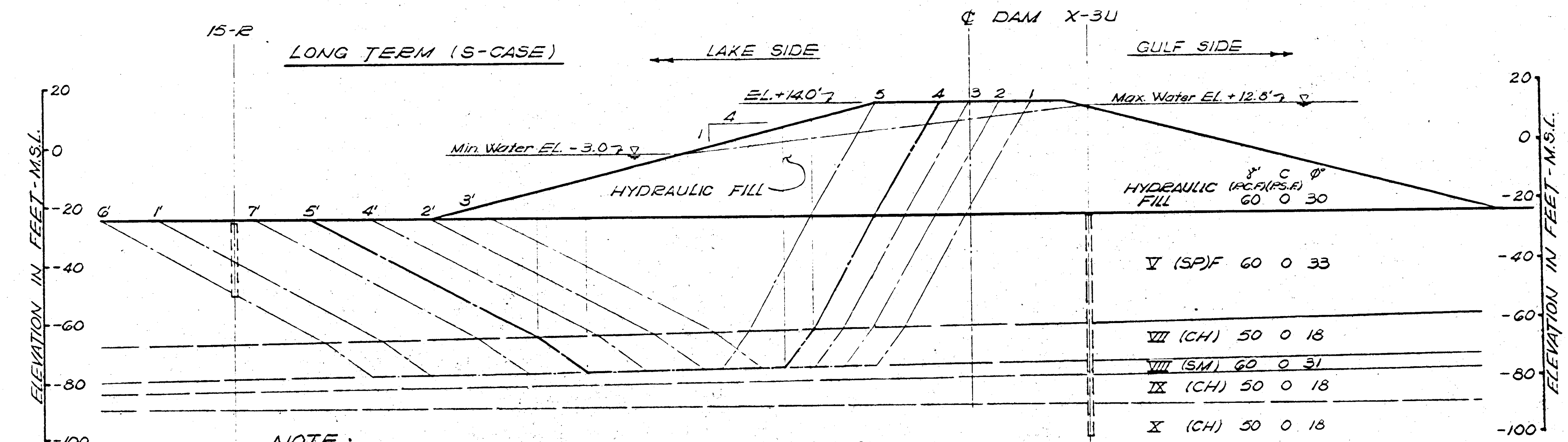
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

STABILITY ANALYSIS
CLOSURE DAM (ALTERNATE SCHEME)
STA. 47+84

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





NOTE: This Section is Typical Where Unsuitable foundation Soils are Removed and Replaced With Sand Fill

HORIZ 10 0 20 40 60 80 100
SCALE IN FEET

SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	FORCES -DP	+DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-78.0	261989	86517	71635	247106	152377	121600	145244	419221	1.69
12'	-78.0	261989	86517	71635	247106	152377	130400	145244	428021	1.73
13'	-78.0	261989	89330	71635	244293	152377	104000	145945	402323	1.64
14'	-78.0	261989	96662	71635	236961	152377	88000	154011	394388	1.66
15'	-77.0	261989	106010	70761	226739	151246	72000	168623	391869	1.72
16'	-77.0	261989	122614	70761	210136	151246	52000	197145	400391	1.90
17'	-76.0	261989	129257	69888	202619	151246	36800	202392	390438	1.92
21'	-78.0	282193	86517	60652	256328	167475	111200	145244	423919	1.65
23'	-78.0	282193	89330	60652	253515	167475	94400	145945	407821	1.60
24'	-78.0	282193	96662	60652	246183	167475	62400	154011	383886	*1.55
25'	-77.0	282193	106010	59904	236086	167475	61600	168623	397698	1.68
26'	-77.0	282193	122614	59904	219482	167475	42400	197145	407020	1.85
27'	-76.0	282193	129257	59155	212090	167475	26400	202392	396267	1.86
31'	-78.0	263708	86517	82867	260058	155129	134400	145244	434773	1.67
33'	-78.0	263708	89330	82867	257244	155129	117600	145945	418674	1.62
34'	-78.0	263708	96662	82867	249913	155129	100800	154011	409940	1.64
35'	-77.0	263708	106010	81868	239566	155129	84800	168623	408552	1.70
36'	-77.0	263708	122614	81868	222963	155129	64800	197145	417074	1.87
37'	-76.0	263708	129257	80870	215321	155129	49600	202392	407121	1.89
43'	-78.0	281296	89330	49920	241885	169573	85600	145945	401119	1.65
44'	-78.0	281296	96662	49920	234553	169573	69600	154011	393184	1.67
45'	-77.0	281296	106010	49296	224581	169573	53600	168623	391796	1.74

Critical Wedge 24' F.S. = 1.55
With Earthquake Loading. +DH = 0.05W = 71664

SLIP SURFACE NO.	ELEV.	+DA	-DP	+DW +DH	TOTAL	+RA	+RB	+RP	TOTAL	R/D
24'	-78.0	282193	96662	71664	257195	167475	62400	154011	383886	1.49

LONG TERM STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	FORCES -DP	+DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-76.0	269571	87202	69888	252256	172265	218513	156891	547669	2.17
12'	-76.0	269571	121471	69888	217987	172265	96228	219476	487970	2.23
13'	-76.0	269571	138644	69888	200814	172265	65978	252201	490445	2.44
14'	-76.0	269571	107070	69888	232389	172265	124896	190412	487574	2.09
15'	-76.0	269571	95486	69888	243973	172265	152991	168447	493703	2.02
16'	-76.0	269571	86483	69888	252976	172265	233374	156248	561887	2.22
17'	-76.0	269571	92052	69888	247407	172265	176947	163643	512856	2.07
22'	-76.0	281193	121471	69888	229609	180788	77773	219476	478038	2.08
24'	-76.0	281193	107070	69888	244010	180788	106441	190412	477643	1.95
25'	-76.0	281037	95486	69888	255439	180685	134535	168447	483668	1.89
27'	-76.0	281037	92052	69888	258873	180685	158491	163643	502820	1.94
21'	-76.0	281037	87202	69888	263722	180685	200057	156891	537634	2.03
34'	-76.0	278765	107070	69888	241583	179712	86967	190412	457092	1.89
35'	-76.0	278765	95486	69888	253167	179712	115061	168447	463221	1.82
37'	-76.0	278765	92052	69888	256601	179712	139018	163643	482374	1.87
44'	-76.0	271937	107070	69888	234755	173871	70144	190412	434429	1.85
45'	-76.0	271937	95486	69888	246339	173871	98239	168447	440557	*1.78
47'	-76.0	271937	92052	69888	249773	173871	122195	163643	459710	1.84
54'	-76.0	233813	107070	69888	196630	150018	34984	190412	375416	1.90
55'	-76.0	233813	95486	69888	208214	150018	63078	168447	381545	1.83
57'	-76.0	233813	92052	69888	211649	150018	87034	163643	400697	1.89

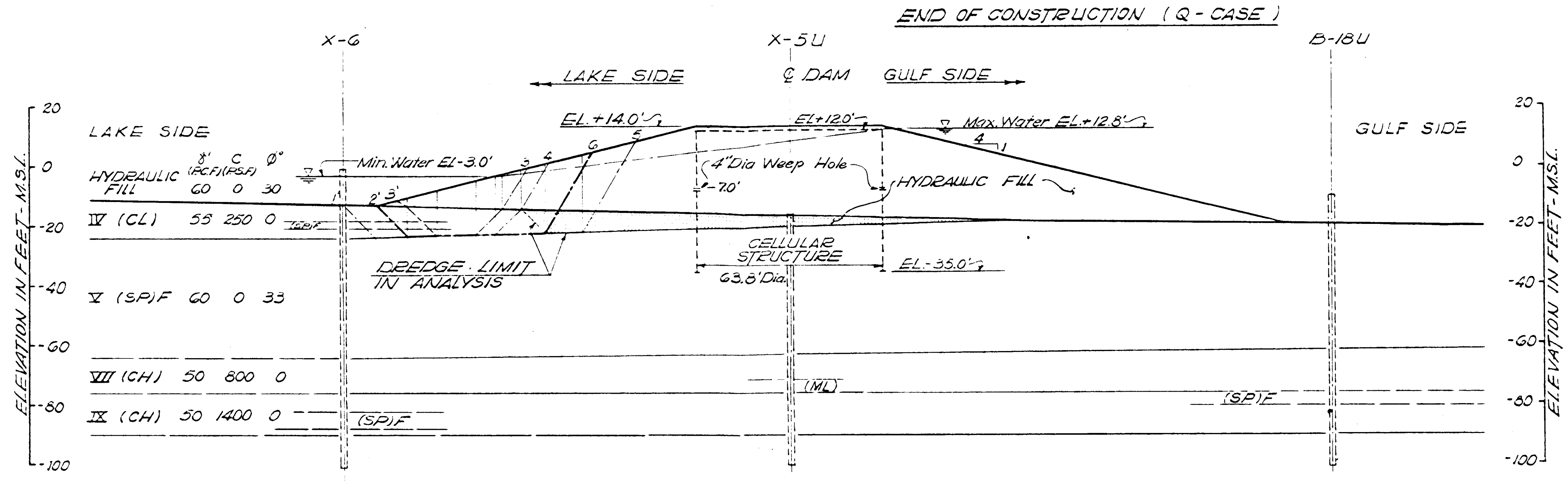
Critical Wedge 45' F.S. = 1.78
With Earthquake Loading. +DH = 0.05W

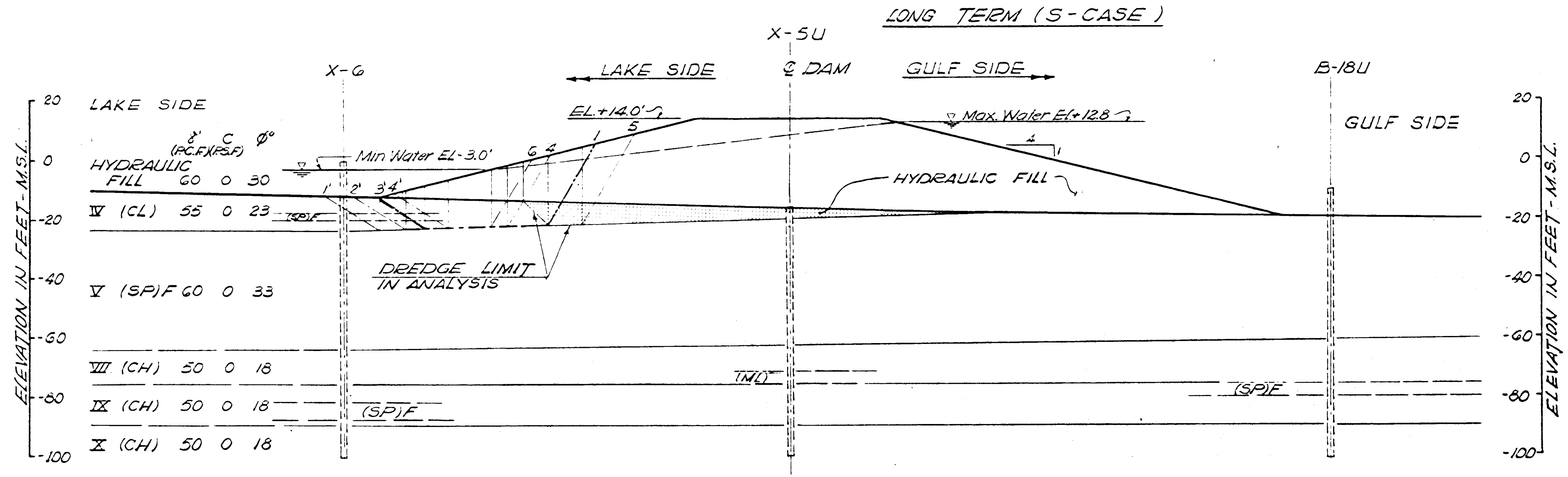
SLIP SURFACE NO.	ELEV.	+DA	-DP	+DW +DH	TOTAL	+RA	+RB	+RP	TOTAL	R/D
45'	-76.0	271937	95486	61329	237780	173871	98239	168447	440557	1.85

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
STABILITY ANALYSIS
CLOSURE DAM (TYPICAL)
STA. 61+74
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417





HORIZ. 10 0 20 40 60 80 100
SCALE IN FEET

SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	FORCES -DP	+DW	TOTAL	RESISTING +RA	FORCES +RB	+RP	TOTAL	SAFETY FACTOR R/D
31'	-24.0	13384	3520	663	10527	8001	8375	5656	22033	2.09
32'	-24.0	13384	3676	663	10371	8001	5750	5303	19054	1.83
33'	-23.0	13384	5481	631	8534	8001	3250	5313	16564	1.94
41'	-24.0	15147	3520	2035	13663	8936	10000	5656	24593	1.79
42'	-23.5	15147	3676	1989	13460	8936	7250	5303	21489	1.59
43'	-23.5	15147	5481	1989	11654	8936	4750	5313	18999	1.63
61'	-24.0	23424	3520	4212	24116	15616	14000	5656	35273	1.46
62'	-23.5	23424	3676	4118	23866	15616	11500	5303	32419	*1.35
63'	-23.0	23424	5481	4024	21967	15616	9000	5313	29929	1.36
51'	-24.0	33116	3520	7332	36928	22077	25430	5656	53163	1.44
52'	-23.5	33116	3676	7176	36616	22077	22930	5303	50310	1.37
53'	-23.0	33116	5481	7020	34654	22077	20430	5313	47820	1.38

Critical Wedge 62' F.S = 1.35
With Earthquake Loading, +DH = 0.05W

				+DH						
62'	-23 5	23424	3676	6800	26548	15616	11500	5303	32420	1.22

LONG TERM STABILITY

SLIP SURFACE NO.	ELEV.	+DA	-DP	+DW	TOTAL	+RA	+RB	+RP	TOTAL	R/D
11'	-24.0	23914	3603	5740	26051	15942	23953	4622	44519	1.70
12'	-24.0	23914	3541	5740	26114	15942	21413	4541	41898	1.60
13'	-24.0	23914	4052	5740	25602	15942	19477	5198	40618	*1.58
14'	-23.0	23914	5216	5491	24188	15942	16591	6765	39300	1.62
41'	-24.0	15366	3603	2745	14508	9430	13486	4622	27539	1.89
42'	-24.0	15366	3541	2745	14571	9430	10945	4541	24918	1.71
43'	-24.0	15366	4052	2745	14059	9430	9010	5198	23638	1.68
44'	-23.0	15366	5216	2620	12770	9430	6123	6765	22320	1.74
51'	-24.0	24543	3603	7332	28271	16362	30817	4622	51802	1.83
52'	-24.0	24543	3541	7332	28334	16362	28277	4541	49181	1.73
53'	-24.0	24543	4052	7332	27822	16362	26341	5198	47901	1.72
54'	-23.0	24543	5216	7020	26346	16362	23455	6765	46583	1.76
61'	-24.0	13011	3603	1341	10749	7896	10544	4622	23063	2.14
62'	-24.0	13011	3541	1341	10812	7896	8004	4541	20442	1.89
63'	-24.0	13011	4052	1341	10300	7896	6068	5198	19163	1.86
64'	-23.0	13011	5216	1279	9075	7896	3182	6765	17844	1.96

Critical Wedge 13' F.S = 1.58
With Eathquake Loading, +DH = 0.05W

				+DH						
13'	-24.0	23914	4052	5729	25591	15942	19477	5198	40618	1.58

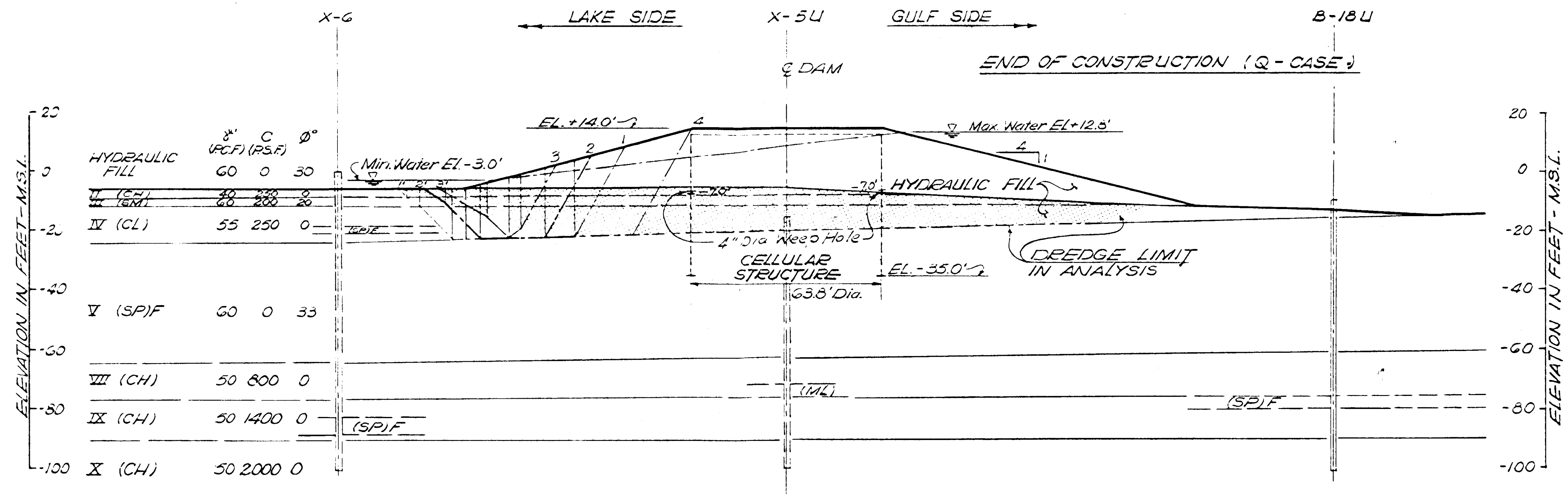
FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA.

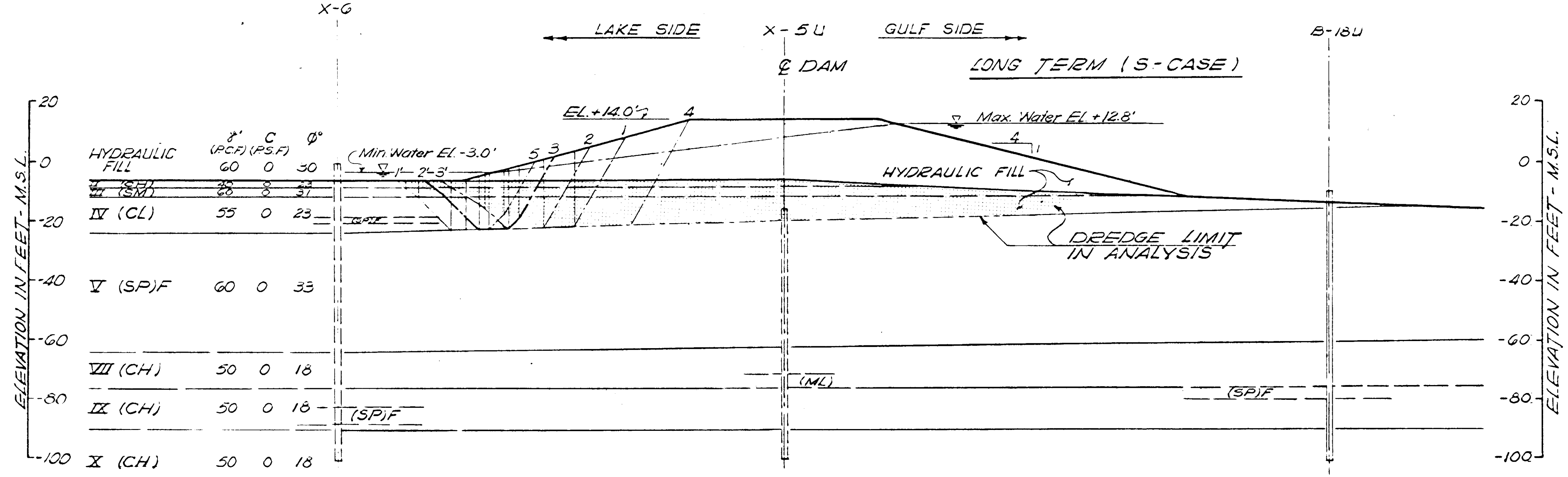
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

STABILITY ANALYSIS CLOSURE DAM. STA. 73+44

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





HORIZ 10 0 20 40 60 80 100

SCALE IN FEET

SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-23.0	31978	8224	5941	29246	21319	17150	9141	47611	1.63
12'	-23.0	31978	8103	5941	29367	21319	14650	7101	43070	1.47
13'	-23.0	31978	9213	5941	28256	21319	12400	8697	42416	1.50
21'	-23.0	23289	8224	4024	19089	15526	7750	9141	32418	1.69
22'	-23.0	23289	8103	4024	19210	15526	5250	7101	27877	*1.45
23'	-23.0	23289	9213	4024	18100	15526	3000	8697	27223	1.50
31'	-23.0	18679	8224	2620	13076	11090	4750	9141	24982	1.91
32'	-23.0	18679	8103	2620	13196	11090	2500	7101	20692	1.56
33'	-23.0	18679	9213	2620	12086	11090		8697	19787	1.63
41'	-23.0	44531	8224	8611	44918	29687	39083	9141	77912	1.73
42'	-23.0	44531	8103	8611	45038	29687	36583	7101	73371	1.63
43'	-23.0	44531	9213	8611	43928	29687	34333	8697	72717	1.66

Critical Wedge 22' F.S. = 1.45
With Earthquake Loading +DH = 0.05W

				+DH						
22'	-23.0	23289	8103	4544	19730	15526	5250	7101	27877	1.41

LONG TERM STABILITY

SLIP SURFACE NO.	ELEV.	DRIVING +DA	-DP	FORCES +DW	TOTAL	RESISTING +RA	+RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-23.0	31978	5558	4024	30445	21319	24162	7647	53128	1.74
12'	-23.0	31978	5548	4024	30455	21319	20031	7507	48851	1.60
13'	-23.0	31978	6971	4024	29032	21319	16289	10792	48400	1.67
21'	-23.0	23289	5553	4024	21759	15526	14765	7637	37928	1.74
22'	-23.0	23289	5548	4024	21765	15526	10634	7507	33667	1.55
23'	-23.0	23289	6971	4024	20342	15526	6892	10792	33210	1.63
31'	-23.0	21077	5553	2620	18144	13308	7768	7637	28713	1.59
32'	-23.0	21077	5548	2620	18150	13308	3636	7507	24451	*1.35
33'	-23.0	21077	6971	2620	16727	13308		10792	24101	1.44
41'	-23.0	44531	5553	8611	47588	29687	44472	7637	81796	1.72
42'	-23.0	44531	5548	8611	47594	29687	40642	7507	77836	1.64
43'	-23.0	44531	6971	8611	46171	29687	36511	10792	76890	1.67
51'	-23.0	16903	5553	631	11981	10521	5566	7637	23725	1.98
52'	-23.0	16903	5548	631	11987	10521	1774	7507	19803	1.65

Critical Wedge 32' F.S. = 1.35
With Earthquake Loading, +DH = 0.05W

				+DH						
32'	-23.0	21077	5548	2887	18516	13308	3636	7507	24451	1.32

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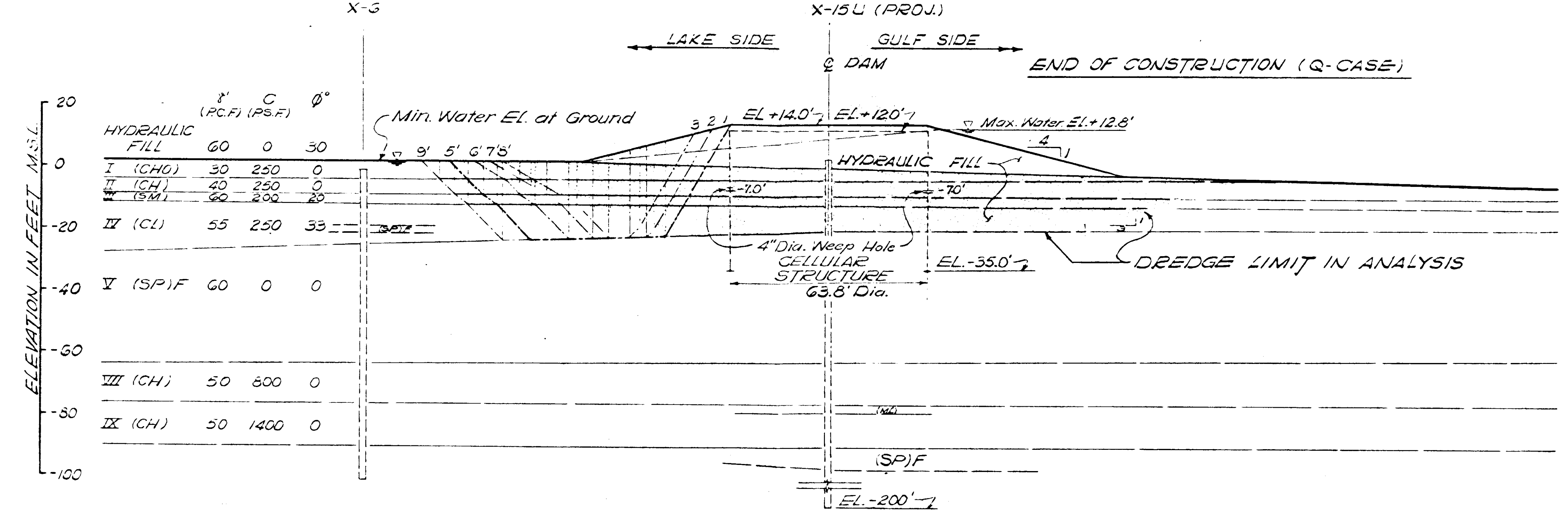
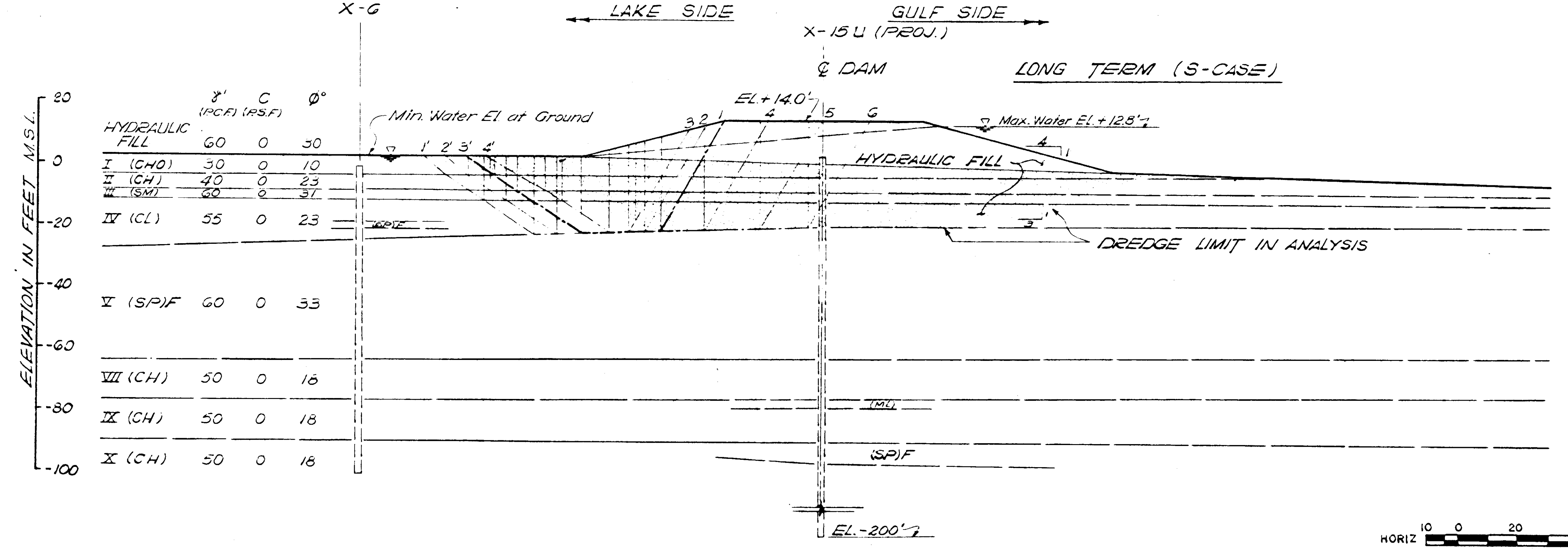
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
STABILITY ANALYSIS
CLOSURE DAM
STA. 77+54
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





SLIDING BLOCK STABILITY ANALYSIS

END OF CONSTRUCTION STABILITY

SLIP SURFACE NO.	ELEV.	+DA	DRIVING -DA	FORCES +DW	TOTAL	+RA	RESISTING +RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
15'	-23.0	43777	12744	5148	36181	29185	10750	12142	52077	*1.43
16'	-23.0	43777	16705	5148	32220	29185	7750	13696	50632	1.57
17'	-23.0	43777	13418	5148	35507	29185	5500	17075	51760	1.45
18'	-23.0	43777	17944	5148	30980	29185	4500	16589	50274	1.62
19'	-23.0	43777	12744	5148	36181	29185	13000	12142	54327	1.50
25'	-23.0	40975	12744	4251	32481	25227	9250	12142	46620	1.43
26'	-23.0	40975	16705	4251	28520	25227	6250	13696	45174	1.58
27'	-23.0	40975	13418	4251	31807	25227	4000	17075	46302	1.45
28'	-23.0	40975	17944	4251	27281	25227	3000	16589	44816	1.64
29'	-23.0	40975	12744	4251	32481	25227	11500	12142	48870	1.50
35'	-23.0	34250	12744	3369	24875	21073	8000	12142	41215	1.65
36'	-23.0	34250	16705	3369	20914	21073	4750	13696	39520	1.88
37'	-23.0	34250	13418	3369	24201	21073	2750	17075	40898	1.68
38'	-23.0	34250	17944	3369	19675	21073	1750	16589	39412	2.00
39'	-23.0	34250	12744	3369	24875	21073	10250	12142	43465	1.74

Critical Wedge 15' F-S = 1.43
With Earthquake Loading +DH = 0.05W

				+DH						
15'	-23.0	43777	12744	10876	41909	29185	10750	12142	52077	1.24

LONG TERM STABILITY

SLIP SURFACE NO.	ELEV.	+DA	DRIVING -DA	FORCES +DW	TOTAL	+RA	RESISTING +RB	FORCES +RP	TOTAL	SAFETY FACTOR R/D
11'	-23.0	43777	14685	6060	35153	29185	28616	19798	77600	2.20
12'	-23.0	43777	15836	6060	34001	29185	24323	21251	74759	2.19
13'	-23.0	43777	16033	6060	33804	29185	19818	23014	72017	*2.13
14'	-23.0	43777	17594	6060	32244	29185	14173	26812	70170	2.17
21'	-23.0	39152	14685	5148	29615	25797	24408	19798	70005	2.36
22'	-23.0	39152	15836	5148	28464	25797	20115	21251	67164	2.35
23'	-23.0	39152	16033	5148	28266	25797	15610	23014	64422	2.27
24'	-23.0	39152	17594	5148	26706	25797	9965	26812	62575	2.34
31'	-23.0	31934	14685	4251	21500	20977	20413	19798	61190	2.84
32'	-23.0	31934	15836	4251	20349	20977	16119	21251	58349	2.86
33'	-23.0	31935	16033	4251	20153	20978	11614	23014	55607	2.75
34'	-23.0	31934	17594	4251	18591	20977	5970	26812	53760	2.89
43'	-23.0	44052	17080	8892	35864	29368	23851	23722	76943	2.14
53'	-23.0	48393	17080	10857	42170	32262	48474	23722	104559	2.48
63'	-23.0	47212	17080	14976	45107	21474	63933	23722	119131	2.64

Critical Wedge 13' F-S = 2.13
With Earthquake Loading +DH = 0.05W

				+DH						
13'	-23.0	43777	16033	8478	36222	29185	19818	23014	72017	1.99

HORIZ 10 0 20 40 60 80 100
SCALE IN FEET

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

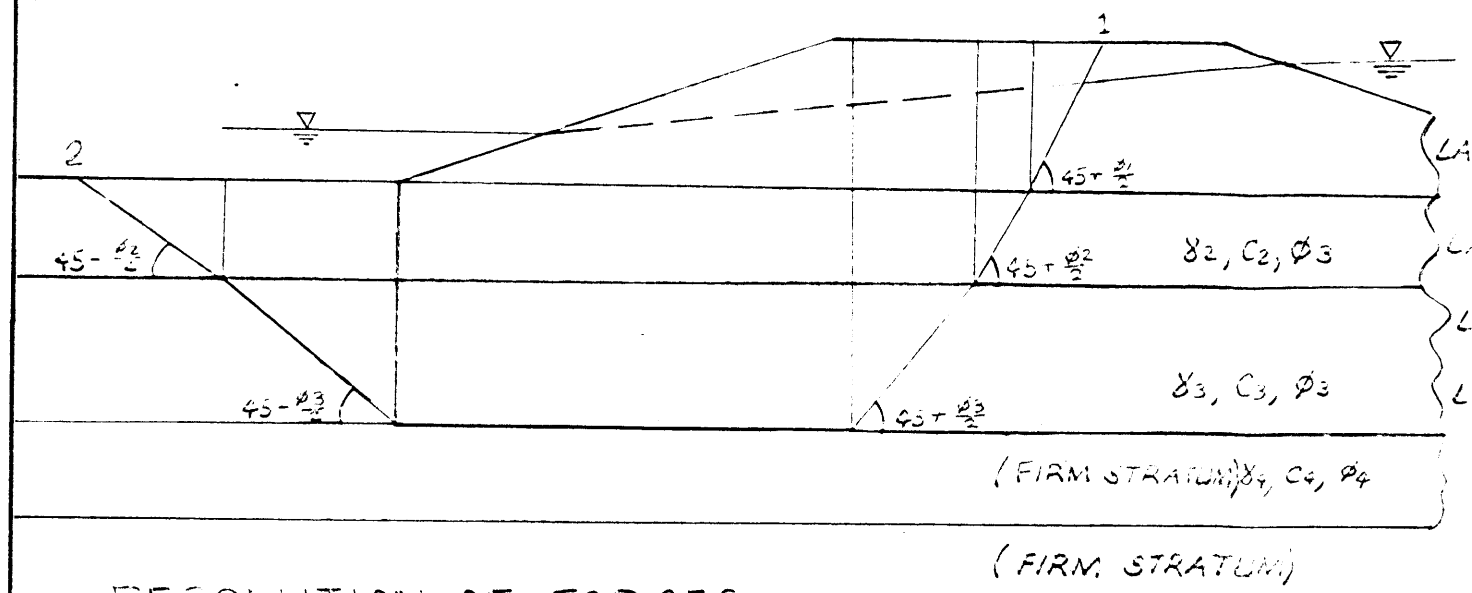
LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

STABILITY ANALYSIS
CLOSURE DAM
STA. 80+54

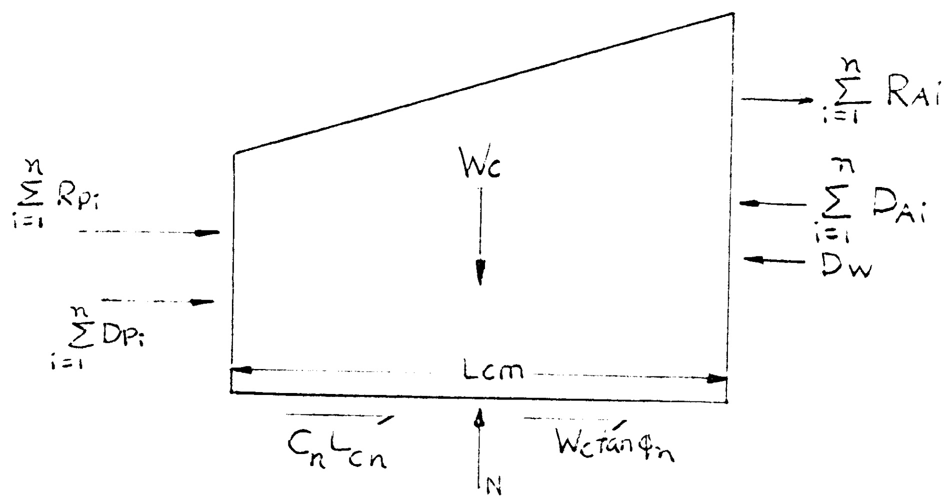
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III-9



CENTRAL WEDGE



Wc = WEIGHT OF CENTRAL WEDGE

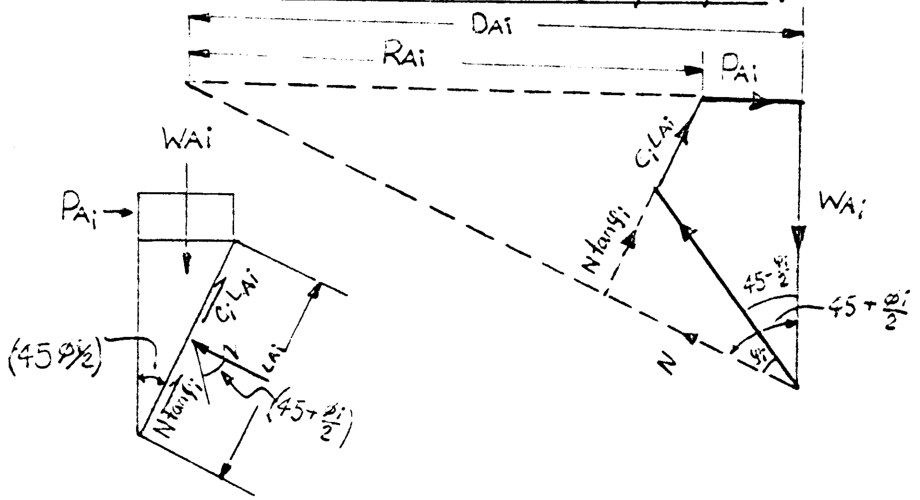
CN, PN = SOIL PROPERTIES OF SLIDING STRATA i = n

Lcn = LENGTH OF CENTRAL WEDGE

Dw = RESULTANT OF NET PORE WATER PRESSURE ACTING ON THE TWO ENDS OF CENTRAL WEDGE

RESOLUTION OF FORCES

ACTIVE WEDGE (say Layer i)



WAi = WEIGHT OF ACTIVE WEDGE IN LAYER i

DAi = WAi tan $(45+\frac{\phi_i}{2})$ = HORIZONTAL DRIVING FORCE ASSUMING NO SHEAR STRENGTH ON LAi

PAi = ORDINARY RESULTANT ACTIVE FORCE

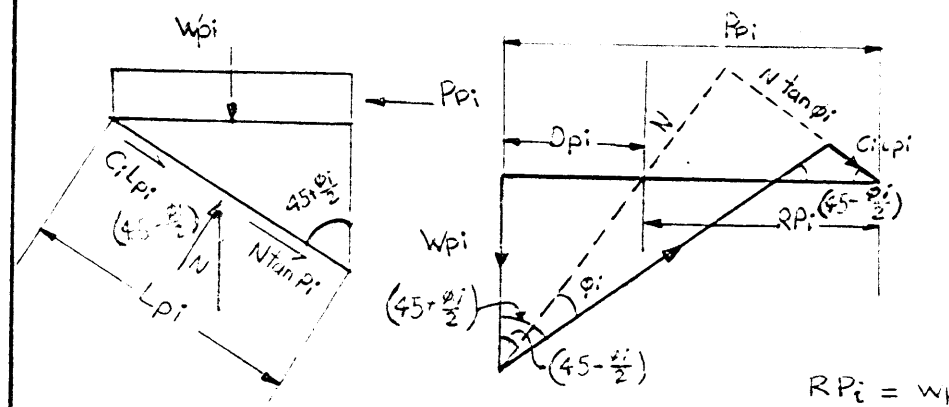
PAi = WAi tan $(45-\frac{\phi_i}{2}) - 2C_i L_{Ai} \cos(45+\frac{\phi_i}{2})$

RAi = HORIZONTAL EQUIVALENT OF SHEAR STRENGTH ON LAi

RAi = DAi - PAi

$$= W_{Ai}\left[\tan(45+\tfrac{\phi_i}{2}) - \tan(45-\tfrac{\phi_i}{2})\right] + 2C_i L_{Ai}\cos(45+\tfrac{\phi_i}{2})$$

PASSIVE WEDGE (say Layer i)



WPi = WEIGHT OF PASSIVE WEDGE IN LAYER i

DPi = WPi tan $(45-\frac{\phi_i}{2})$ = HORIZONTAL DRIVING FORCE ASSUMING NO SHEAR STRENGTH ON LPi

PPi = ORDINARY RESULTANT PASSIVE FORCE

PPi = WPi tan $(45+\frac{\phi_i}{2}) + 2C_i L_{Pi}\cos(45-\frac{\phi_i}{2})$

RPi = HORIZONTAL EQUIVALENT OF SHEAR STRENGTH ON LPi = PPi - DPi

$$R_{Pi} = W_{Pi}\left[\tan(45+\tfrac{\phi_i}{2}) - \tan(45-\tfrac{\phi_i}{2})\right] + 2C_i L_{Pi}\cos(45-\tfrac{\phi_i}{2})$$

$$D_W = \gamma_W\left[\tfrac{1}{2}(\text{EL. UP} - \text{EL. DN})^2 + (\text{EL. UP} - \text{EL. DN})(\text{EL. DN} - \text{EL. SL})\right]$$

EL. UP = WATER ELEVATION ON ACTIVE SIDE OF CENTRAL WEDGE

EL. DN = WATER ELEVATION ON PASSIVE SIDE OF CENTRAL WEDGE

EL. SL = ELEVATION OF THE SLIDING SURFACE OF THE SLIDING WEDGE

γ_w = UNIT WEIGHT OF WATER = 62.4 PCF

FACTOR OF SAFETY OF ENTIRE SLIDING MASS $= \dfrac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}} = \dfrac{\sum_{i=1}^{n} R_{Ai} + \sum_{i=1}^{n} R_{Pi} + C_n L_{cn} + W_c \tan\phi_n}{\sum_{i=1}^{n} D_{Ai} - \sum_{i=1}^{n} D_{Pi} + D_W}$

$$\text{F.S.} = \frac{R}{D} = \frac{RA + RP + RB}{DA - DP - DW}$$

$$\sum_{i=1}^{n} R_{Ai} = RA, \quad \sum_{i=1}^{n} R_{Pi} = RP$$

$$\sum_{i=1}^{n} D_{Ai} = DA, \quad \sum_{i=1}^{n} D_{Pi} = DP, \quad D_W = DW$$

$$C_n L_{cn} + W_c \tan\phi_n = RB$$

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

STABILITY ANALYSIS - METHOD -

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

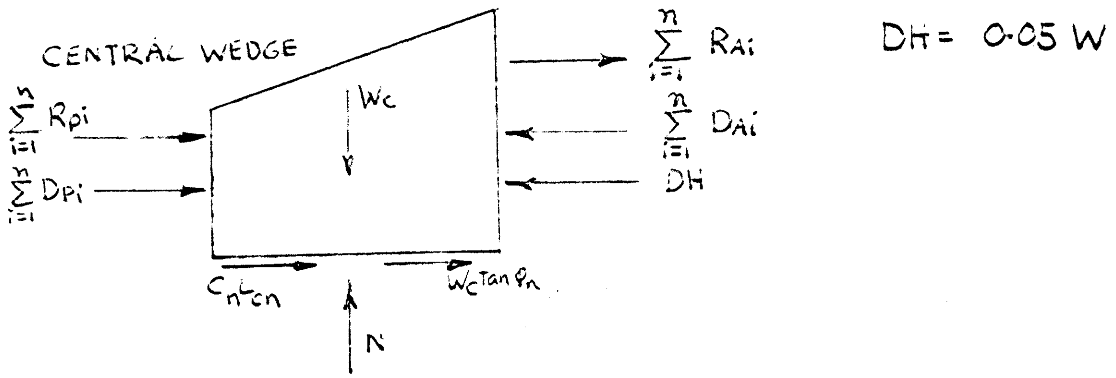
DATE JULY 1972 FILE NO. H-2-24417

ANALYSIS FOR EARTHQUAKE CONDITION

SEISMIC COEFFICIENT FOR THIS AREA IS 0.05.

HORIZONTAL DRIVING FORCE DUE TO EARTHQUAKE, DH = 0.05W. W = SATURATED WEIGHT OF THE ENTIRE SLIDING MASS. THIS ANALYSIS WAS PERFORMED FOR THE CRITICAL SLIDING WEDGE DETERMINED FOR END OF CONSTRUCTION (Q - CASE) AND LONG TERM (S - CASE) STABILITY.

COMBINATION OF EARTHQUAKE AND WORST COMBINATION OF STORM AND HIGH TIDE OCCURRING SIMULTANEOUSLY IS REMOTE. HENCE, NORMAL WATER LEVEL CONDITION, EL + 1.50 ON BOTH SIDES OF THE DAM IS USED FOR EARTHQUAKE CONDITION ANALYSIS. DW = 0.



FACTOR OF SAFETY (F.S.) $= \dfrac{\text{RESISTING FORCES}}{\text{DRIVING FORCES}}$

$$= \frac{\sum_{i=1}^{n} R_{Ai} + \sum_{i=1}^{n} R_{Pi} + C_n L_{cn} + W_c \tan\phi_n}{\sum_{i=1}^{n} D_{Ai} - \sum_{i=1}^{n} D_{Pi} + D_h}$$

$$= \frac{RA + RP + RB}{DA - DP + DH}$$

USE RA, RP, RB, DA AND DP FROM THE CRITICAL WEDGE DATA FOR COMPUTING FACTOR OF SAFETY AGAINST SLIDING FOR EARTHQUAKE CONDITION.

WANG CALCULATOR WAS USED TO COMPUTE F.S. GIVEN THE SOIL PROPERTIES, IDENTIFICATION OF SLIDING LAYER, WEIGHT AND LENGTH OF SLIDING SURFACE OF ACTIVE, PASSIVE AND CENTRAL WEDGES OF THE SLIDING BLOCK.

WANG CALCULATOR INPUT

ϕ_i	i =1 . . . . J. J = TOTAL NUMBER OF
c_i	i =1 . . . . J, SOIL STRATA IN CROSS SECTION
N = n	NUMBER OF LAYER IN WHICH SLIDING TAKES PLACE, $n < J$
W_{Ai}, L_{Ai}	ACTIVE WEDGE WEIGHT AND LENGTH OF SLIDING SURFACE IN EACH STRATA $i = 1, \ldots, n$
W_{Pi}, L_{Pi}	PASSIVE WEDGE, WEIGHT AND LENGTH OF SLIDING SURFACE IN EACH STRATA $i = 1, \ldots, n$
W_c, L_{cn}	CENTRAL WEDGE WEIGHT AND LENGTH OF SLIDING SURFACE IN n^{th} STRATA
EL. SL, EL. UP, EL. DN,	ELEVATIONS OF SLIDING SURFACE, WATER ELEVATIONS UP, UPSTREAM (ACTIVE) SIDE, DN, DOWN STREAM (PASSIVE) SIDE OF CENTRAL WEDGE.
TRIAL NO.	TO IDENTIFY THE TRIAL SLIDING WEDGE.

STABILITY COMPUTATION SHEETS INDICATE COMPUTATION OF WEIGHTS AND LENGTH OF SLIDING SURFACES OF ACTIVE, PASSIVE AND CENTRAL (NEUTRAL) SLIDING WEDGES OF SEVERAL TRIALS TO DETERMINE THE SLIDING WEDGE WITH THE LEAST FACTOR OF SAFETY.

THE CRITICAL WEDGE COMPUTATION WAS CHECKED BY HAND COMPUTATION.

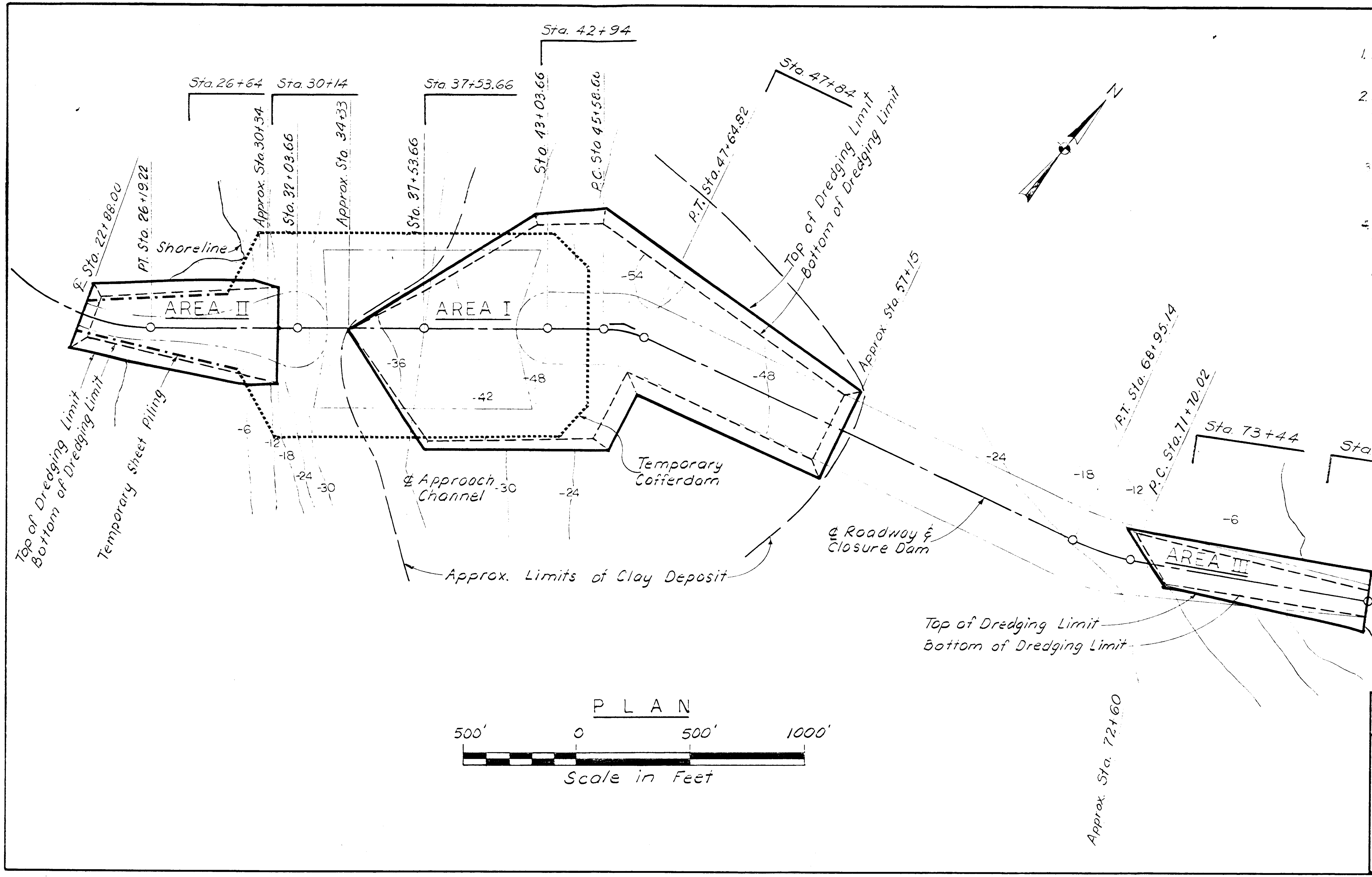
REFERENCE: DEPARTMENT OF NAVY, "DESIGN MANUAL - SOIL MECHANICS, FOUNDATIONS AND EARTH STRUCTURES" NAVFAC, DM-7, 1971

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
STABILITY ANALYSIS
– METHOD (CONT'D) –

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417



NOTES:

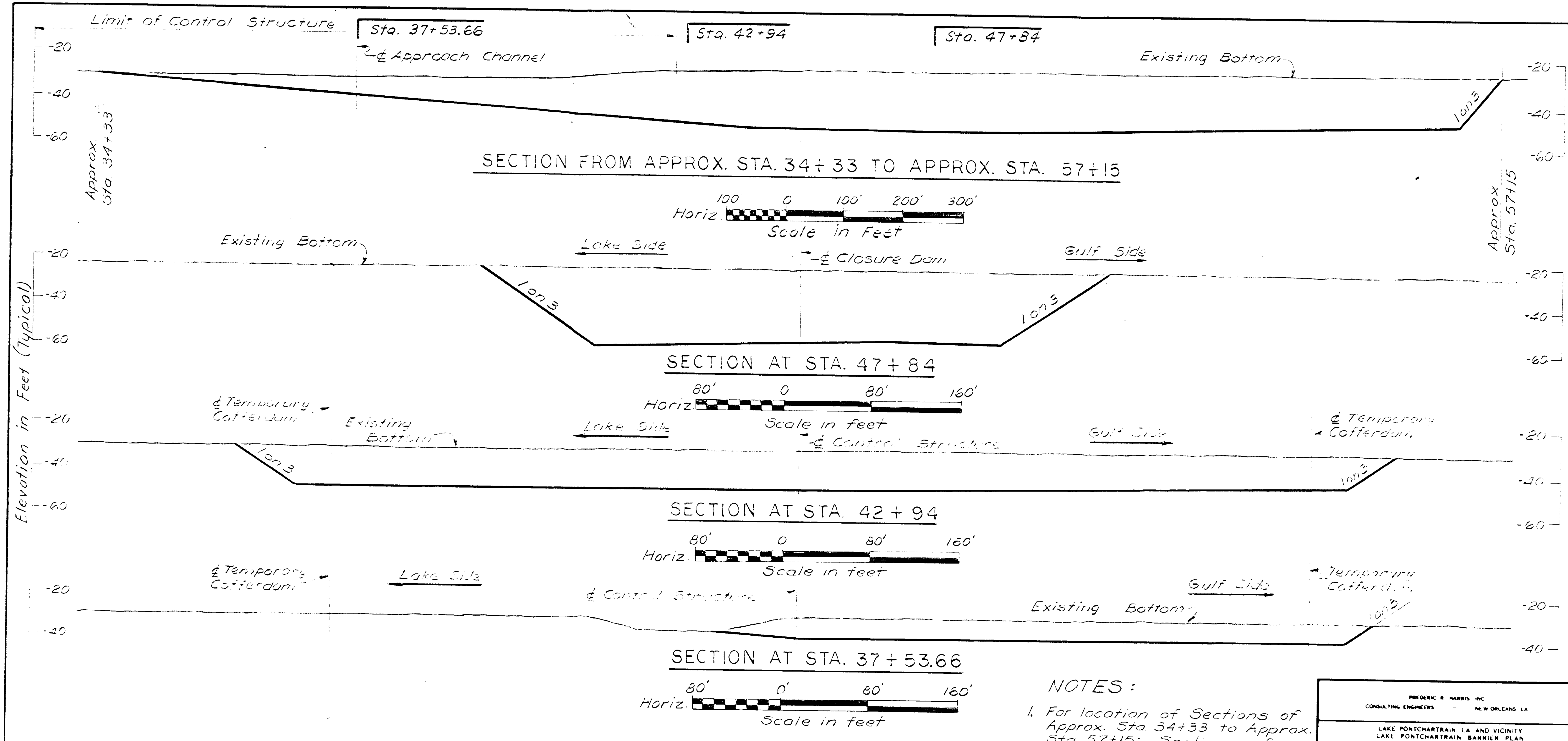
1. For Sections at Stations 26+64, 73+44 & 80+54 See Plate III-14
2. For Sections at Stations 37+53.66, 42+94, 47+84 and from Approx. Sta. 34+33 to Approx Sta. 57+15 See Plate III-13
3. Spoils Shall be deposited in in the area as directed by the Corp. of Engineers.
4. Dredged areas Shall be back filled to the original ground line, with Hydraulic Sand Fill from the borrow areas

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM
SITE PREPARATION
DREDGING PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III-12



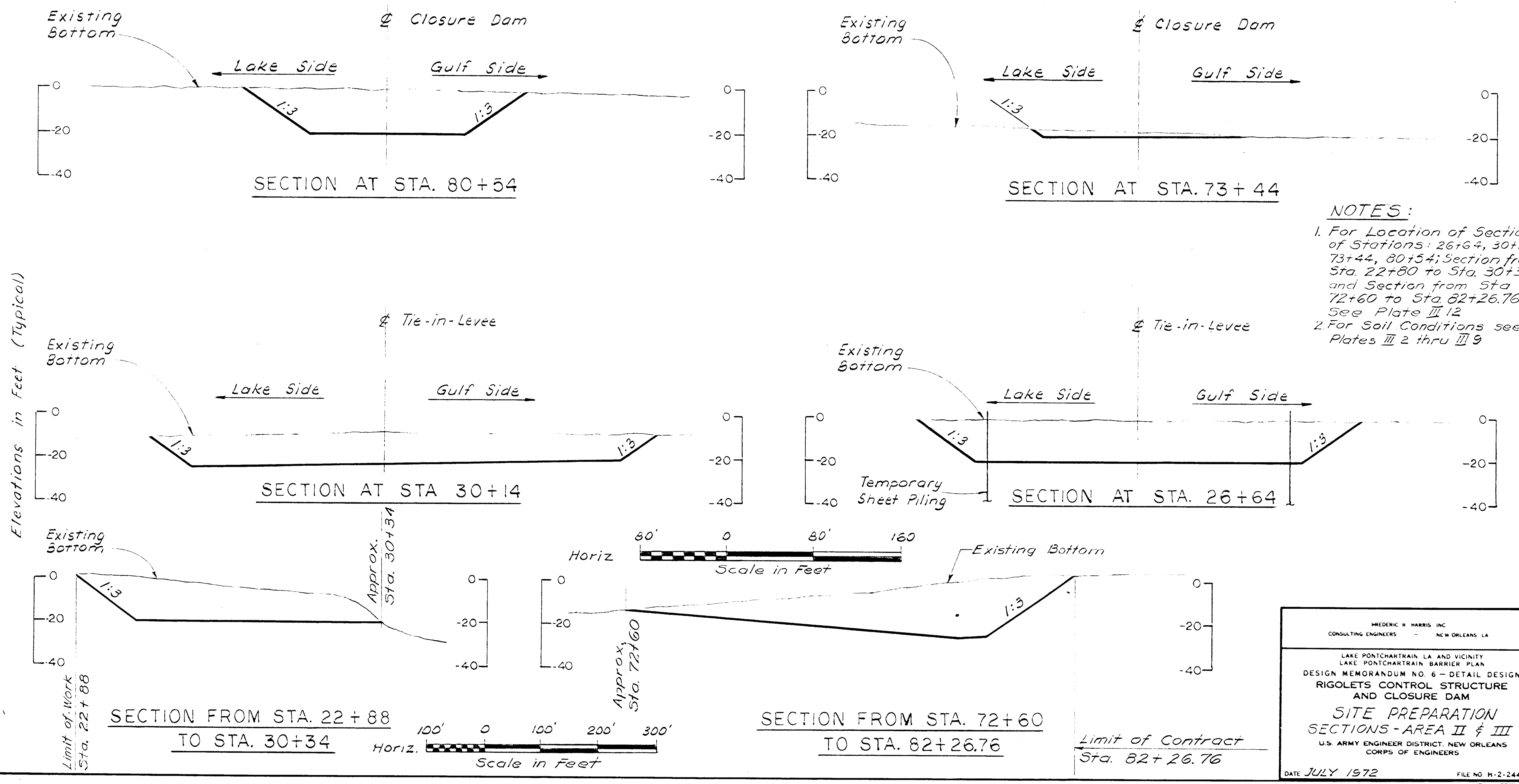
NOTES:

1. For location of Sections of Approx. Sta 34+33 to Approx. Sta. 57+15; Sections of Stations 37+53.66, 42+94 and 47+84 See Plate III 12
2. For Soil Condition See Plates III 2 thru III 9

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM
SITE PREPARATION
SECTIONS - AREA I
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417



NOTES:

1. For Location of Sections of Stations: 26+64, 30+14, 73+44, 80+54; Section from Sta. 22+80 to Sta. 30+34 and Section from Sta 72+60 to Sta 82+26.76 See Plate III 12
2. For Soil Conditions see Plates III 2 thru III 9

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS — NEW ORLEANS LA

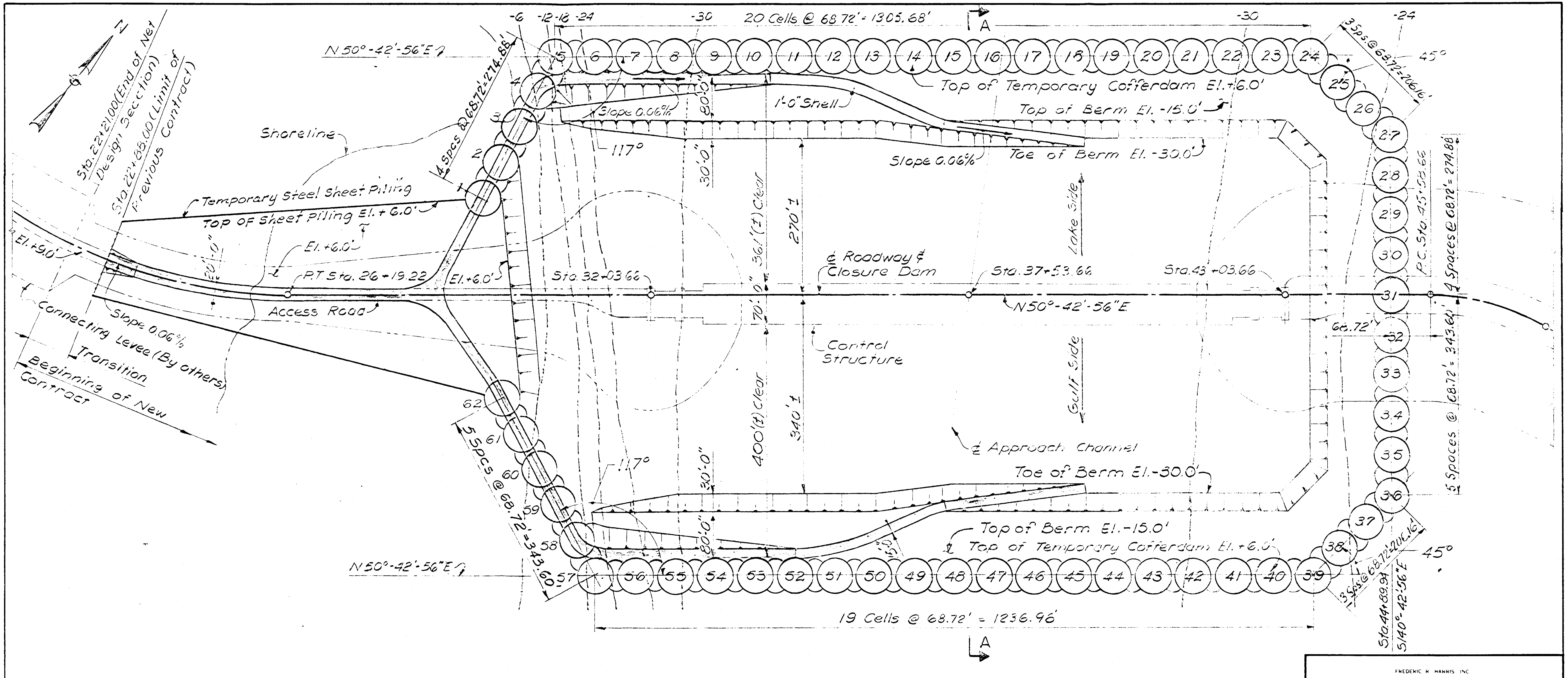
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

SITE PREPARATION
SECTIONS - AREA II & III

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III 14



Note:
For Section A-A see Plates V1, V2, V3 ＆ V4

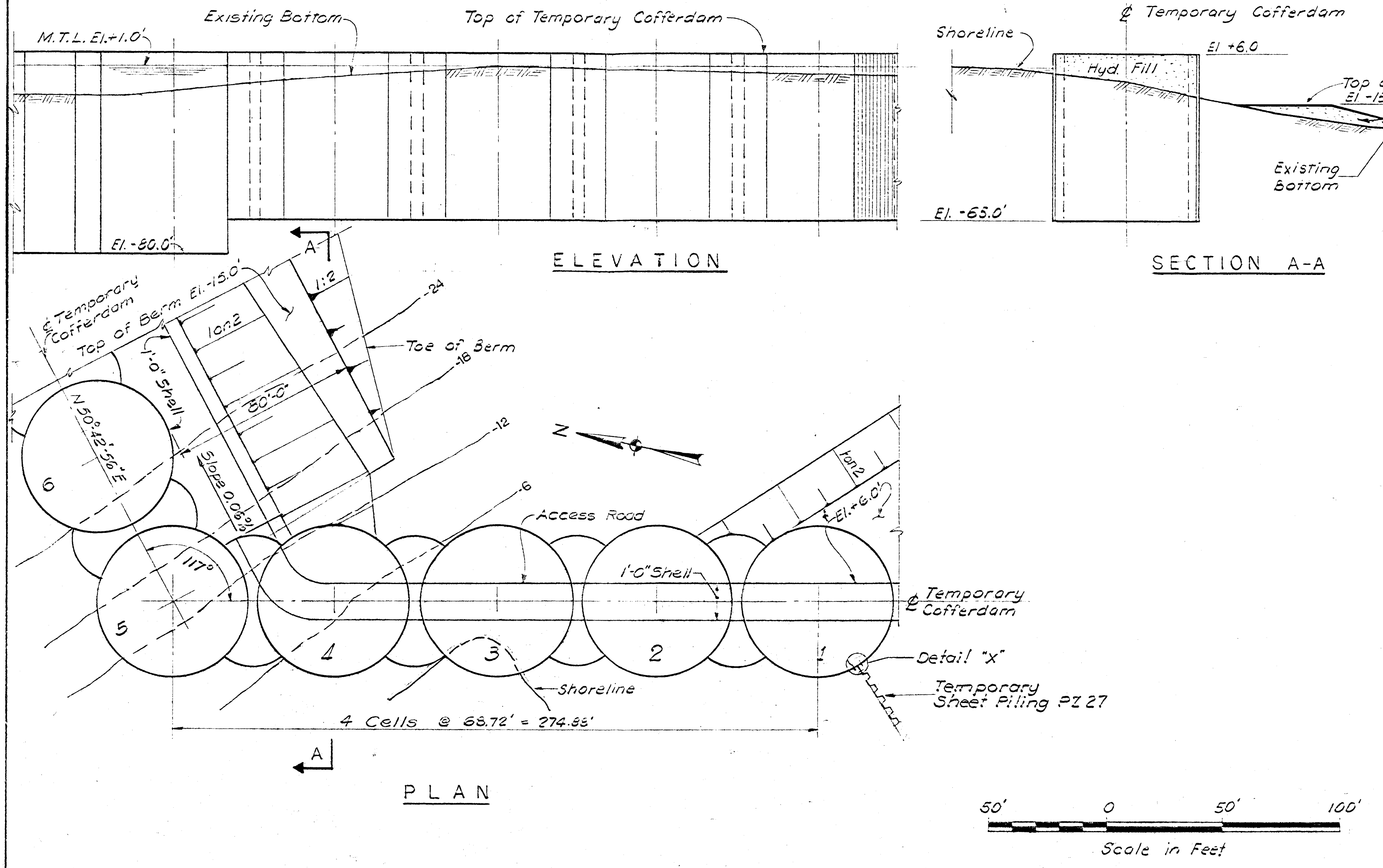
FREDERIC R. HARRIS INC
CONSULTING ENGINEERS — NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

COFFERDAM PLAN

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 — FILE NO. H-2-24417



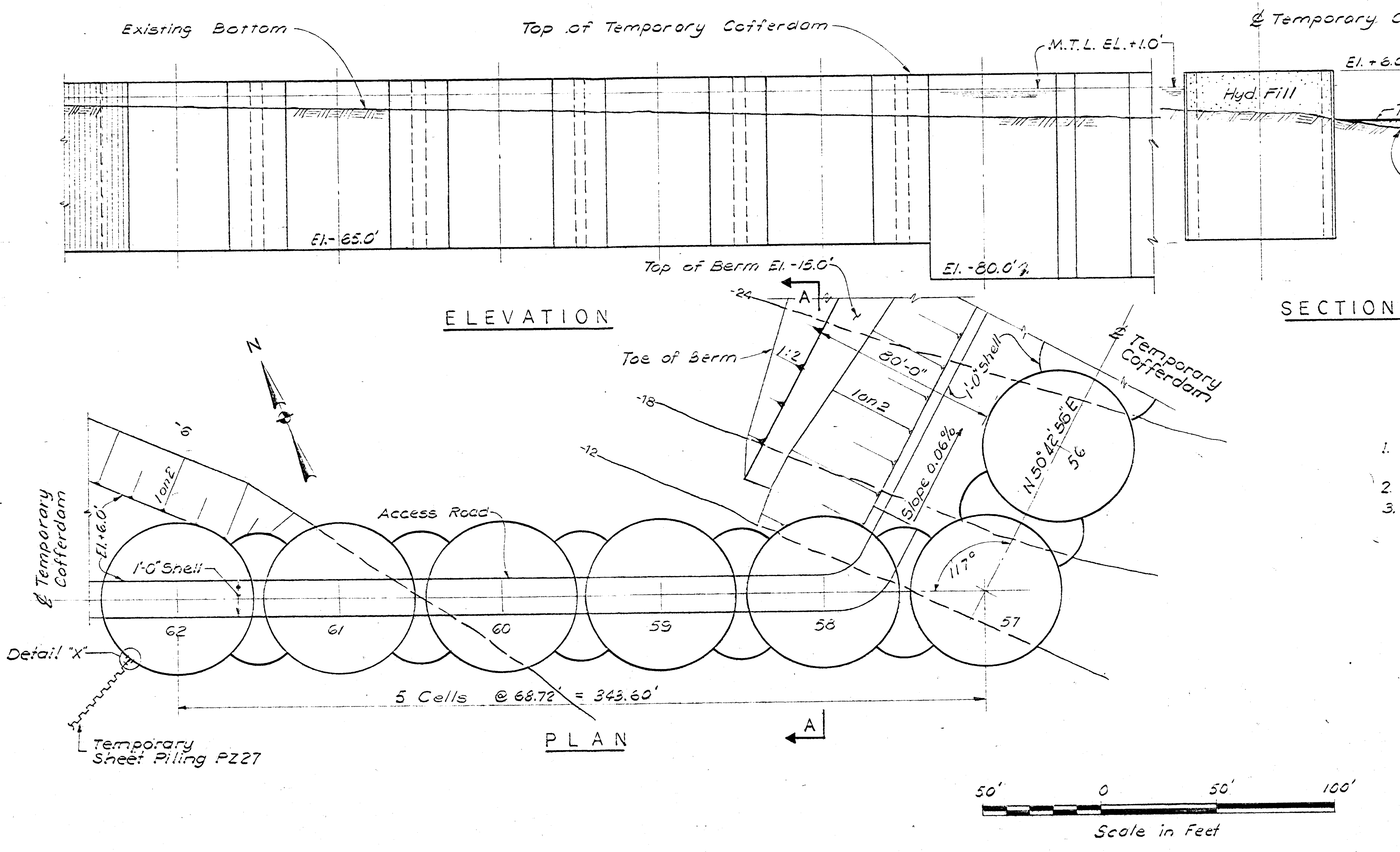
NOTES:

1. For Location of Cells see Plate III 15
2. For Detail "X" see Plate III 21
3. For Detail of Cells 1 & 5 see Plate III 21

FREDERIC R. HARRIS, INC
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
COFFERDAM
NW CORNER, PLAN & ELEVATION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417



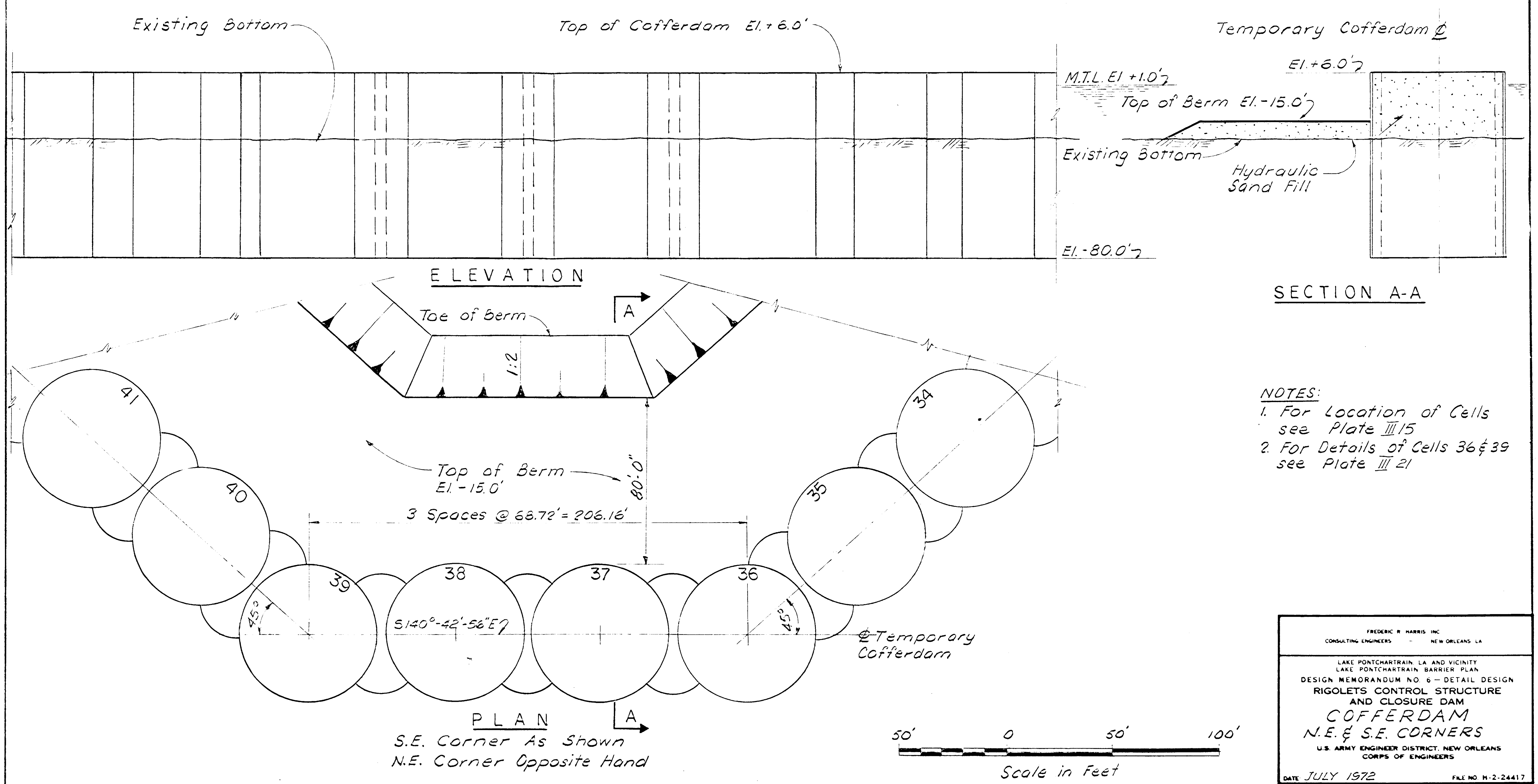
1. For Location of Cells see Plate III 15
2. For Detail "X" see Plate III 21
3. For Detail of Cells 57 & 62 see Plate III 21

FREDERIC R HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
COFFERDAM
SW CORNER, PLAN & ELEV.
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III 17



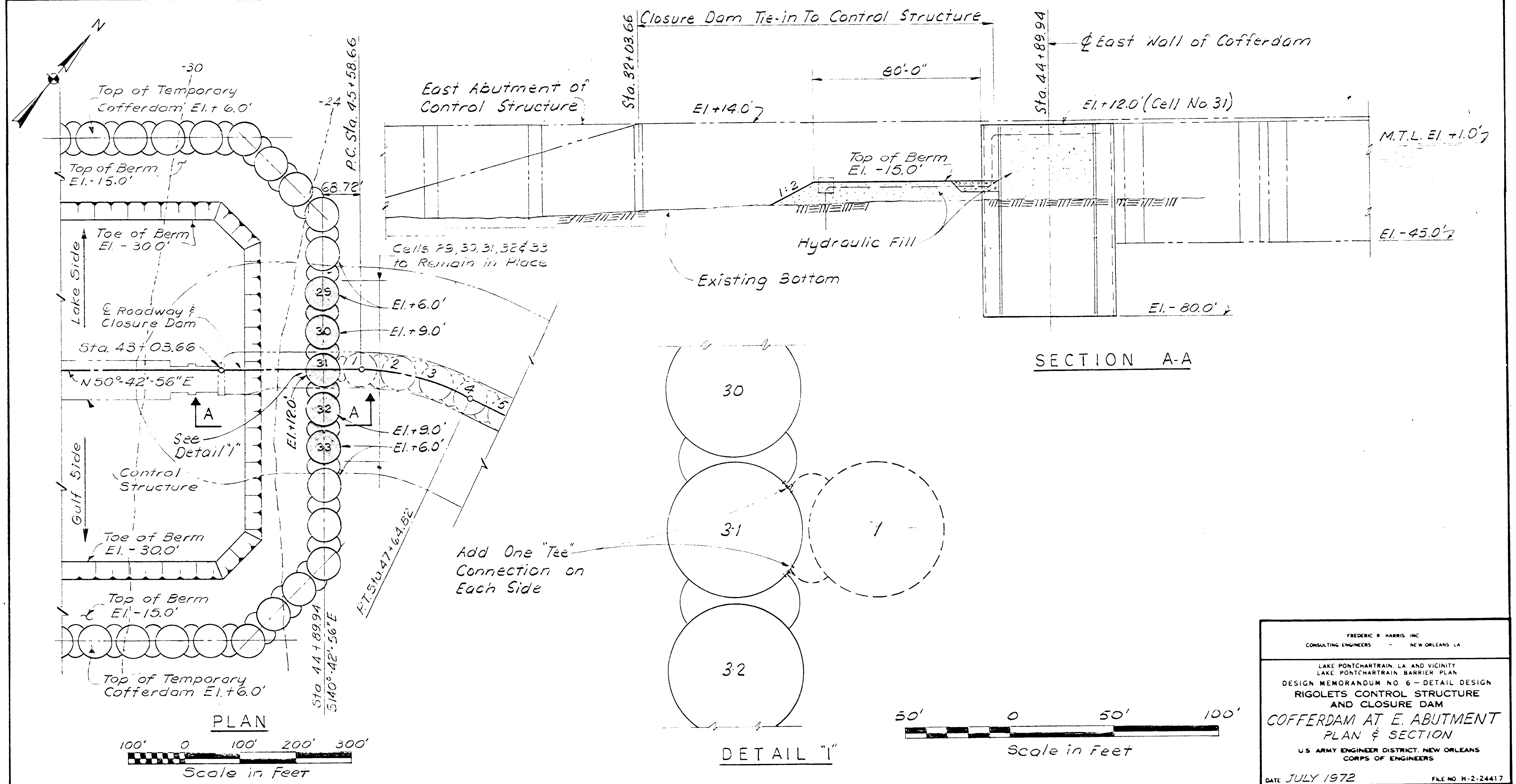
NOTES:
1. For Location of Cells see Plate III 15
2. For Details of Cells 36 & 39 see Plate III 21

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
COFFERDAM
N.E. & S.E. CORNERS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III 18



FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

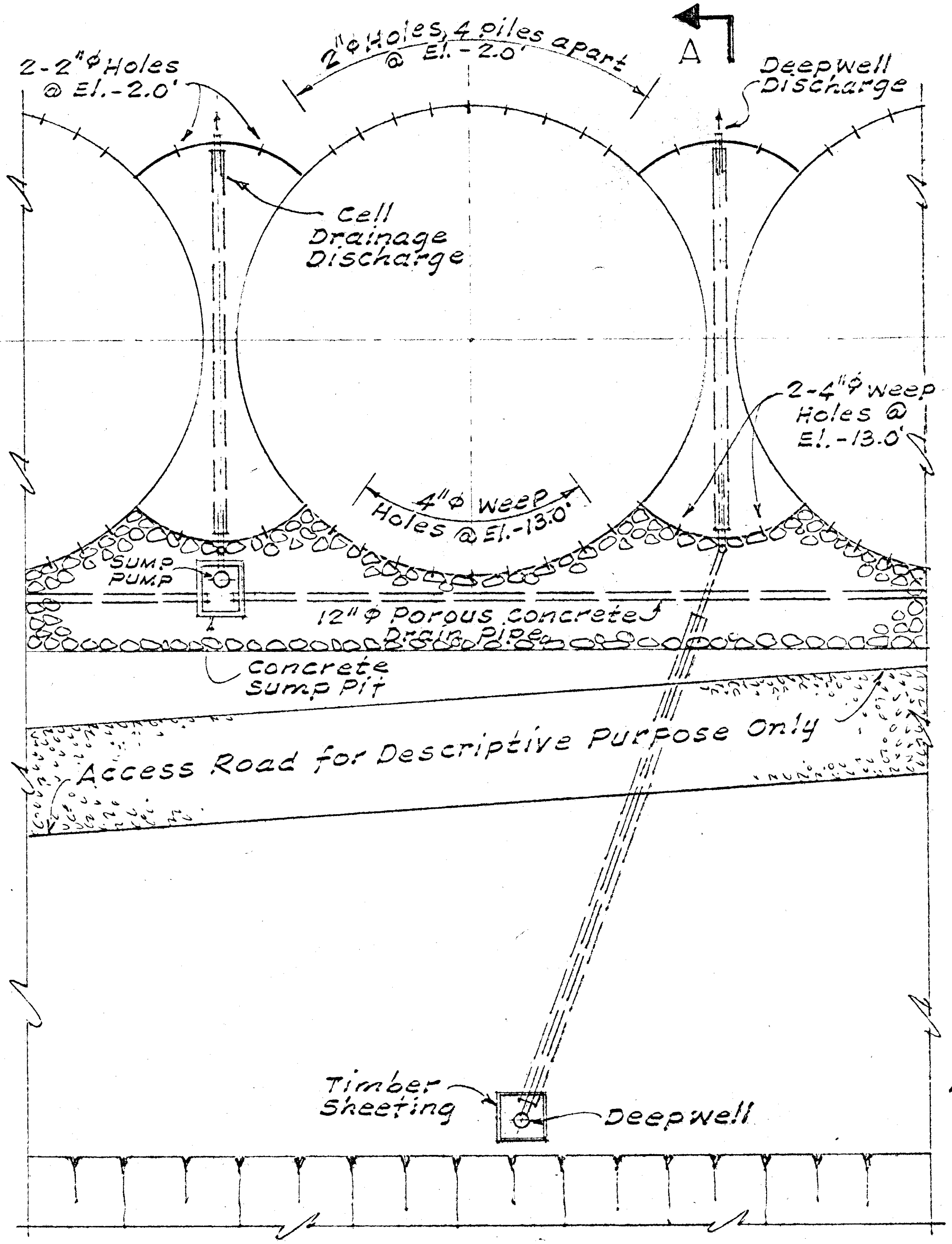
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

COFFERDAM AT E. ABUTMENT
PLAN & SECTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

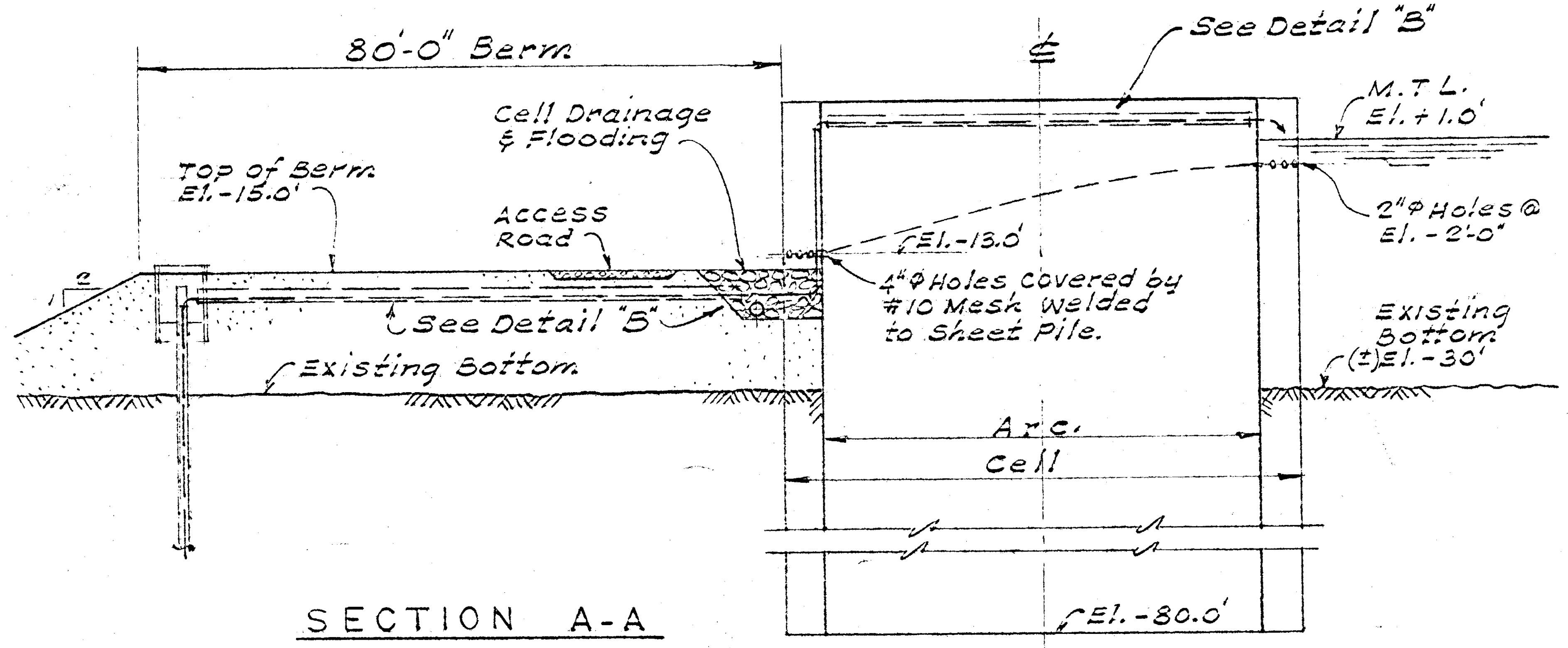
PLATE III/19



PART PLAN

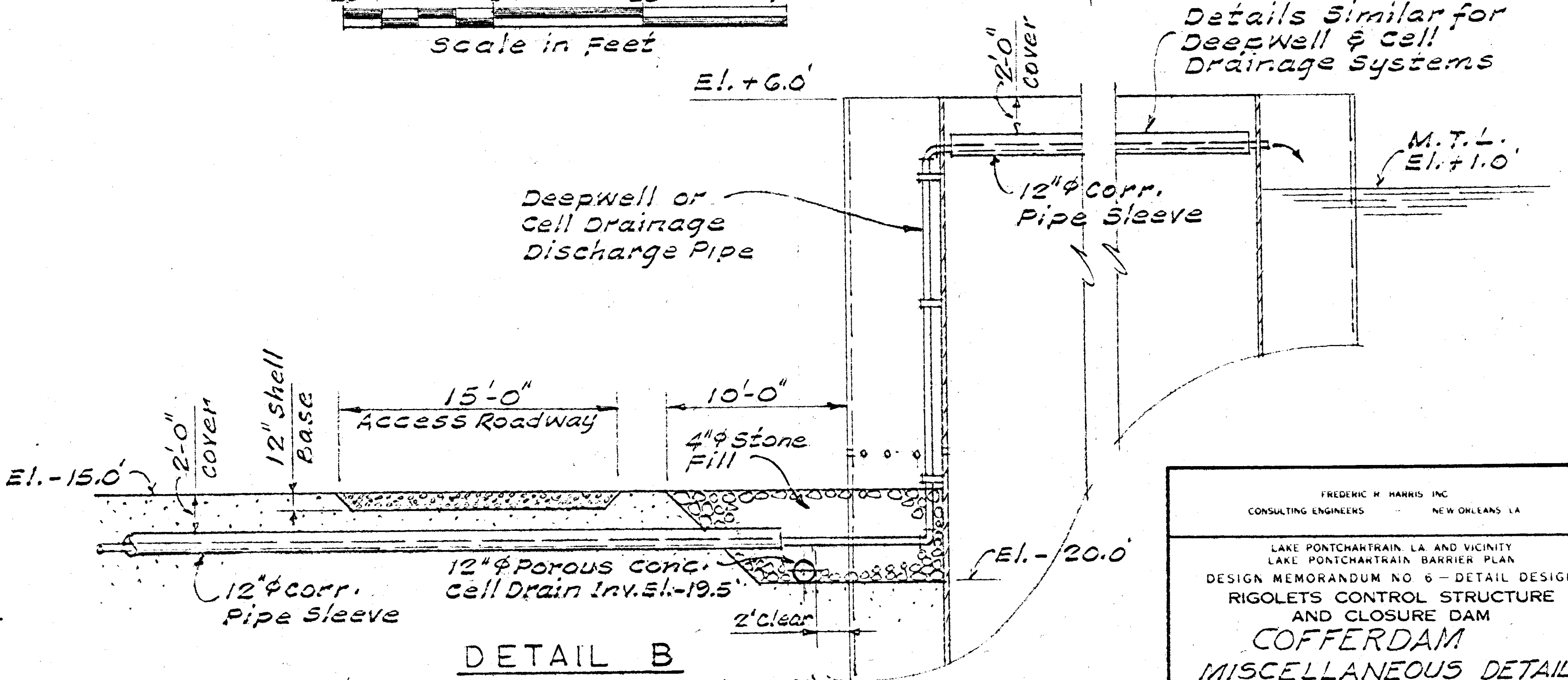
20' 0' 20' 40'
Scale in Feet

A



SECTION A-A

20' 0' 20' 40'
Scale in Feet



DETAIL B

5' 0' 5' 10' 15' 20'
Scale in Feet

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS — NEW ORLEANS LA

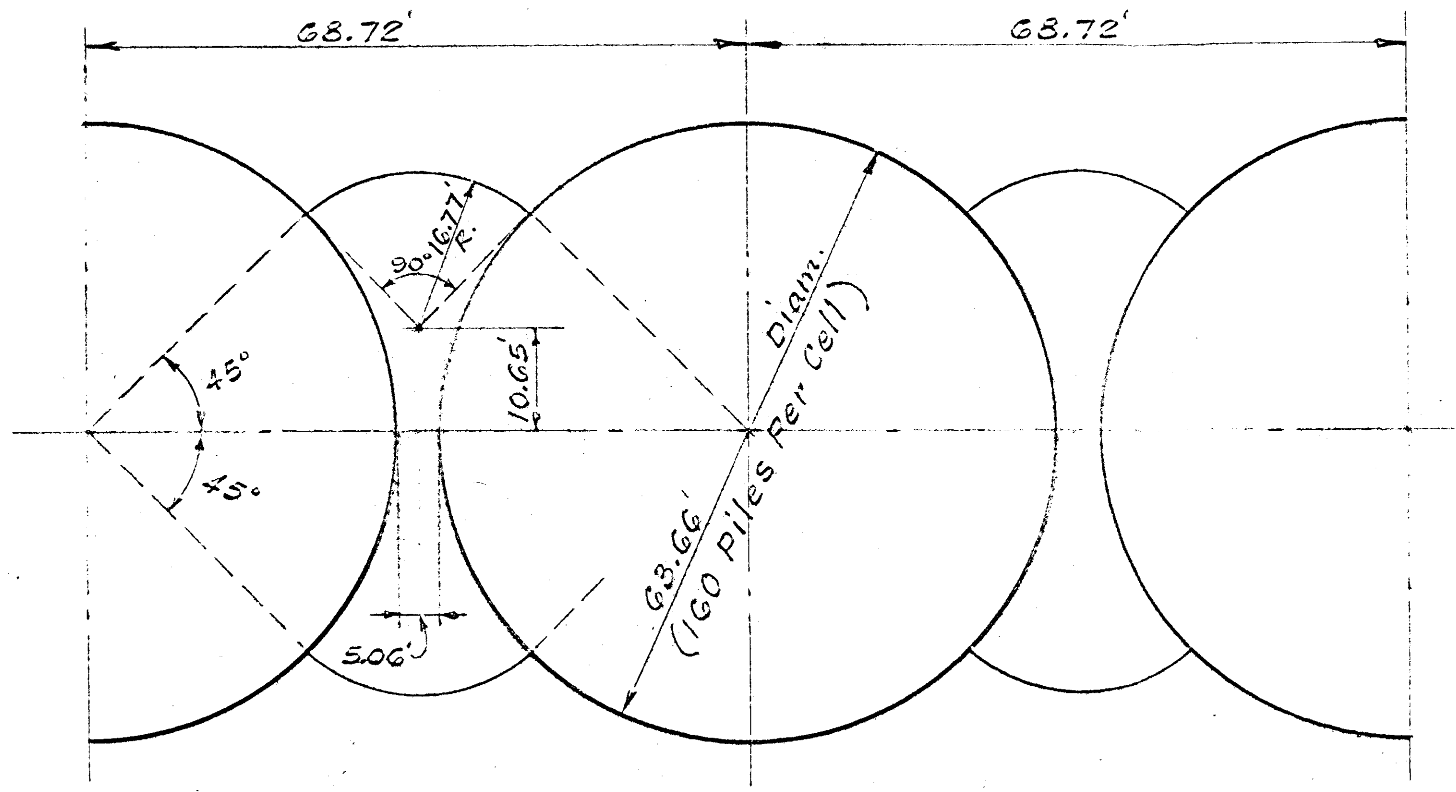
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

COFFERDAM
MISCELLANEOUS DETAILS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

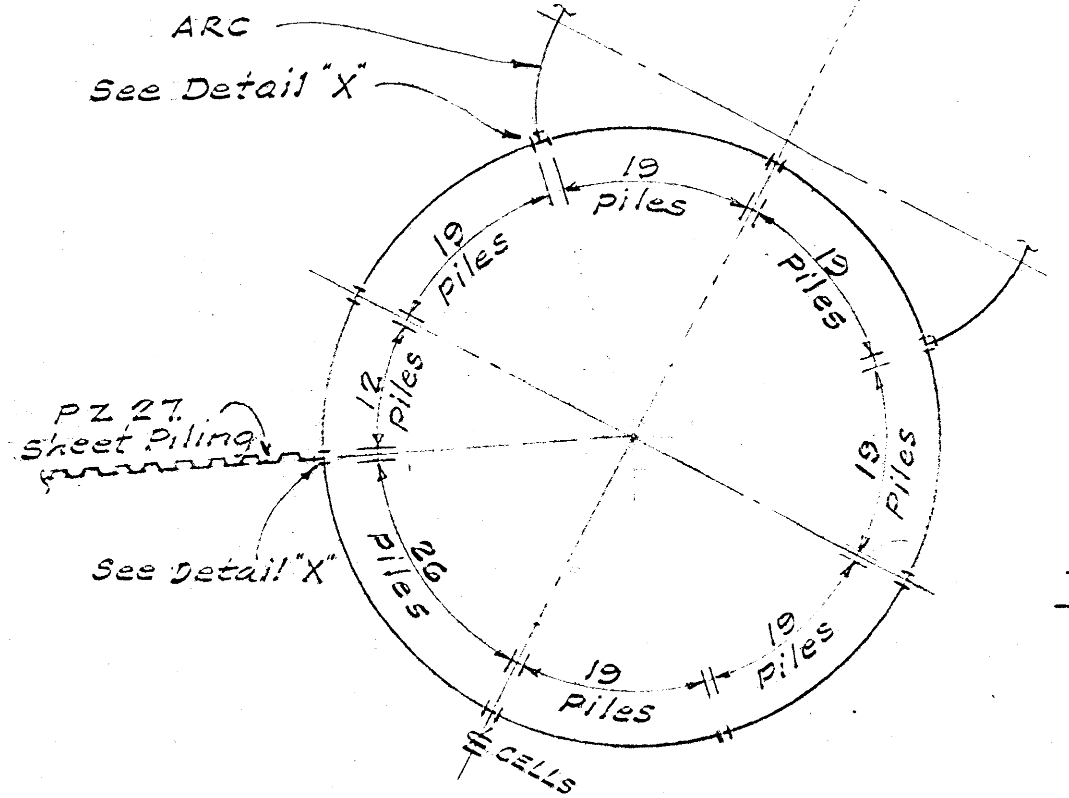
DATE: JULY 1972 FILE NO. H-2-24417

PLATE III 20



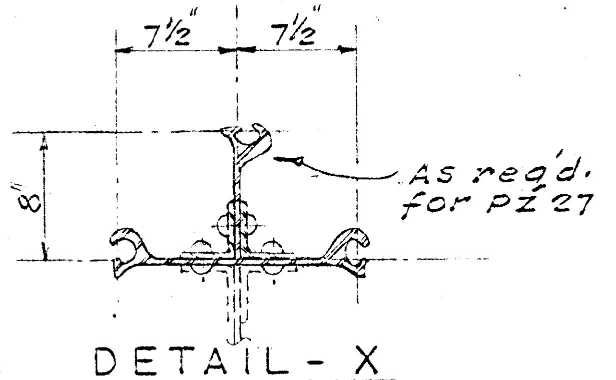
GEOMETRICAL LAYOUT

20' 0' 20' 40'
Scale in Feet

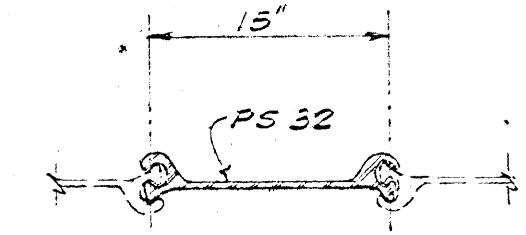


DETAIL AT CELL NO. 1

20' 0' 20' 40'
Scale in Feet

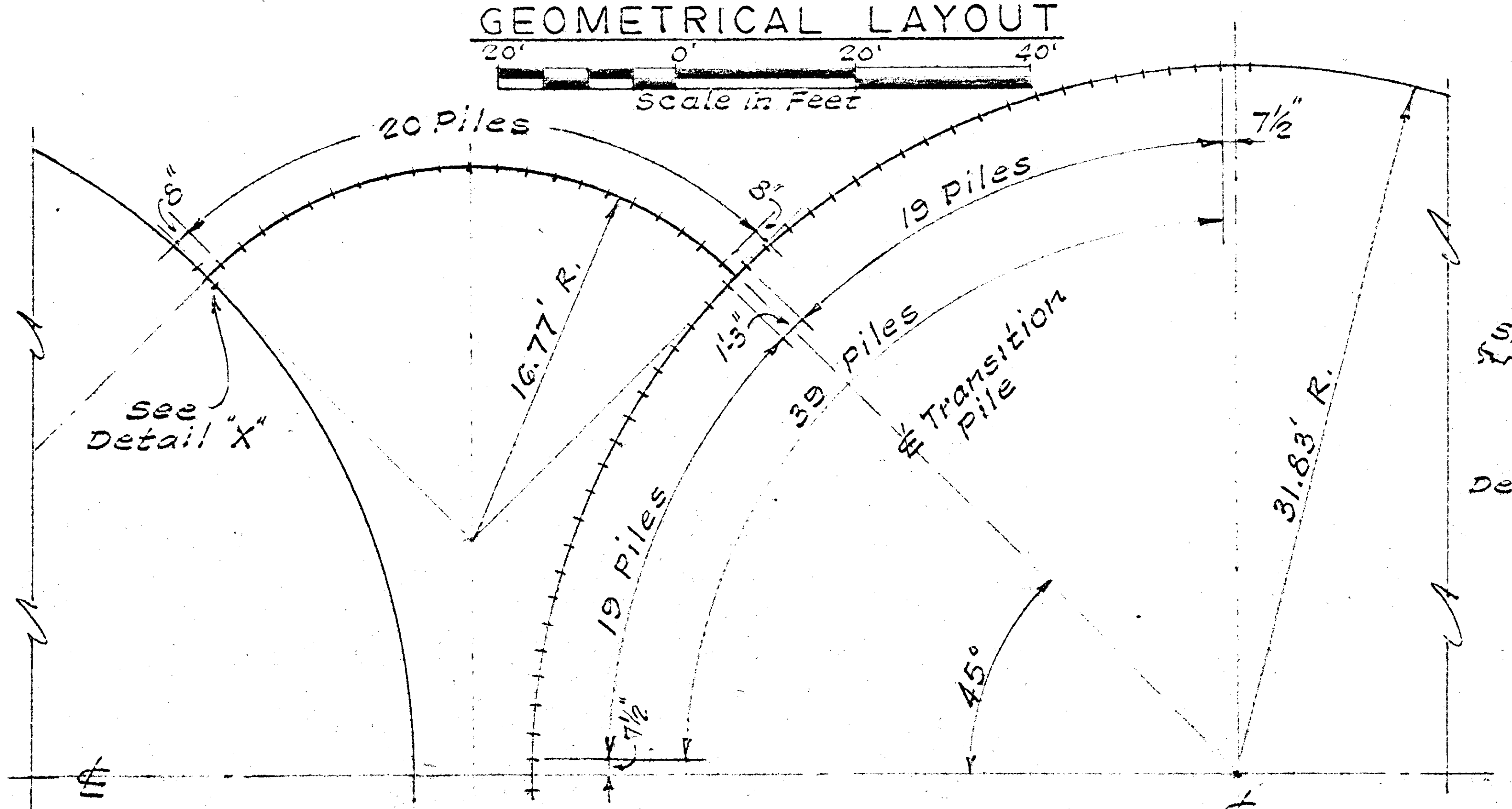


DETAIL - X
TRANSITION SHEET PILE
FOR ARCS



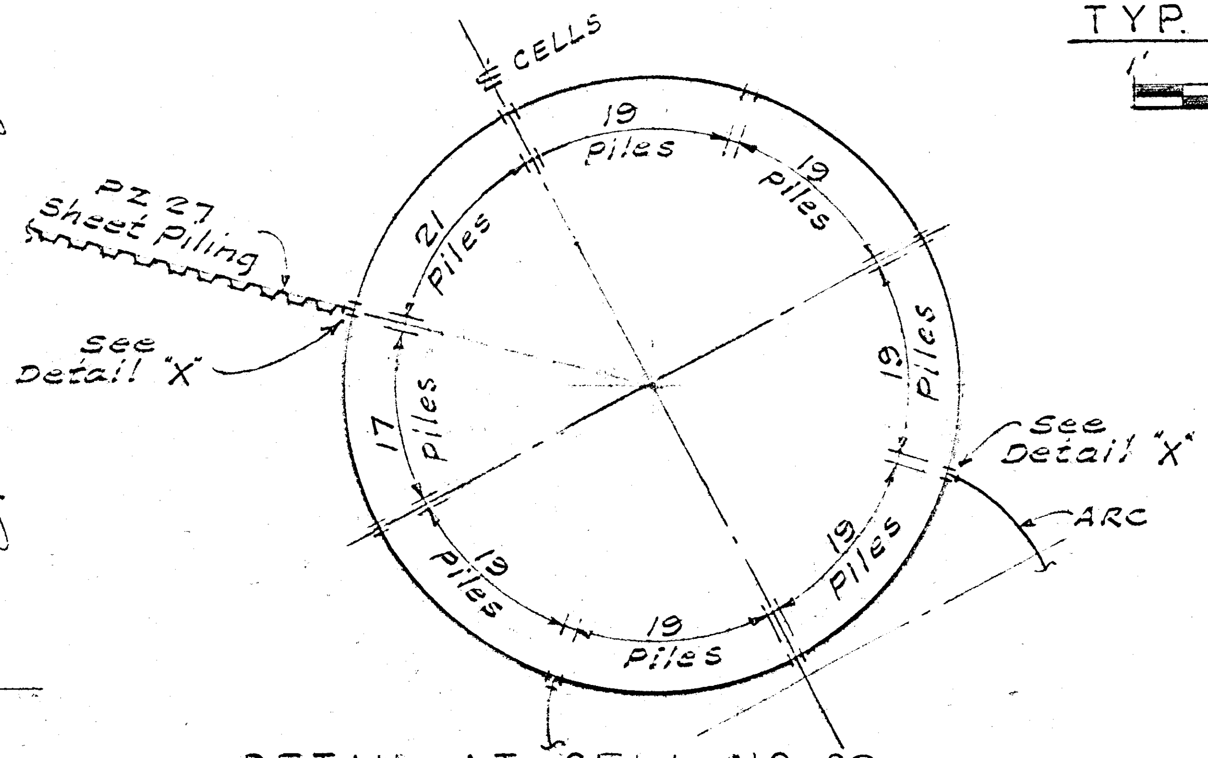
TYP. CELLULAR SECTION

1' 0' 1' 2'
Scale in Feet



TYP. COFFERDAM PILE LAYOUT

5' 0' 5' 10' 15' 20'
Scale in Feet



DETAIL AT CELL NO. 62

FREDERIC R. HARRIS INC.
CONSULTING ENGINEERS - NEW ORLEANS LA

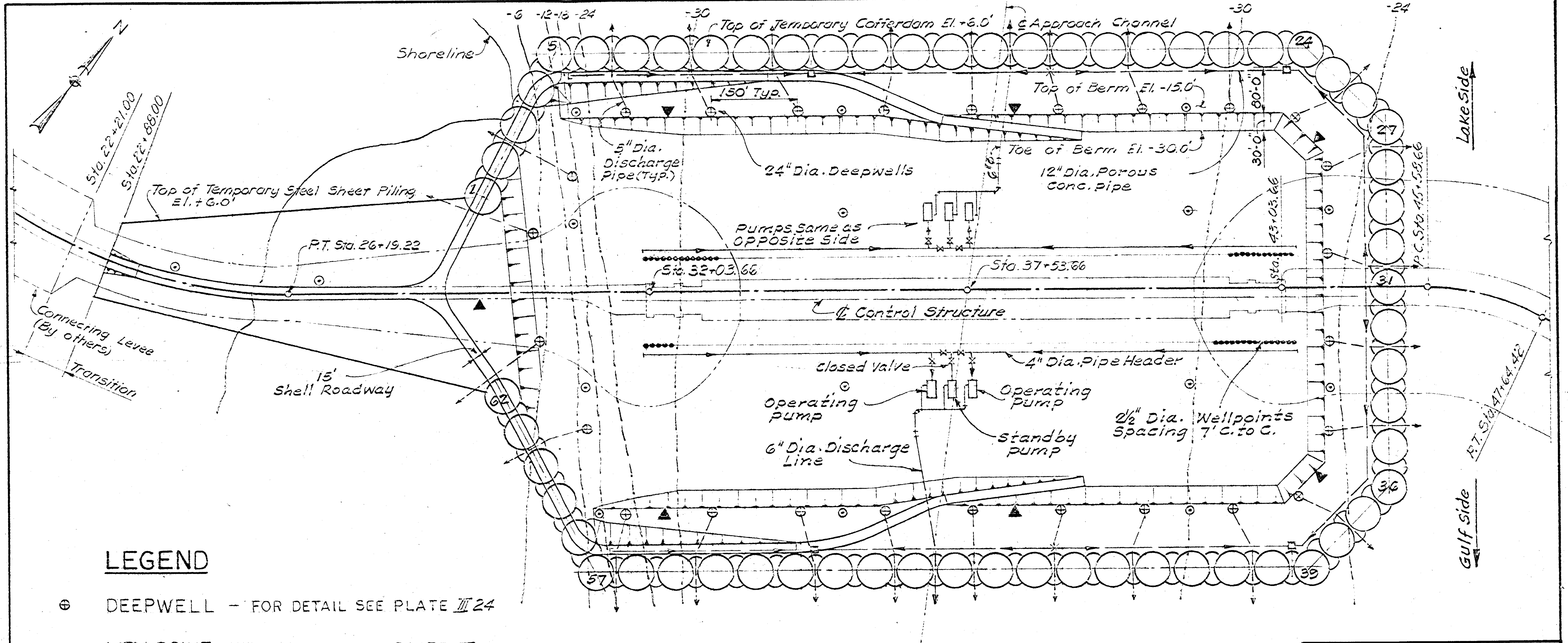
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

COFFERDAM PILE LAYOUT
SHEET PILE DETAILS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III-21



LEGEND

⊕ DEEPWELL - FOR DETAIL SEE PLATE III 24

• WELLPOINT - " " " PLATE III 24

⊙ PIEZOMETER - " " " PLATE III 25

▼ AUTOMATIC WATER LEVEL CONTROL DEVICE
FOR DETAIL SEE PLATE III 25

□ SUMP PIT - (FOR COFFERDAM DRAINAGE)

PLAN

100' 0 100' 200' 300'

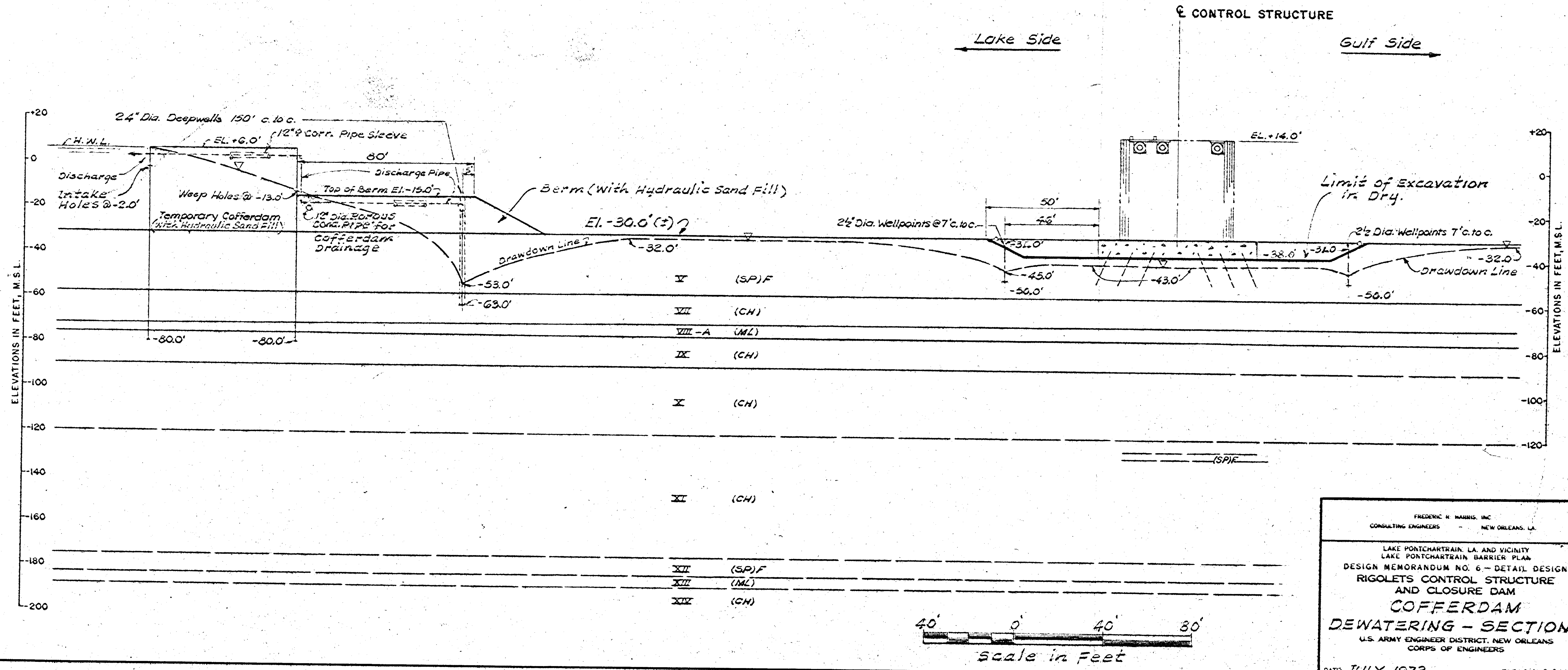
Scale in Feet

FREDERIC R. HARRIS, INC
CONSULTING ENGINEERS - NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
COFFERDAM
DEWATERING — PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III 22



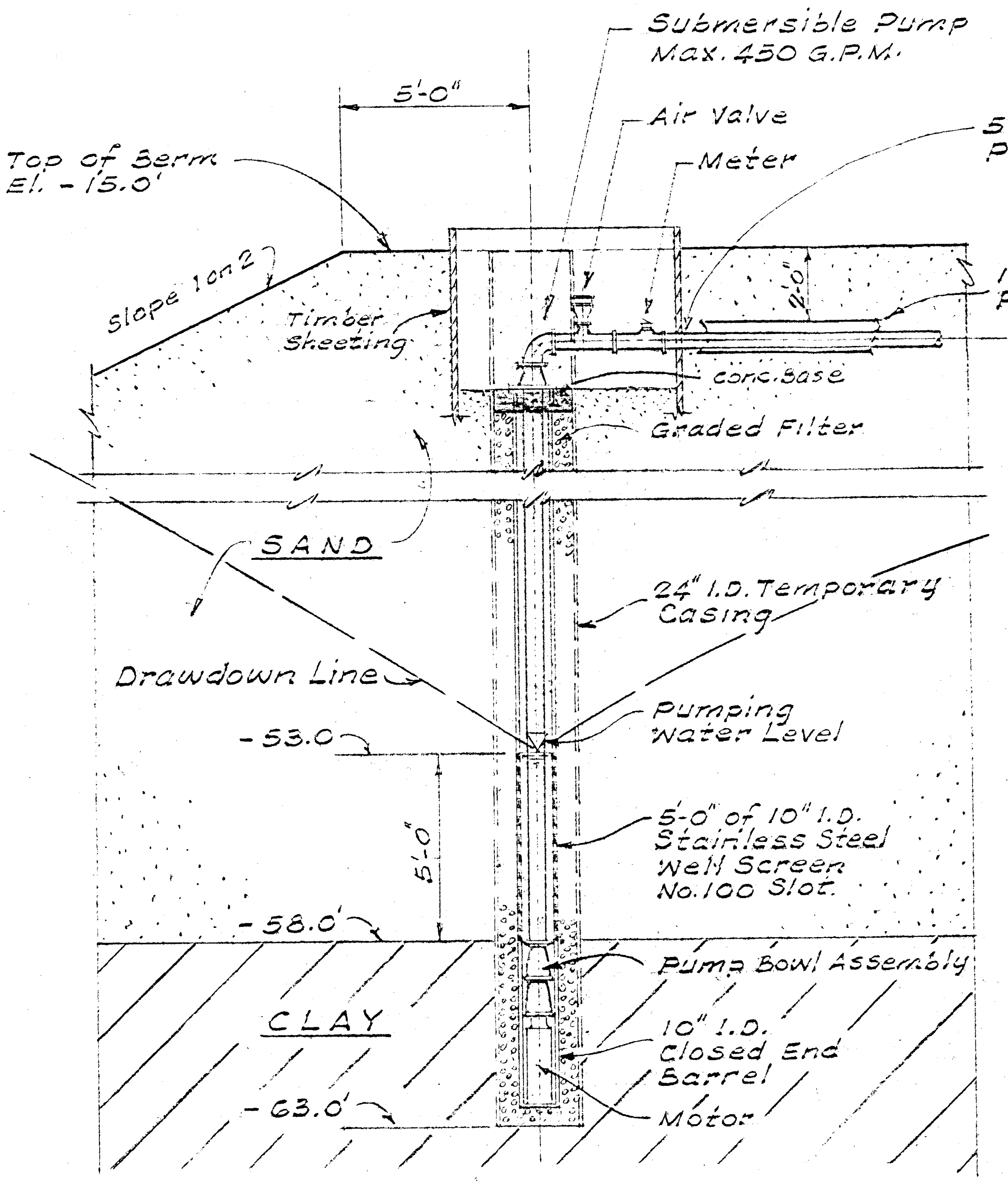
FREDERIC R. HARRIS, INC
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

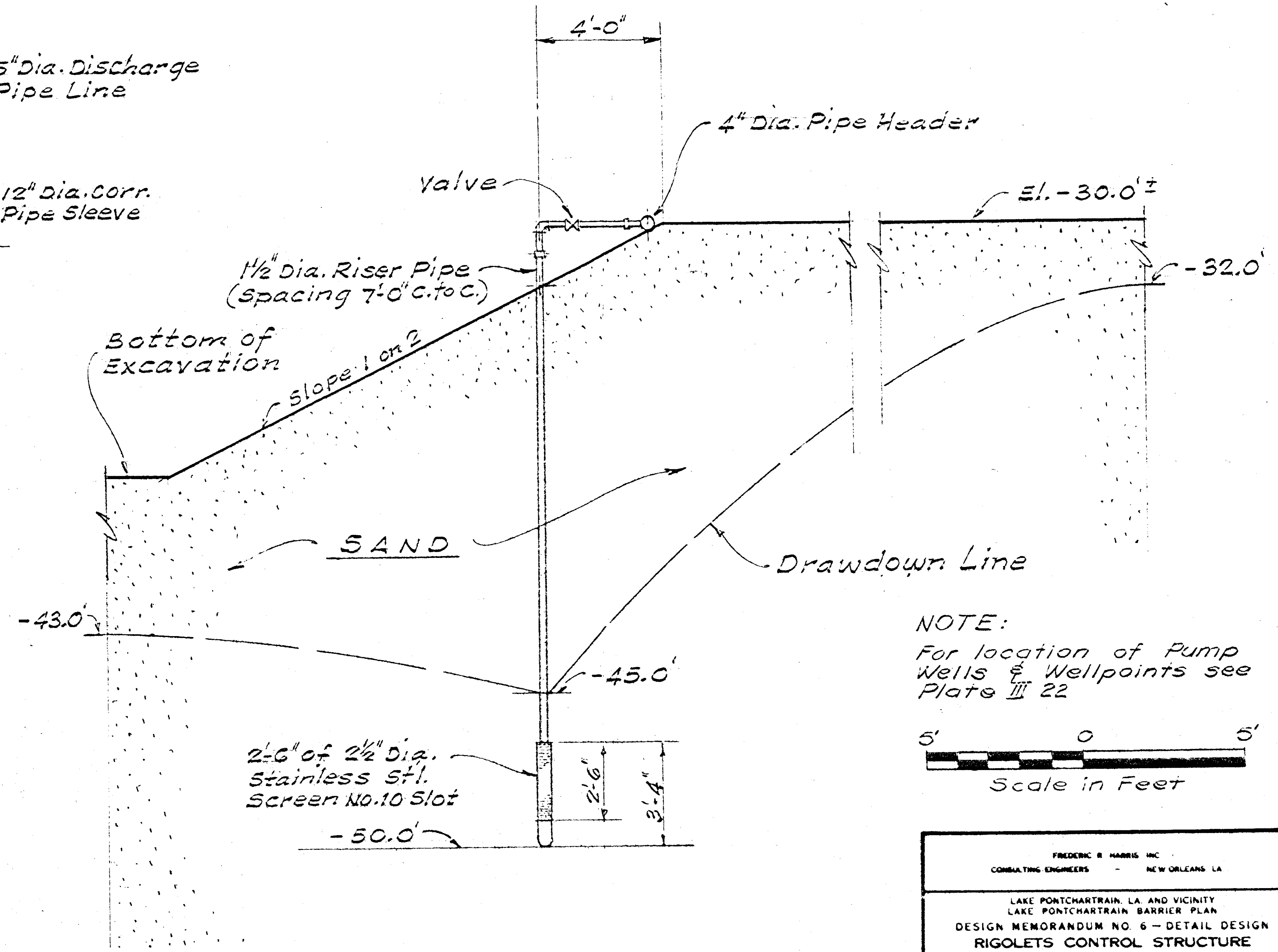
COFFERDAM
DEWATERING — SECTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417



DETAIL OF DEEPWELL



DETAIL OF WELLPOINT

NOTE:
For location of Pump Wells & Wellpoints see Plate III 22

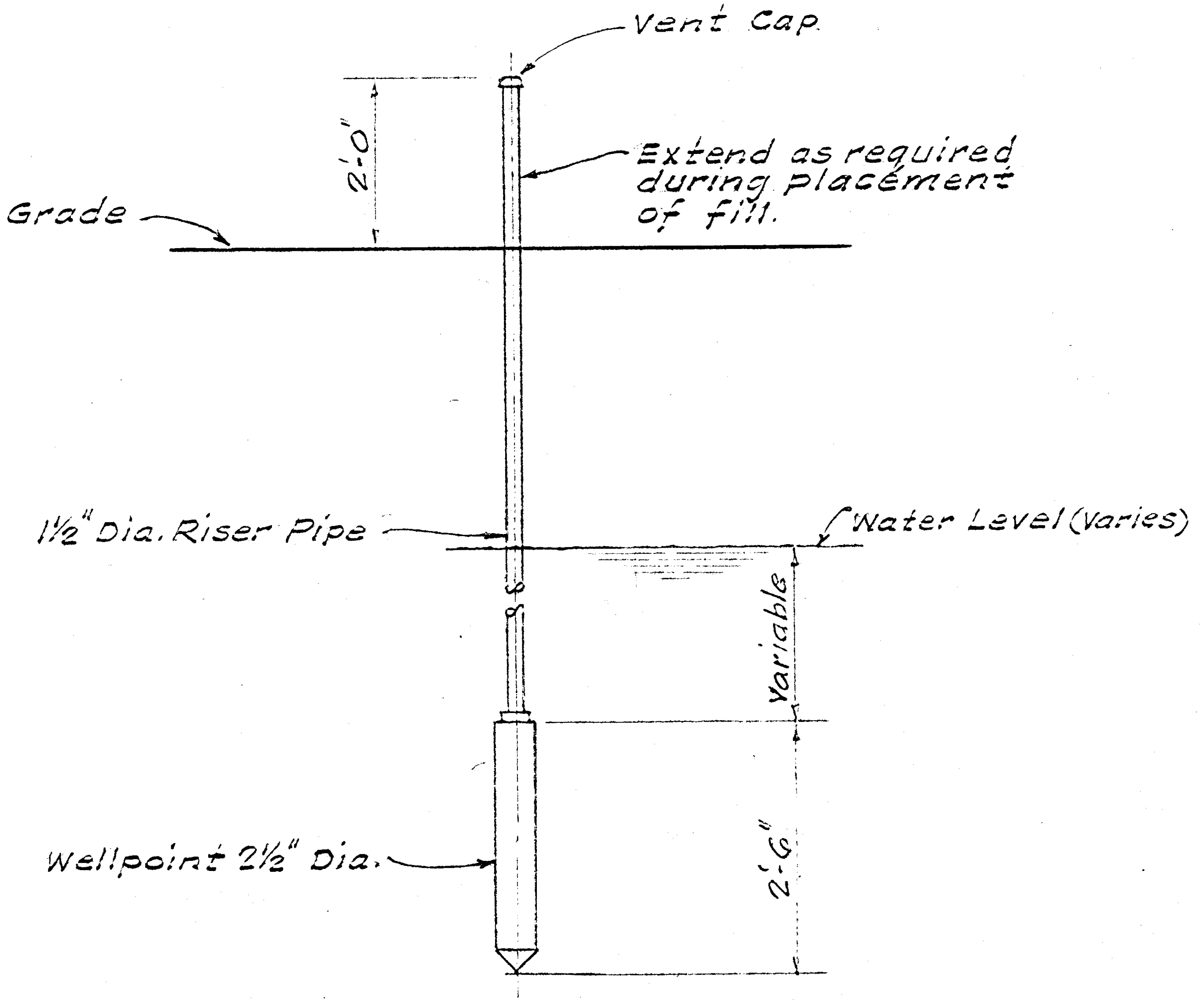
FREDERIC R. HARRIS INC.
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

COFFERDAM
DEEPWELL & WELLPOINT

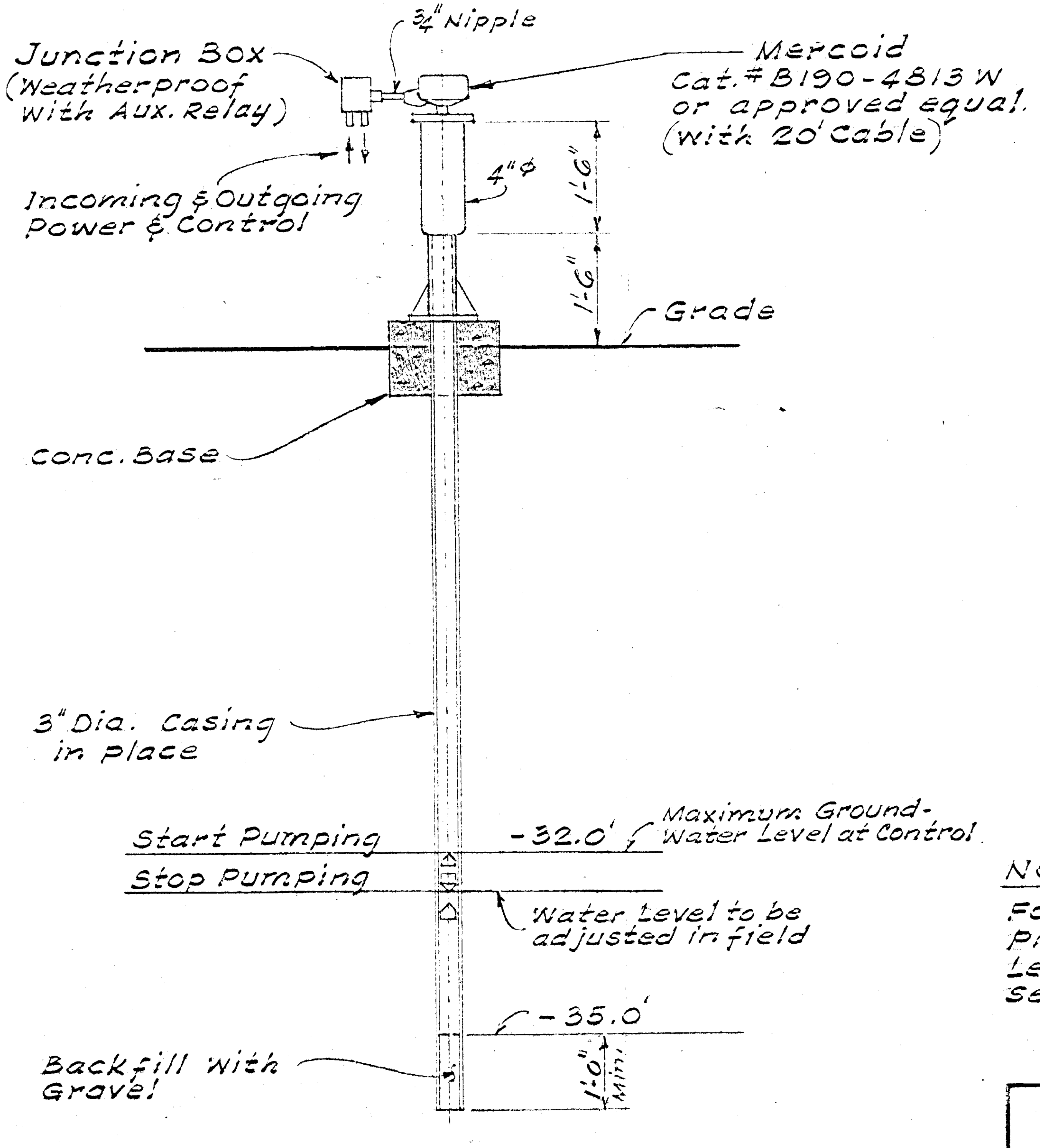
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417



PIEZOMETER INSTALLATION

NOT TO SCALE



AUTOMATIC WATER LEVEL CONTROL DEVICE

NOT TO SCALE

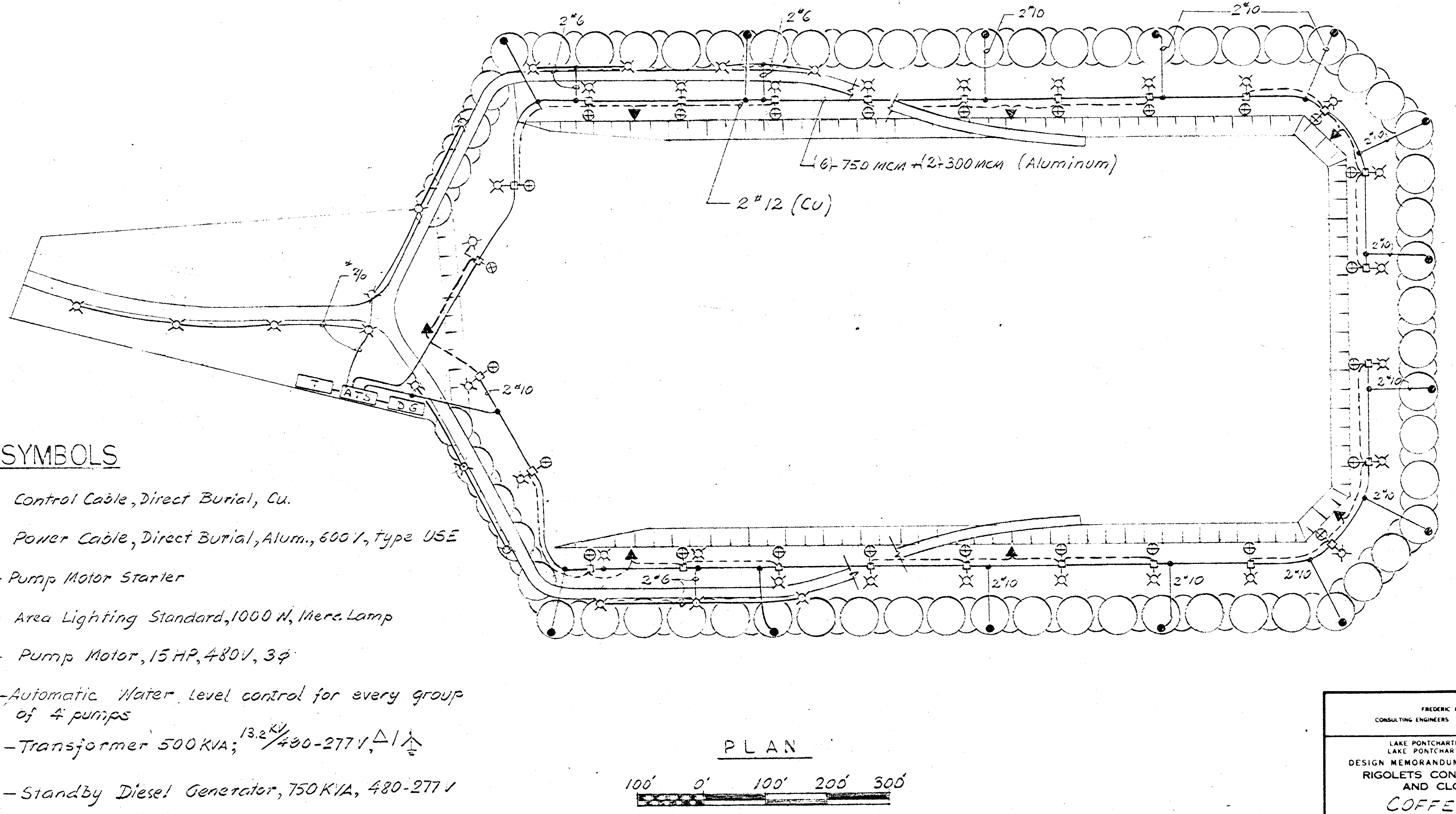
NOTE:
For location of Piezometers & Water Level Control Devices See Plate III 22

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM
COFFERDAM – PIEZOMETER & WATER LEVEL CONTROL DEVICE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

PLATE III 25



SYMBOLS

- - - Control Cable, Direct Burial, Cu.

— Power Cable, Direct Burial, Alum., 600 V, type USE

□ - Pump Motor Starter

Area Lighting Standard, 1000 W, Merc. Lamp

⊗ - Pump Motor, 15 HP, 480V, 3φ

▲ - Automatic Water level control for every group of 4 pumps

T - Transformer 500 KVA; 13.2 KV/480-277 V, Δ/⏚

D.G - Standby Diesel Generator, 750 KVA, 480-277 V

ATS - Automatic Transfer Switch

● - Marine Lights, 300 Watts, Crouse Hi. # PLB 47425 with Transf. 480/120 V

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

COFFERDAM
POWER DISTRIBUTION SYSTEM

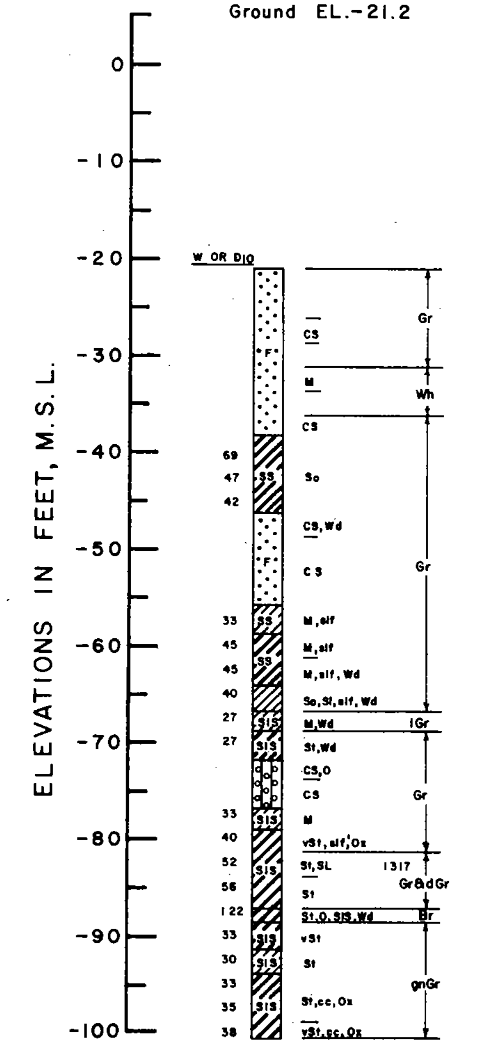
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417

PLATE III 26

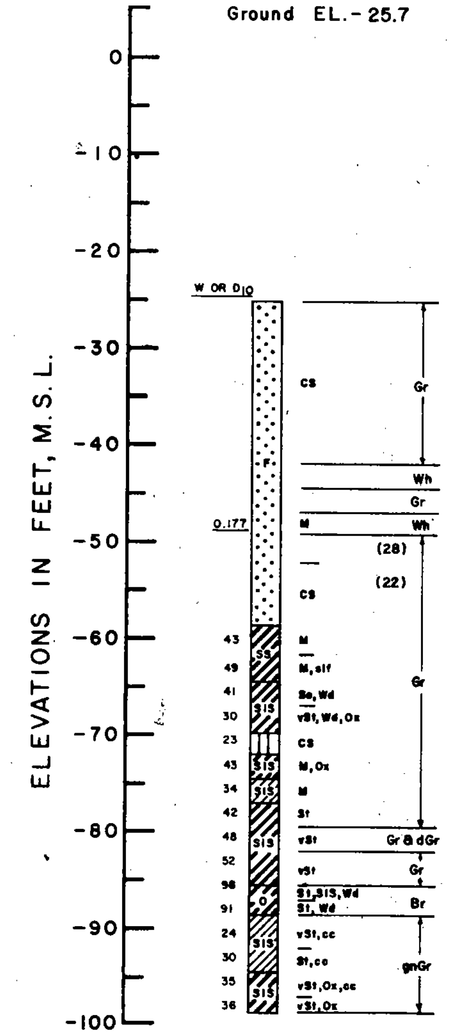
BORING NO. X-2

STA. 53+00
14 - 15 APRIL 1971
150 FT. RIGHT OF B.L.
Ground EL.-21.2



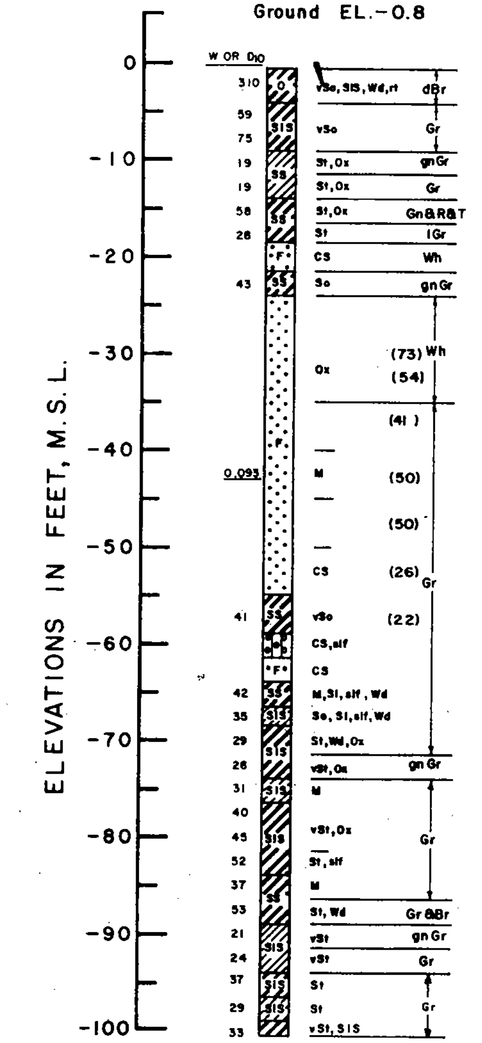
BORING NO. X-4

STA. 65+00
7 - 8 APRIL 1971
400 FT. RIGHT OF B.L.
Ground EL.-25.7



BORING NO. X-6

STA. 77+00
1 APRIL 1971
340 FT. RIGHT OF B.L.
Ground EL.-0.8



NOTES

1. FOR BORING LOCATION SEE PLATE III-1
2. FOR SOIL BORING LEGEND SEE PLATE A

FREDERIC R. HARRIS INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA. AND VICINITY,
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

GENERAL SOIL BORINGS
NO. X-2, X-4 AND X-6

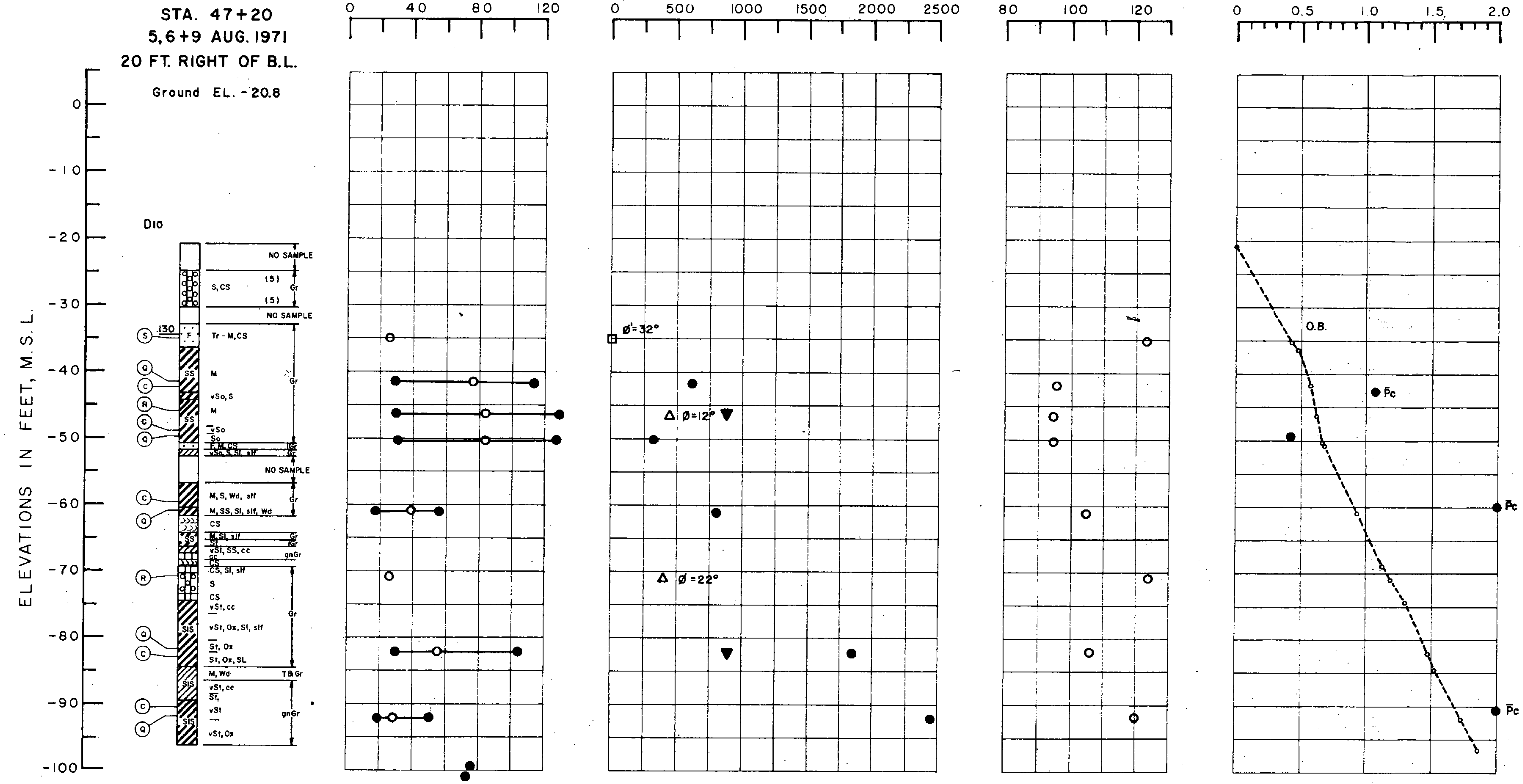
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE *JULY 1972* FILE NO. H-2-24417

PLATE A-1

BORING NO. X-1U

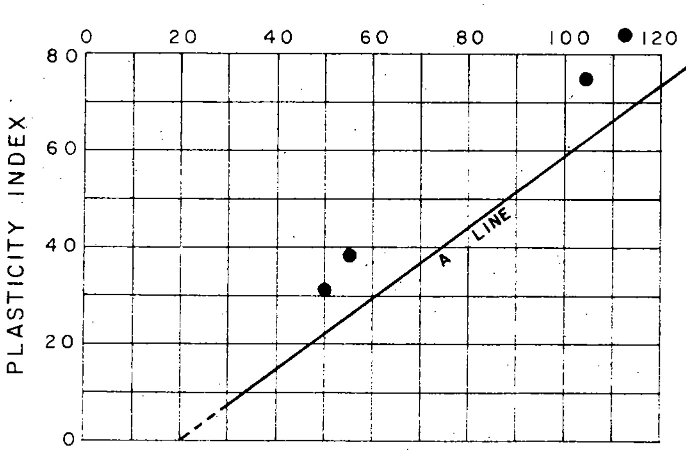
STA. 47+20
5,6+9 AUG. 1971
20 FT. RIGHT OF B.L.
Ground EL. -20.8



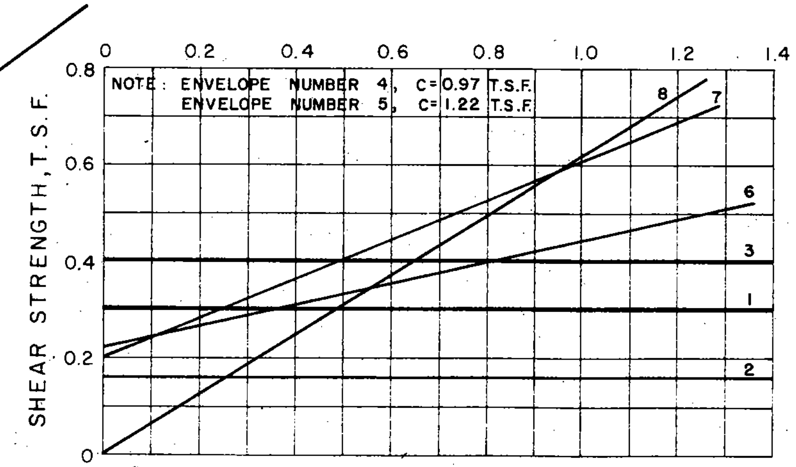
GENERAL NOTES

▼ UC—Unconfined compression shear
● Ⓠ Unconsolidated undrained triaxial shear
△ Ⓡ Consolidated undrained triaxial shear
⊡ Ⓢ Consolidated drained direct shear
Ⓒ Consolidatation test
-○- W—Natural water content
-● L.L.—Liquid limit
●- P.L.—Plastic limit
c—Unit cohesion
ø—Angle of friction
○ γ—Unit weight of soil-wet density
σ̄—Normal stress
O.B.—Overburden
●P̄c—Preconsolidation pressure
e—Void ratio
Cc—Compression index

For boring legend see plate A
For boring location see plate III-1
For consolidation test results see plates A-18 thru A-29



PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F	CLASS
1	-41.7	Q-●	0	0.30	CH
2	-50.1		0	0.16	CH
3	-61.0		0	0.40	CH
4	-82.1		0	0.97	CH
5	-92.0		0	1.22	CH
6	-46.3	R-△	12	0.22	CH
7	-71.0		22	0.20	SM
8	-35.1	S-□	32	0	SM

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

UNDISTURBED SOIL BORING NO. X-1U, SOIL TEST DATA

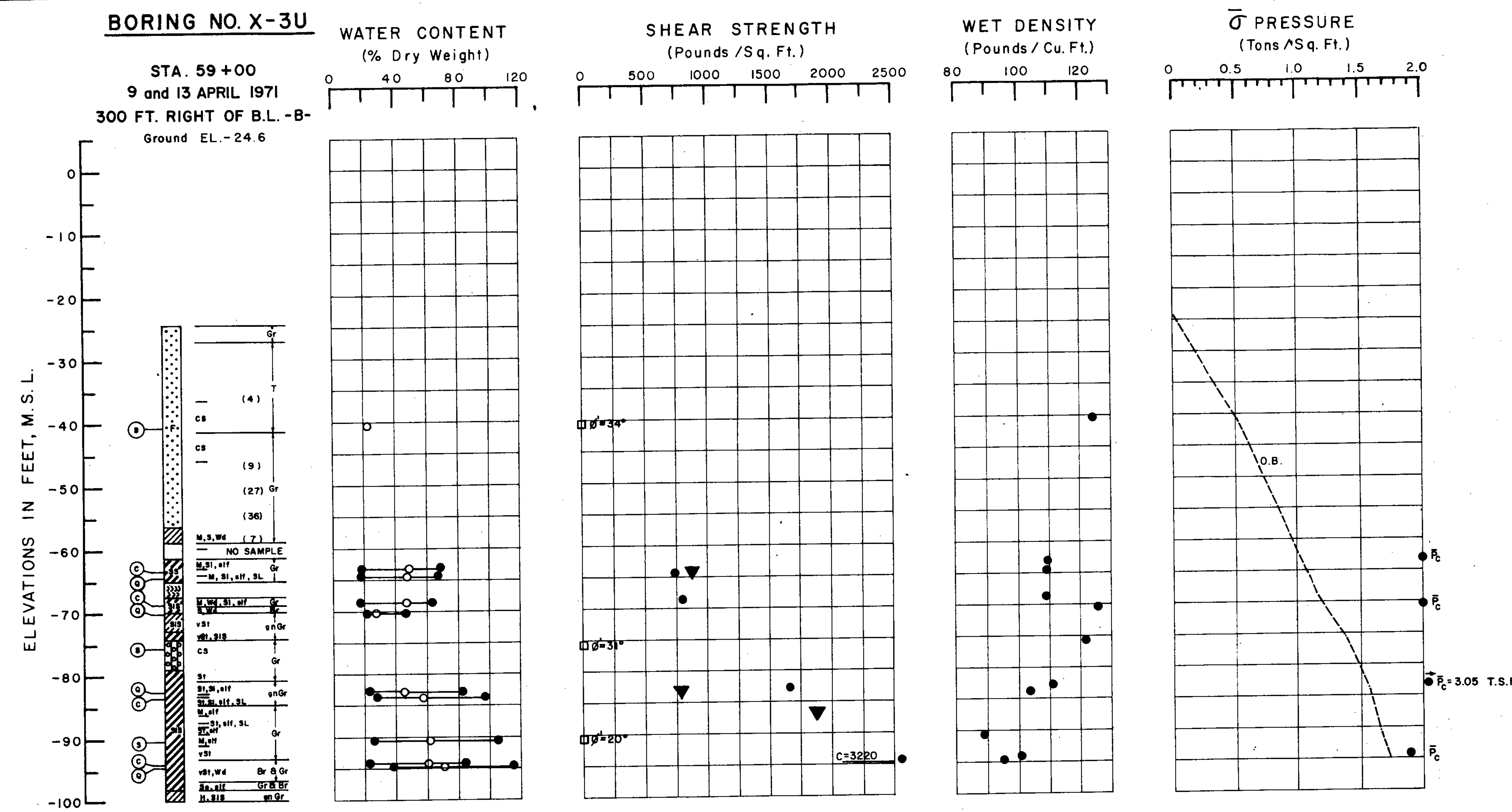
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

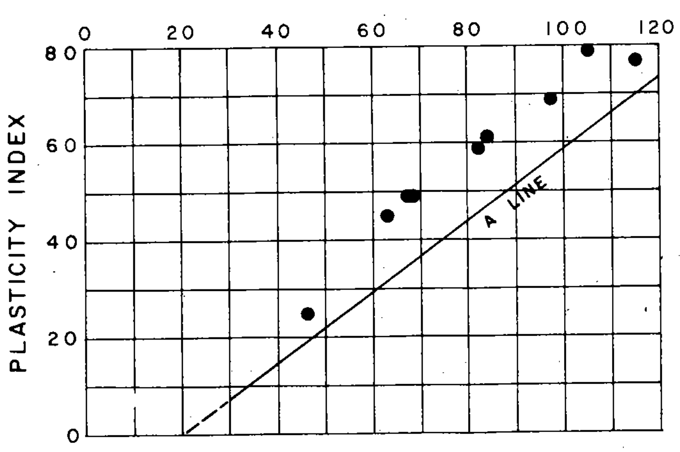
DATE JULY 1972 FILE NO. H-2-24417

PLATE **A-2**

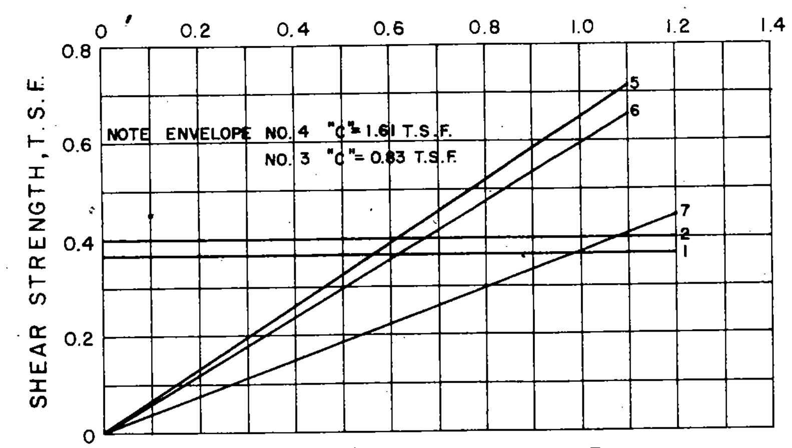
BORING NO. X-3U

STA. 59+00
9 and 13 APRIL 1971
300 FT. RIGHT OF B.L. -B-
Ground EL. -24.6





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F.	CLASS
1	-64.8	Q-●	0	0.37	CH
2	-68.7		0	0.40	CH
3	-82.8		0	0.83	CH
4	-94.9		0	1.61	CH
5	-40.8	S-□	34°	0	SM
6	-76.0		31°	0	SM
7	-90.9		20°	0	CH

For General Notes See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS — NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X-3U, SOIL TEST DATA

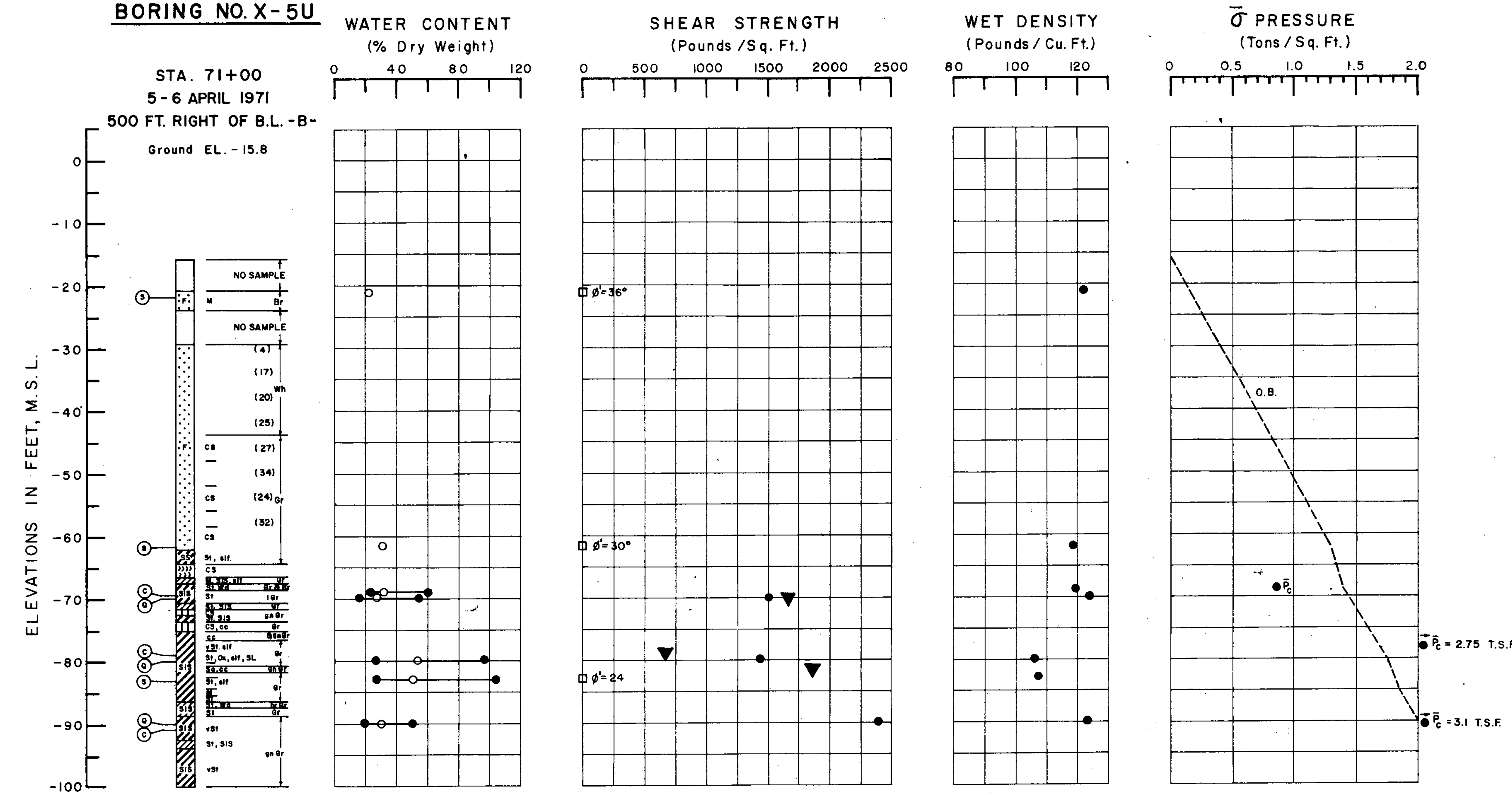
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

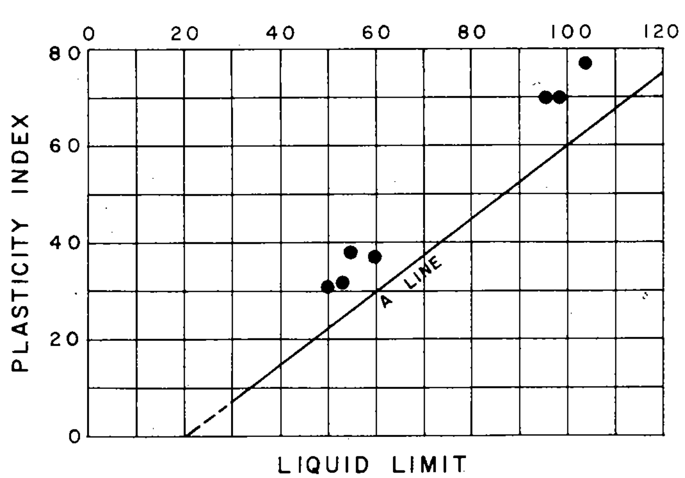
DATE JULY 1972 FILE NO. H-2-24417

PLATE A-3

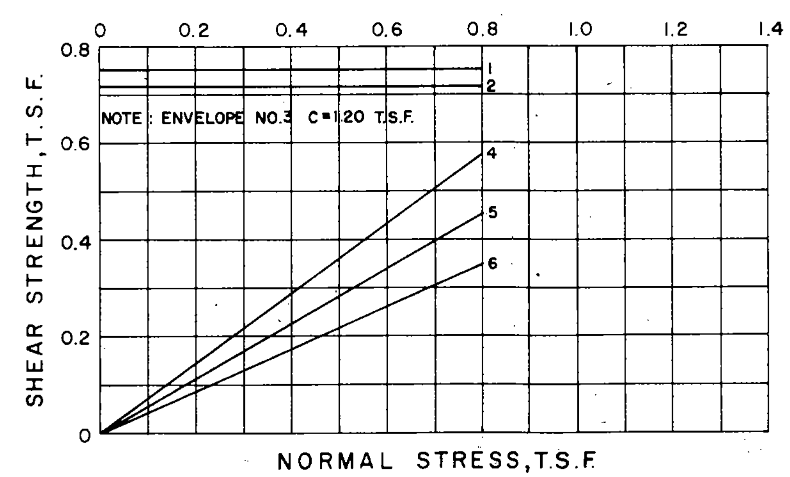
BORING NO. X-5U

STA. 71+00
5-6 APRIL 1971
500 FT. RIGHT OF B.L. -B-





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
NO.	EL.		ø°	C,T.S.F.	
1	-69.6		0	0.75	CH
2	-79.7	Q-●	0	0.72	CH
3	-89.8		0	1.20	CH
4	-21.9		36°	0	SP
5	-61.9	S-□	30°	0	SM
6	- 83.2		24	0	CH

For General Notes
See Plate A-2

FREDERIC R HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

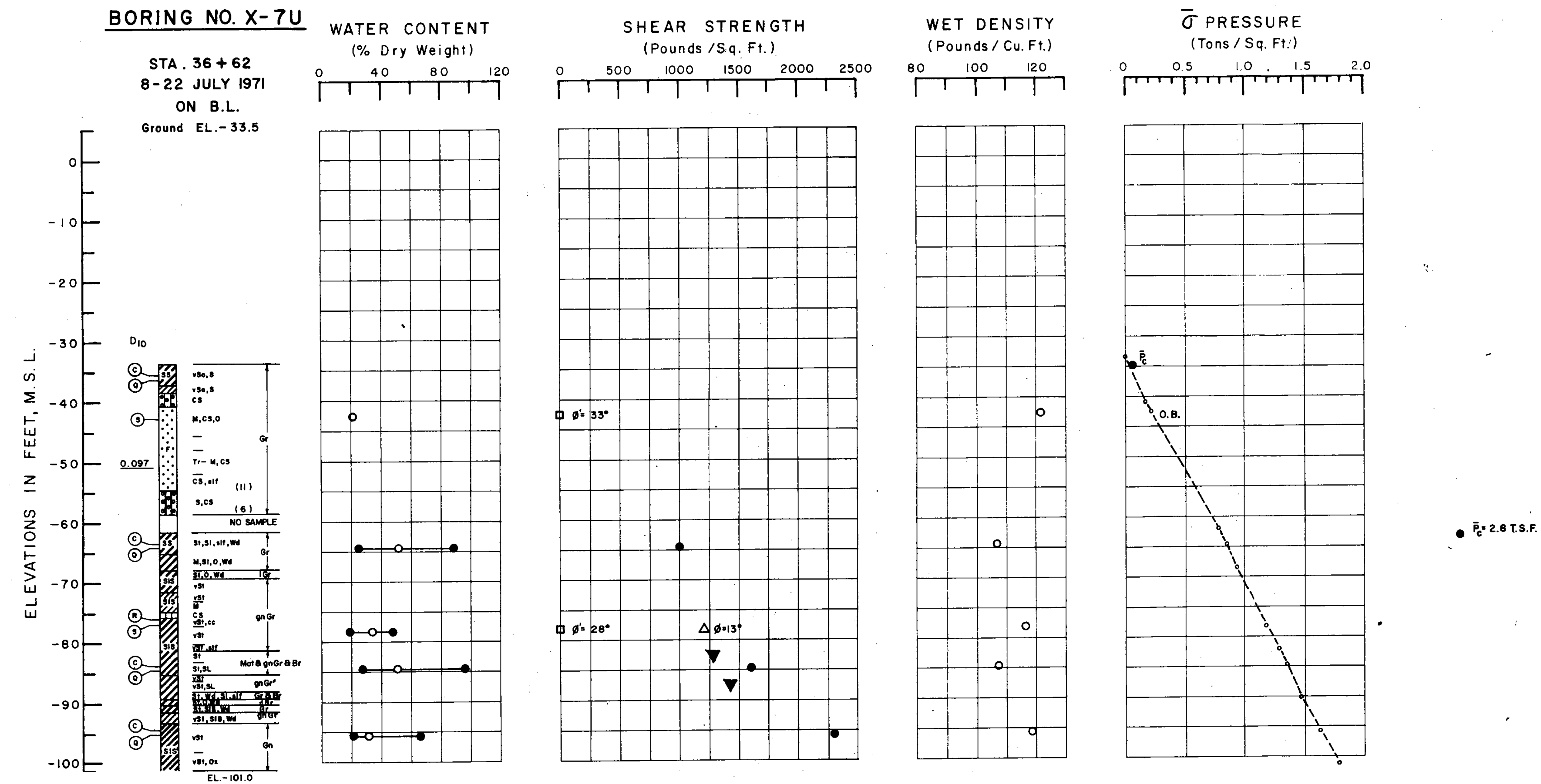
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

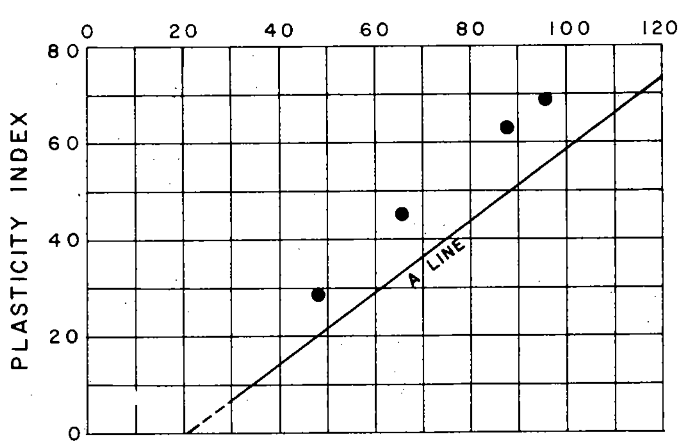
UNDISTURBED SOIL BORING
NO. X- 5U, SOIL TEST DATA

U S ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

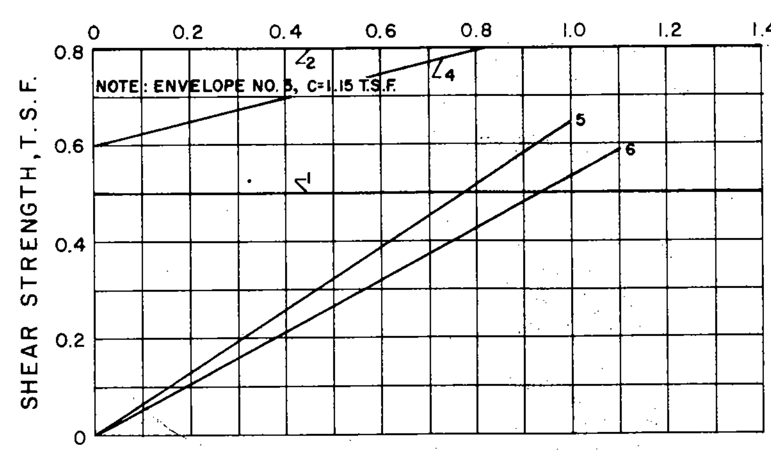
DAT. JULY 1972 FILE NO. H-2-24417

PLATE A-4





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
NO.	EL.		Ø°	C,T.S.F.	
1	- 64.7	Q-●	0	0.50	CH
2	- 84.7		0	0.80	CH
3	- 95.5		0	1.15	CH
4	- 78.1	R-△	13	0.60	CH-ML
5	- 42.7	S-□	33	0	SP
6	- 78.1		28	0	CH-ML

For General Notes
See Plate A-2

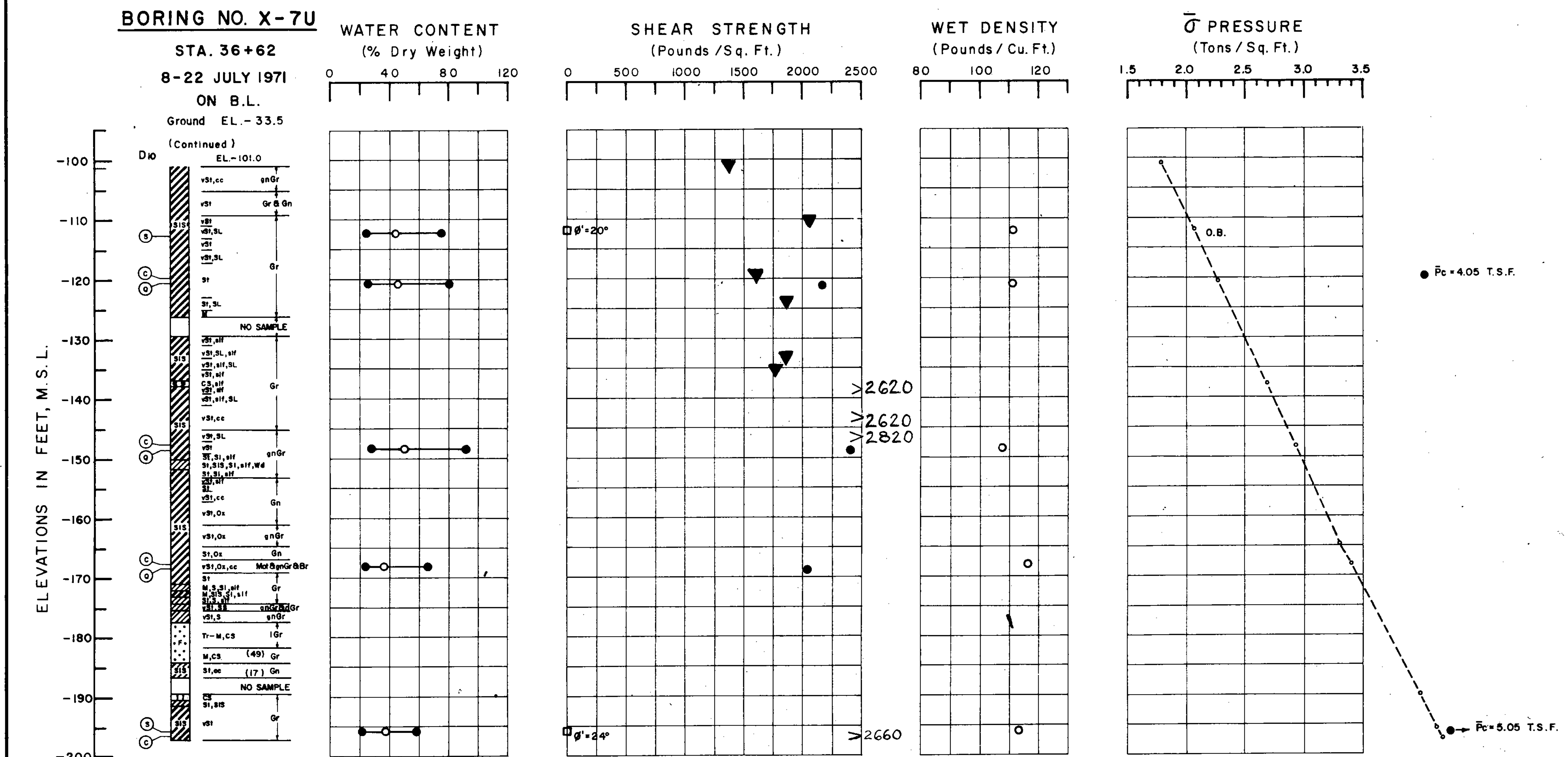
FREDERIC R. HARRIS, INC
CONSULTING ENGINEERS - NEW ORLEANS LA

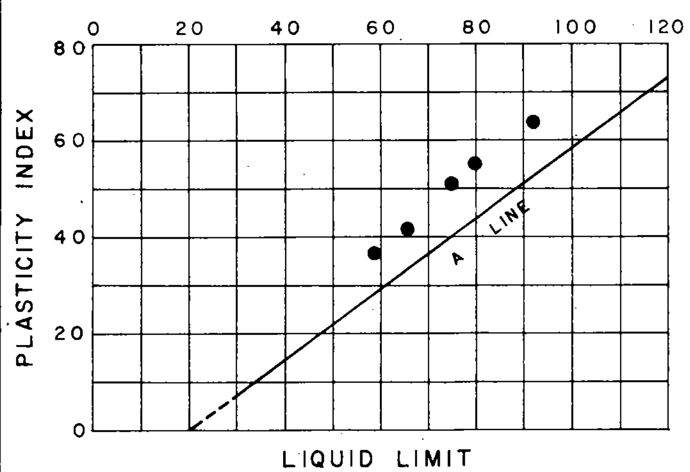
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X- 7U, SOIL TEST DATA

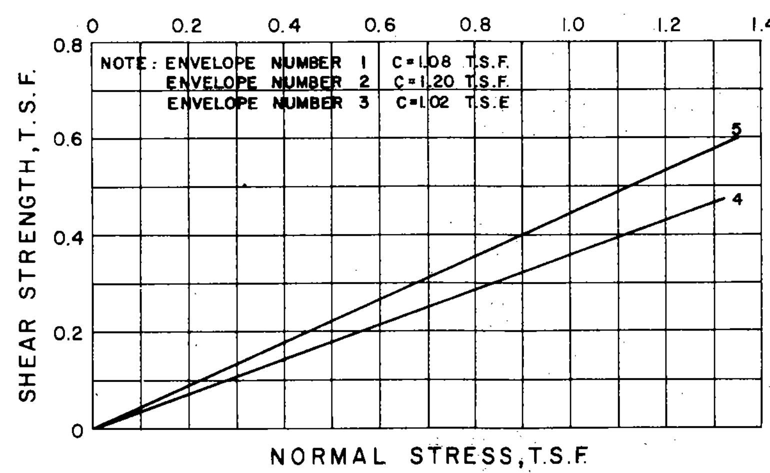
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH Ø°	C, T.S.F.	CLASS
1	-120.7	Q-●	0	1.08	CH
2	-148.7		0	1.20	CH
3	-168.5		0	1.02	CH
4	-112.2	S-□	20	0	CH
5	-195.4		24	0	

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

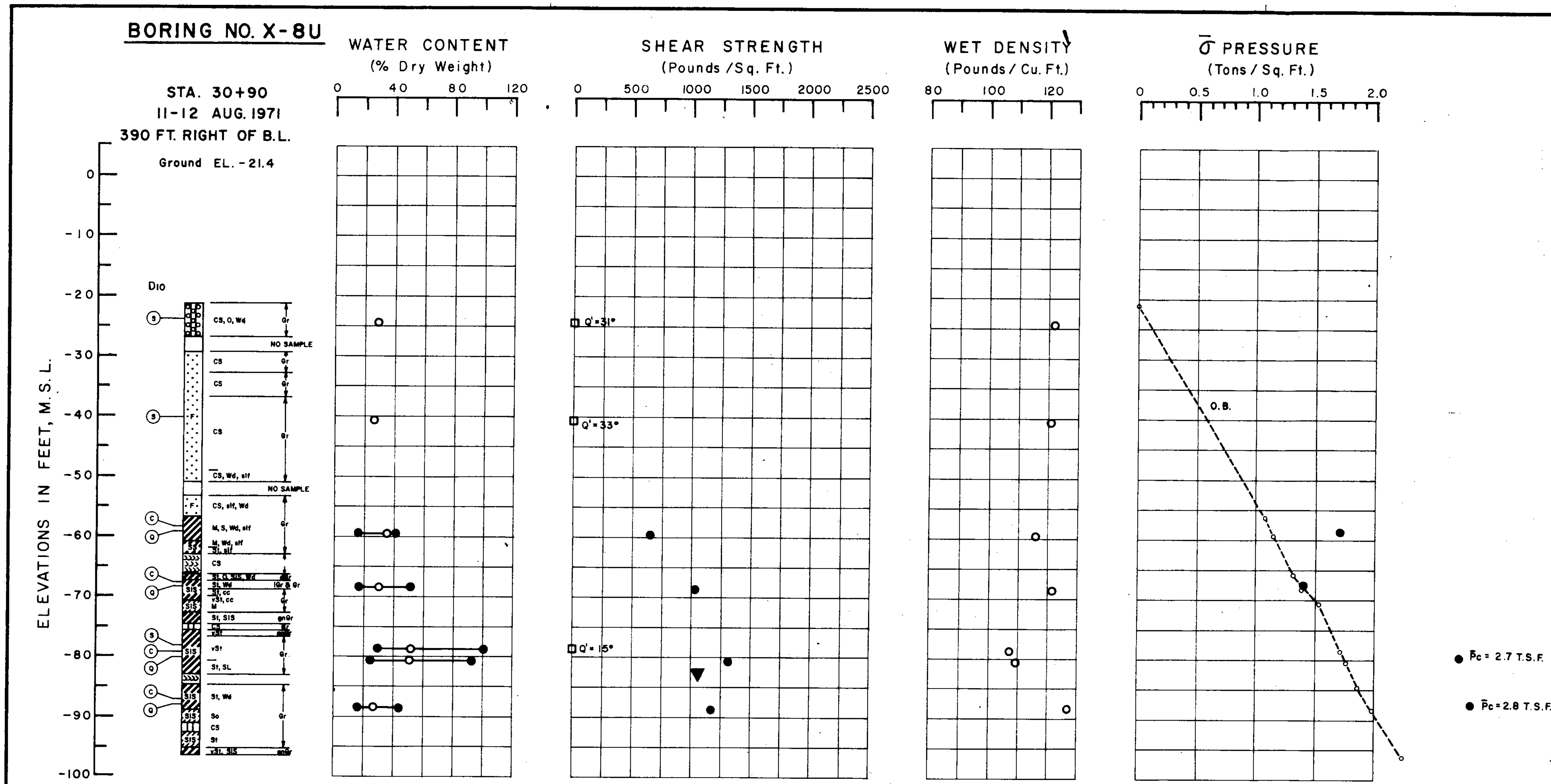
LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

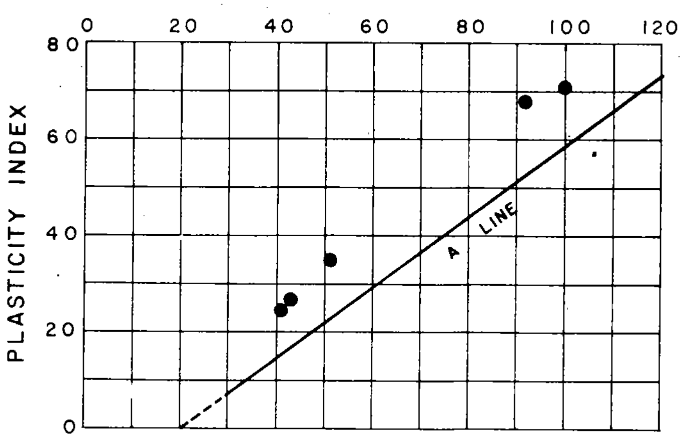
UNDISTURBED SOIL BORING
NO. X-7U, SOIL TEST DATA

U S ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

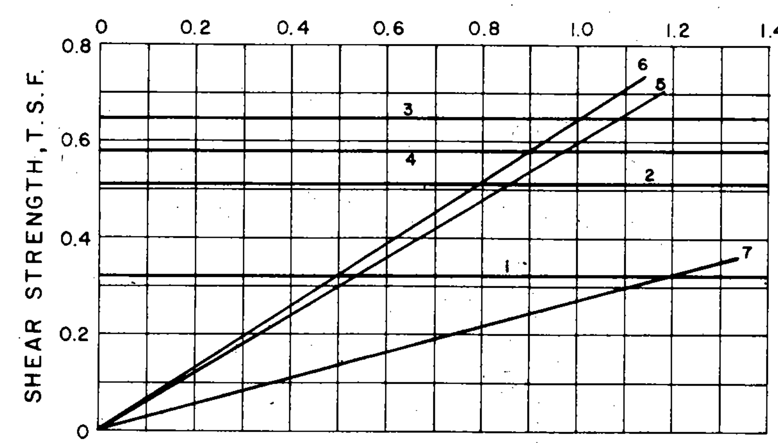
DATE JULY 1972 FILE NO. H-2-24417

PLATE A-6





LIQUID LIMIT
PLASTICITY CHART



NORMAL STRESS, T.S.F.
SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
NO.	EL.		Ø°	C,T.S.F.	
1	- 59.6	Q-●	0	0.32	CL
2	- 68.5		0	0.51	CL
3	- 80.5		0	0.65	CH
4	- 88.5		0	0.58	CL
5	- 24.3	S-□	31	0	SM
6	- 40.3		33	0	SM
7	- 78.7		15	0	CH

For General Notes
See Plate A-2

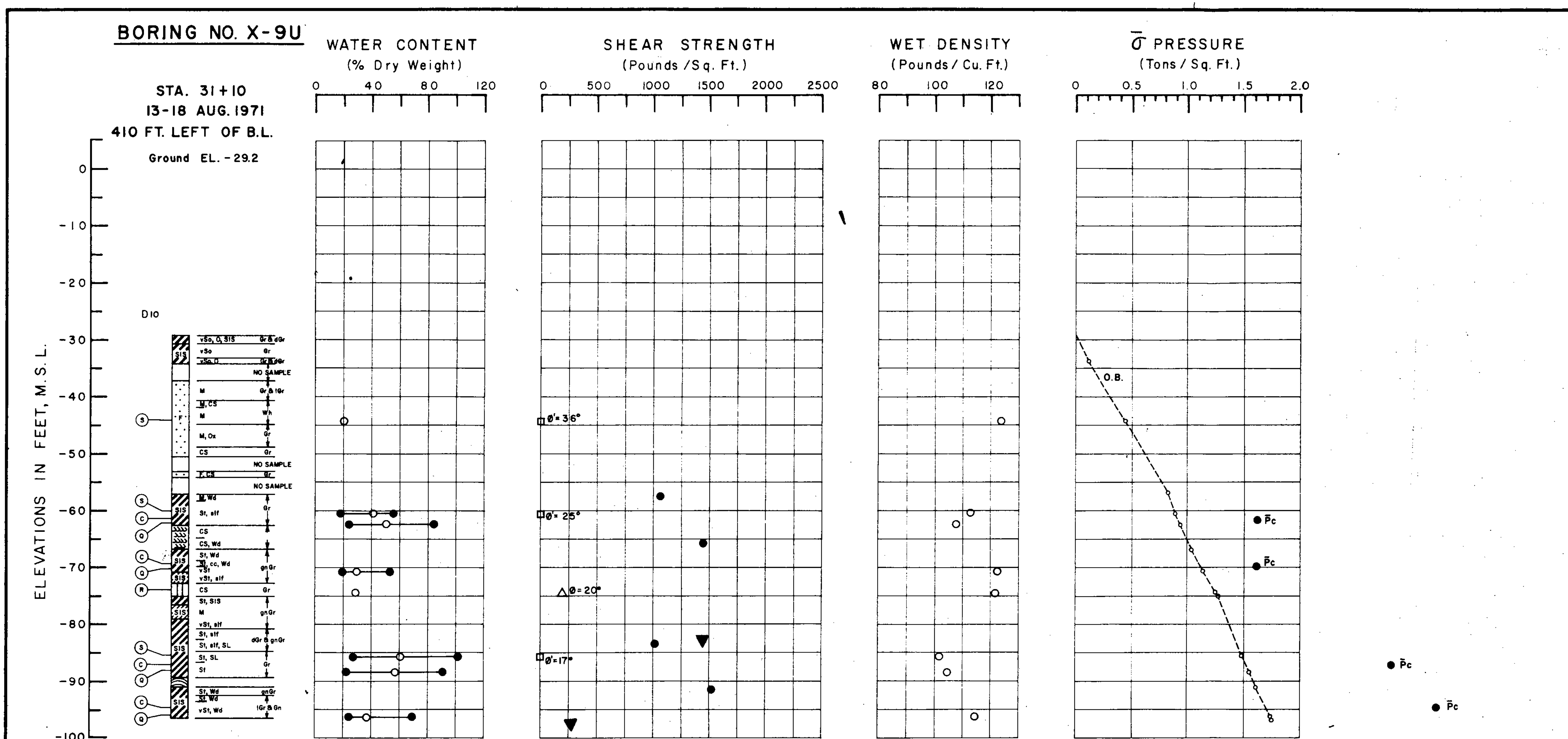
FREDERIC R. HARRIS INC.
CONSULTING ENGINEERS - NEW ORLEANS LA

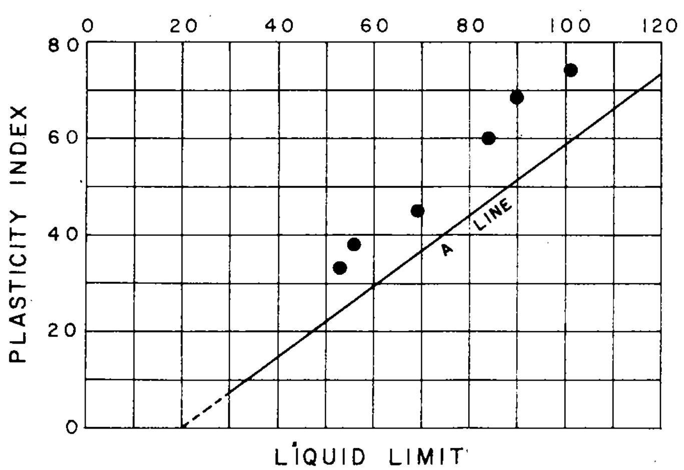
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X- 8U, SOIL TEST DATA

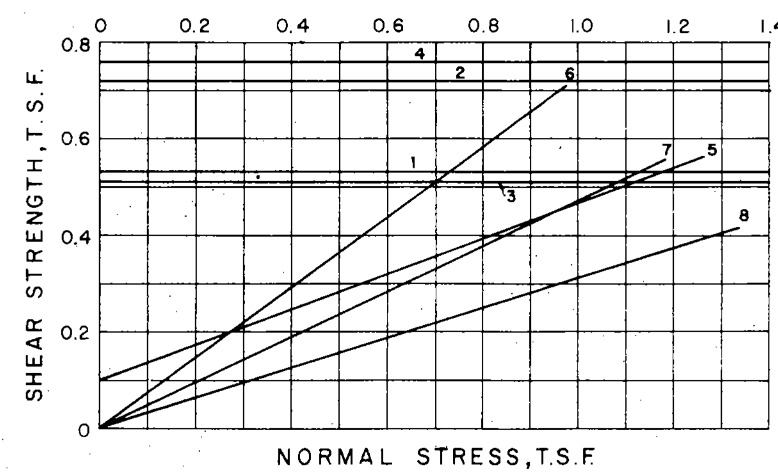
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH Ø°	STRENGTH C,T.S.F.	CLASS
1	-62.5	Q-●	0	0.53	CH
2	-70.6		0	0.72	CH
3	-88.4		0	0.51	CH
4	-96.2		0	0.76	CH
5	-74.3	R-△	20	0.10	SM
6	-44.4	S-□	36	0	SP
7	-60.5		25	0	CH
8	-85.7		17	0	CH

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS, LA

LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

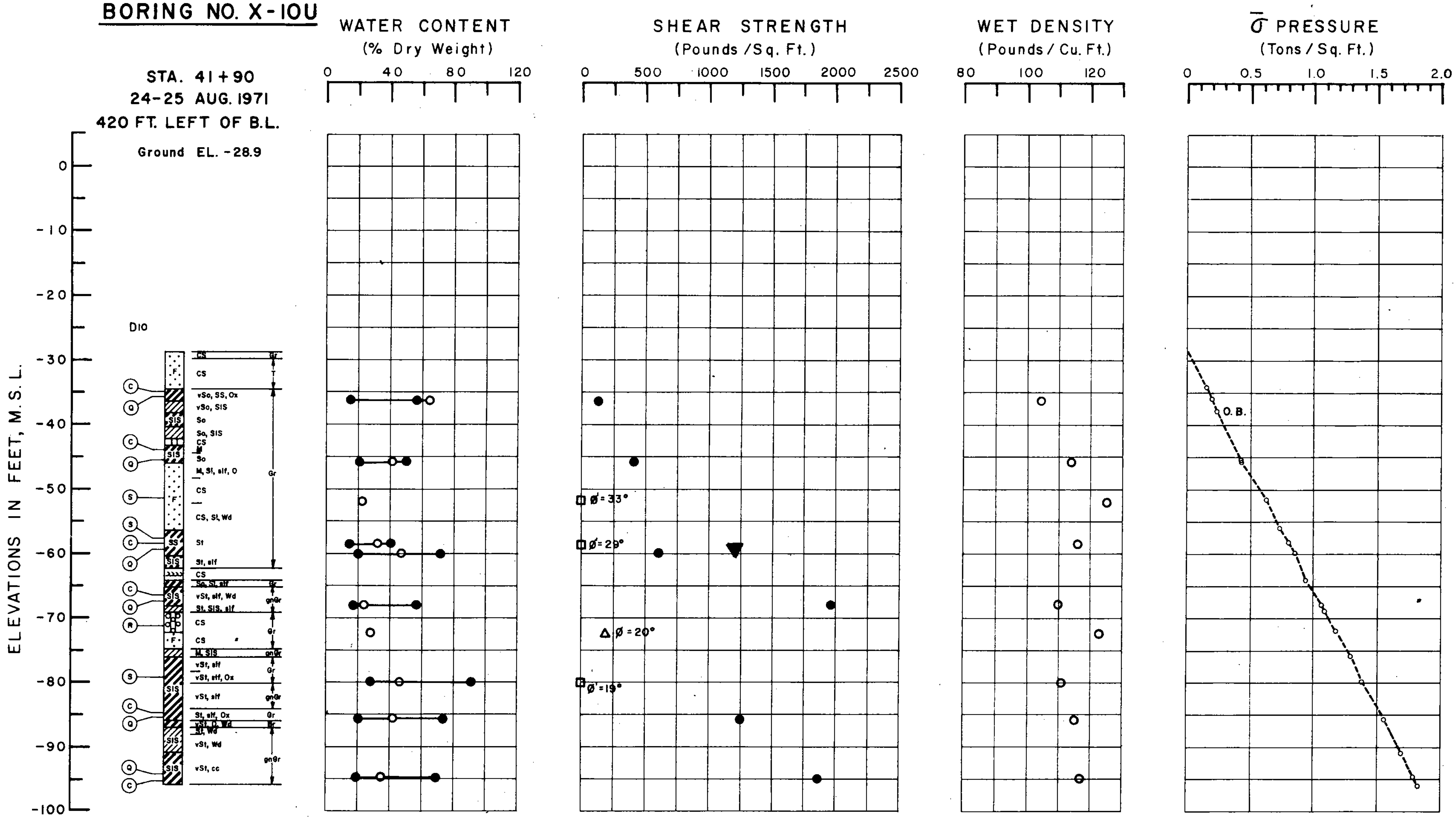
UNDISTURBED SOIL BORING
NO. X-9U, SOIL TEST DATA

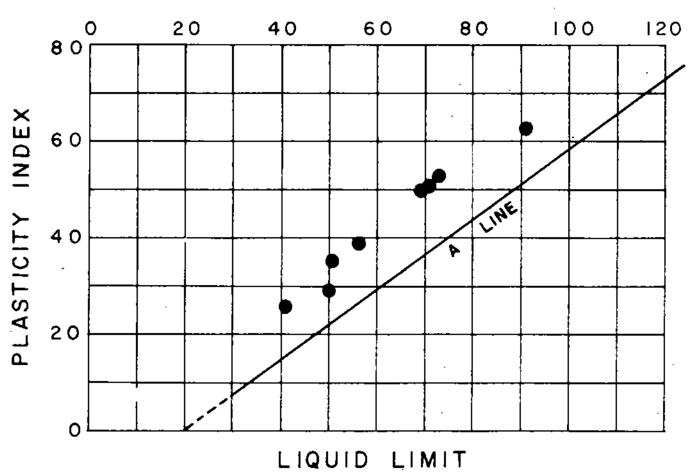
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

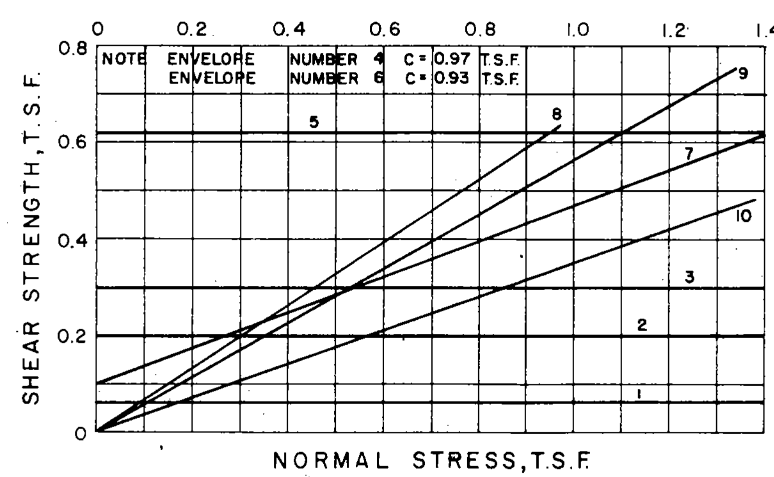
BORING NO. X-10U

STA. 41+90
24-25 AUG. 1971
420 FT. LEFT OF B.L.
Ground EL. -28.9





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
NO.	EL.		$\phi°$	C, T.S.F.	
1	-36.1	Q-●	0	0.06	CH
2	-45.7	Q-●	0	0.20	CH
3	-60.0	Q-●	0	0.30	CH
4	-68.0	Q-●	0	0.97	CH
5	-85.8	Q-●	0	0.62	CH
6	-94.9	Q-●	0	0.93	CH
7	-72.0	R-△	20	0.10	SM
8	-51.9	S-□	33	0	SP
9	-58.2	S-□	29	0	CL
10	-80.0	S-□	19	0	CH

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

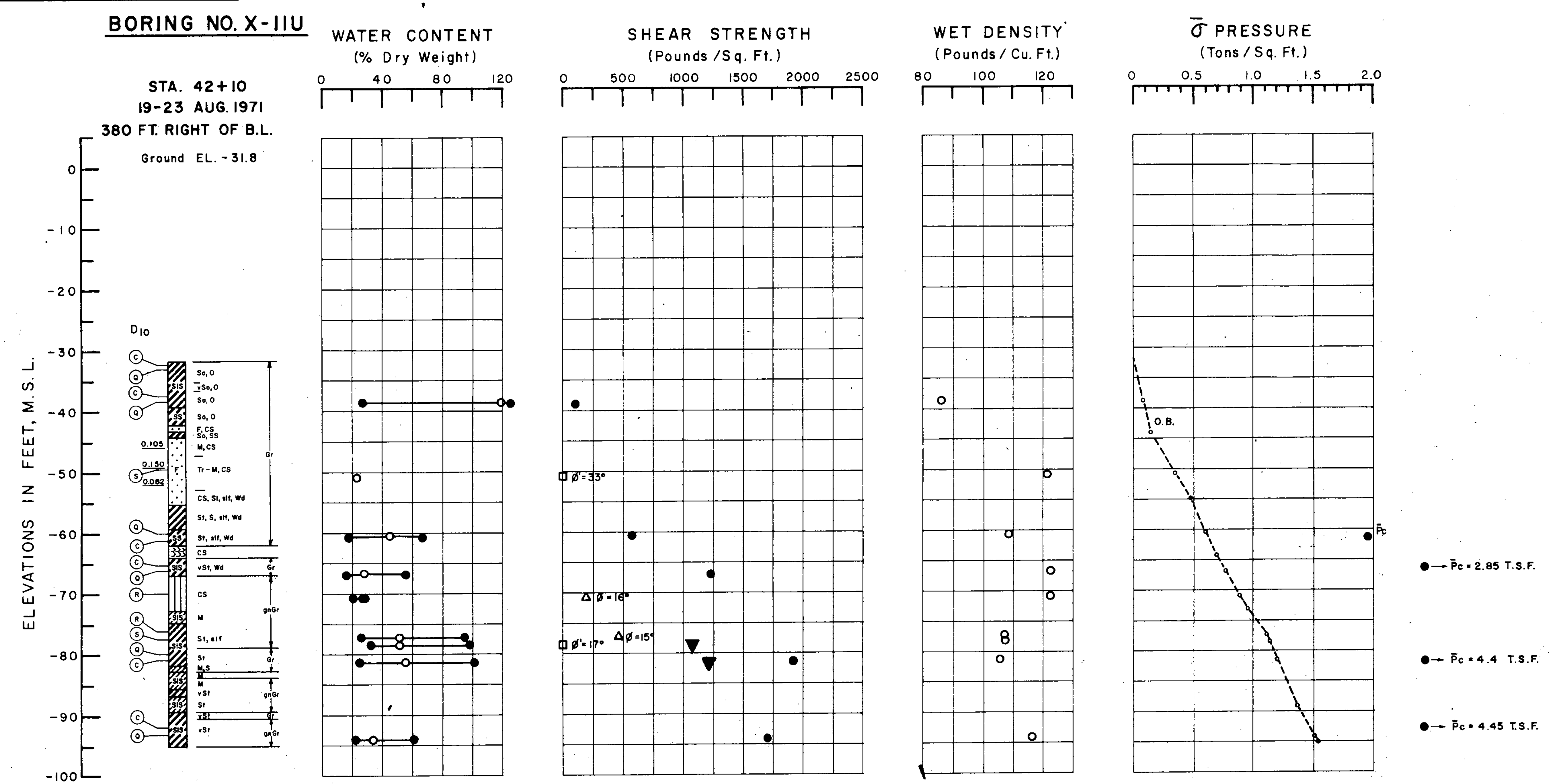
LAKE PONTCHARTRAIN LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

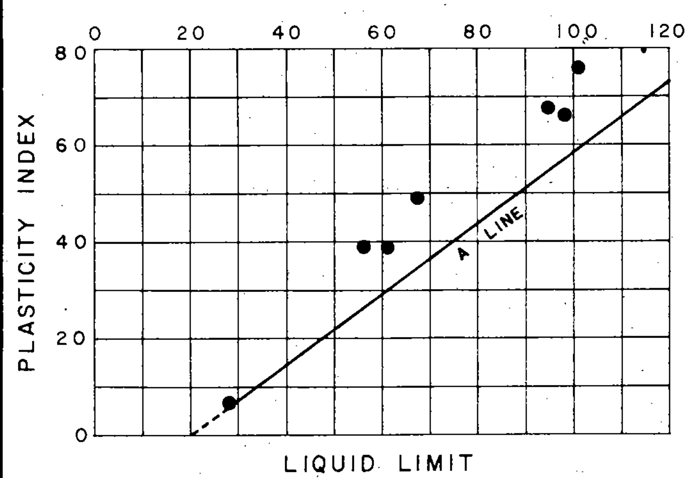
UNDISTURBED SOIL BORING
NO. X-10U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

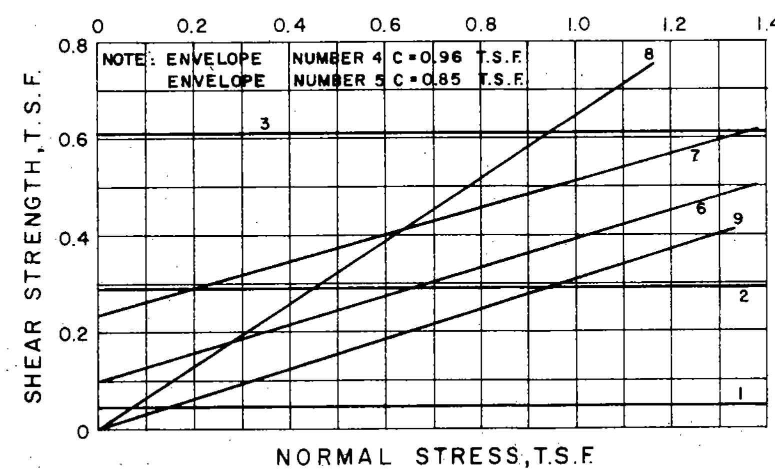
DATE [illegible] FILE NO. H-2-24417

PLATE **A-9**





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH Ø°	STRENGTH C, T.S.F.	CLASS
1	-38.8	Q-●	0	0.05	CH
2	-60.3		0	0.29	CH
3	-66.9		0	0.61	CH
4	-81.3		0	0.96	CH
5	-94.0		0	0.85	CH
6	-70.8	R-△	16	0.10	CL-ML
7	-77.2		15	0.24	CH
8	-50.8	S-□	33	0	SP
9	-78.5		17	0	CH

For General Notes See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

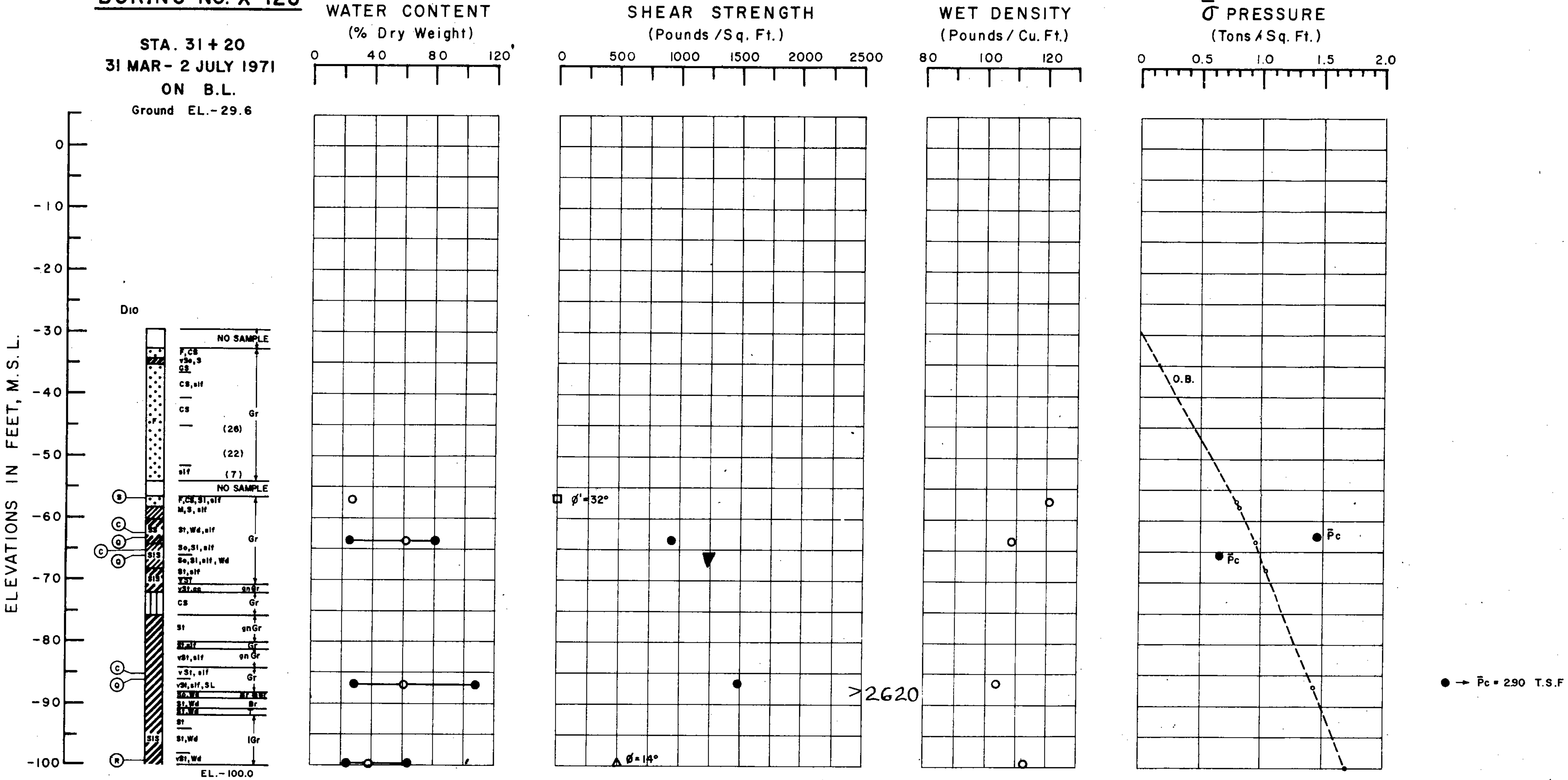
UNDISTURBED SOIL BORING NO. X-11U, SOIL TEST DATA

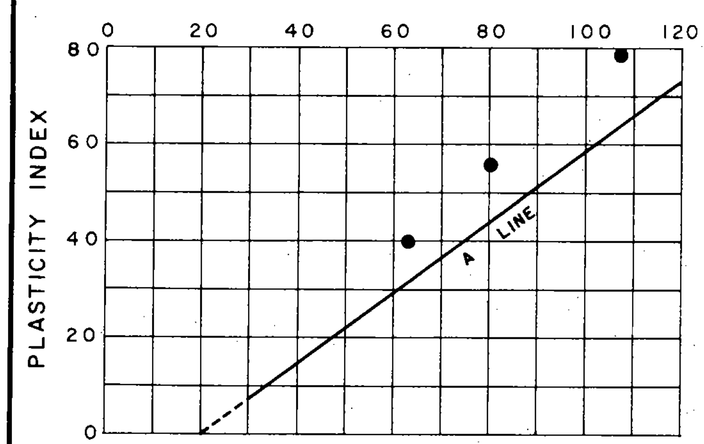
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417

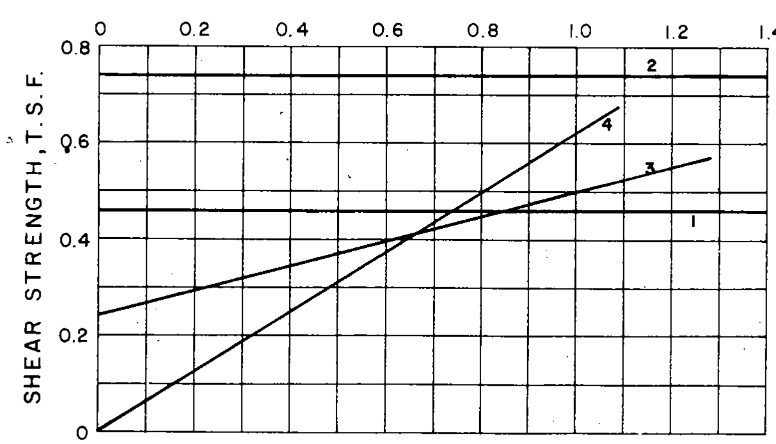
BORING NO. X-12U

STA. 31 + 20
31 MAR - 2 JULY 1971
ON B.L.
Ground EL. - 29.6





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
NO.	EL.		ø°	C,T.S.F.	
1	-63.8	Q-●	0	0.46	CH
2	-86.9		0	0.74	CH
3	-99.8	R-△	14	0.25	CH
4	-57.2	S-□	32	0	SM

For General Notes
See Plate A-2

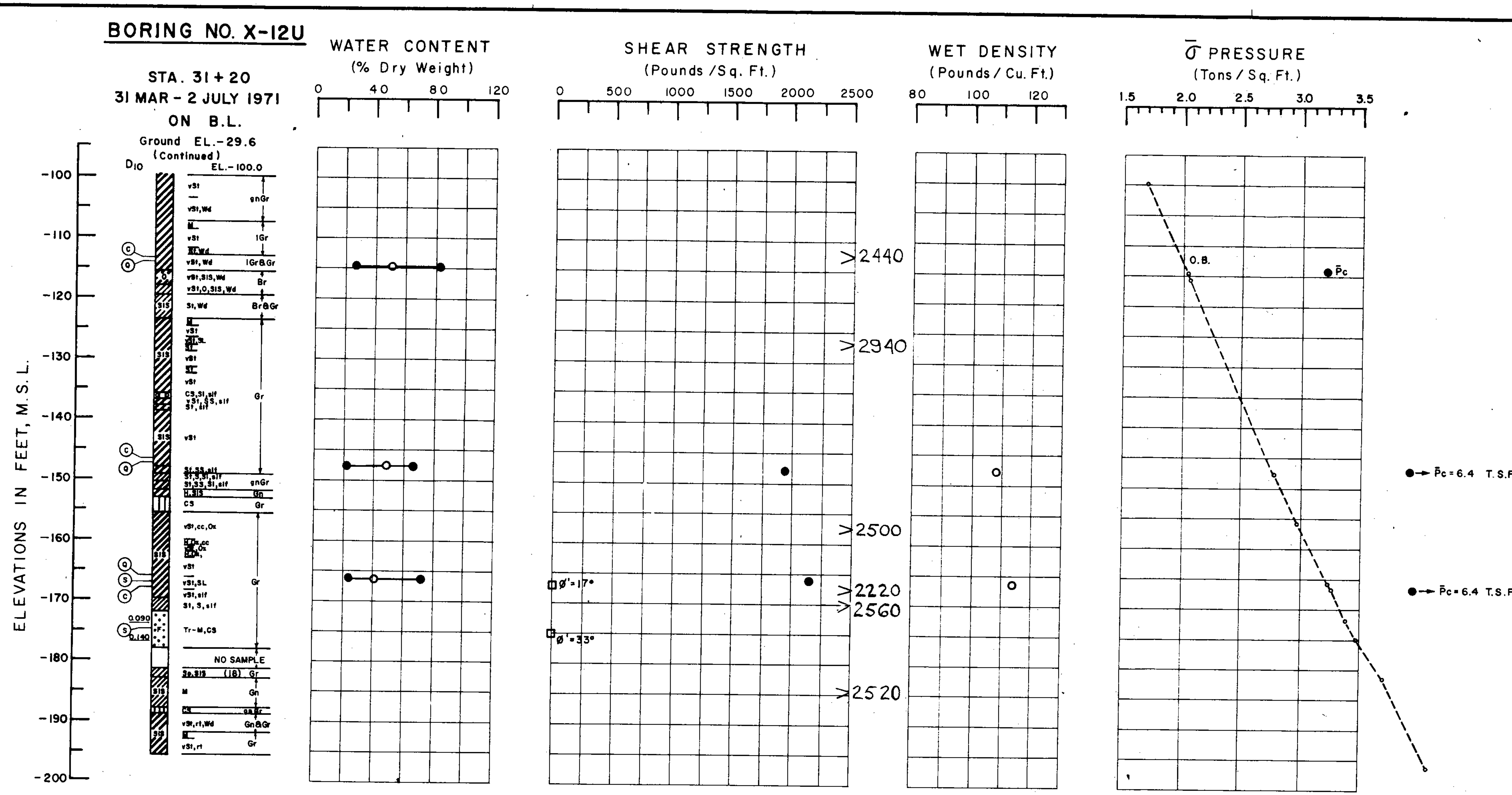
FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

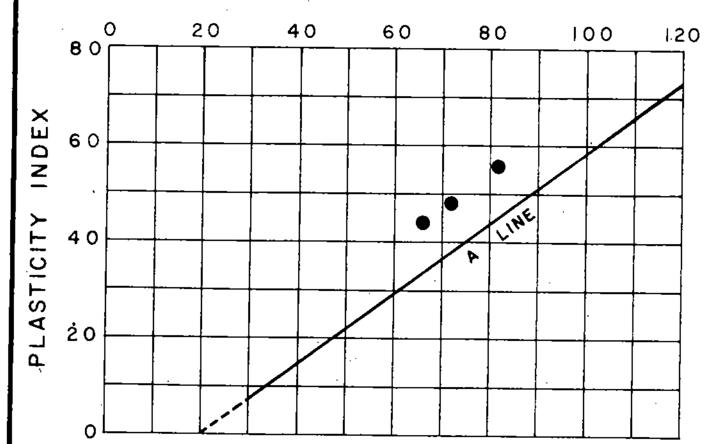
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X-12U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417





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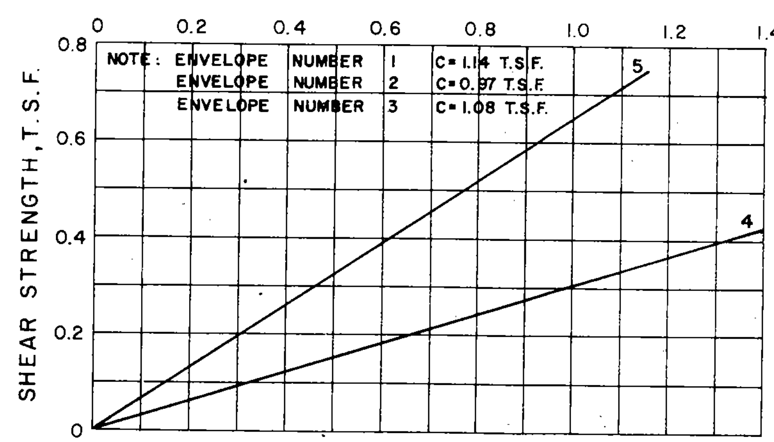


SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH Ø°	STRENGTH C,T.S.F.	CLASS
1	-114.9	Q-●	0	1.14	CH
2	-147.8	Q-●	0	0.97	CH
3	-166.1	Q-●	0	1.08	CH
4	-167.0	S-□	17	0	CH
5	-175.0	S-□	33	0	SP-CH

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA.

LAKE PONTCHARTRAIN LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

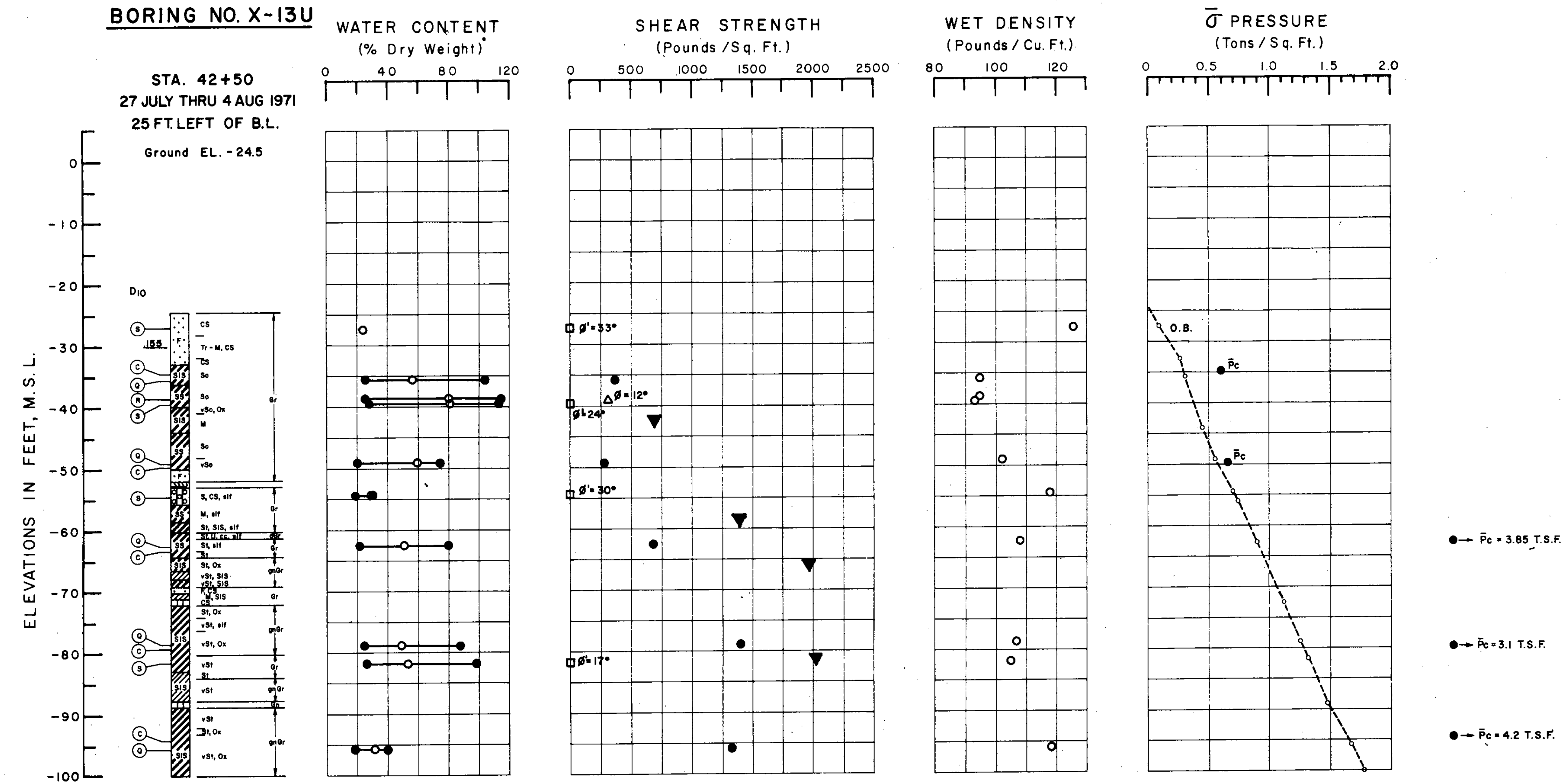
UNDERSTURBED SOIL BORING

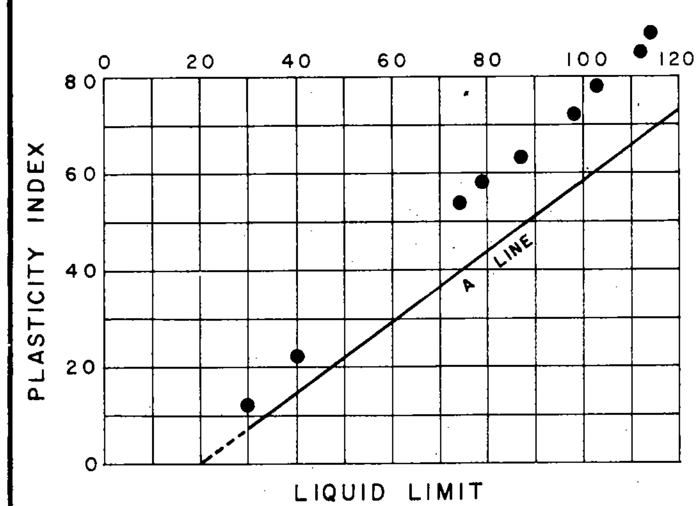
NO. X-12U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417

PLATE **A-12**





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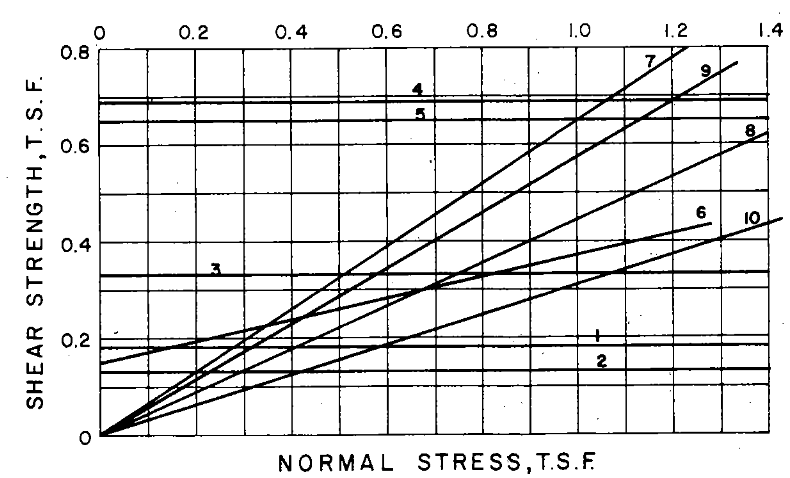


SHEAR STRENGTH DATA

ENVELOPE NO.	EL.	TYPE	STRENGTH φ°	C, T.S.F.	CLASS
1	-35.7	Q-●	0	0.18	CH
2	-49.1		0	0.13	CH
3	-62.8		0	0.33	CH
4	-78.9		0	0.69	CH
5	-95.6		0	0.65	CL
6	-38.9	R-△	12	0.15	CH
7	-27.4	S-□	33	0	SP
8	-39.6		24	0	CH
9	-54.4		30	0	CL
10	-81.7		17	0	CH

For General Notes
See Plate A-2

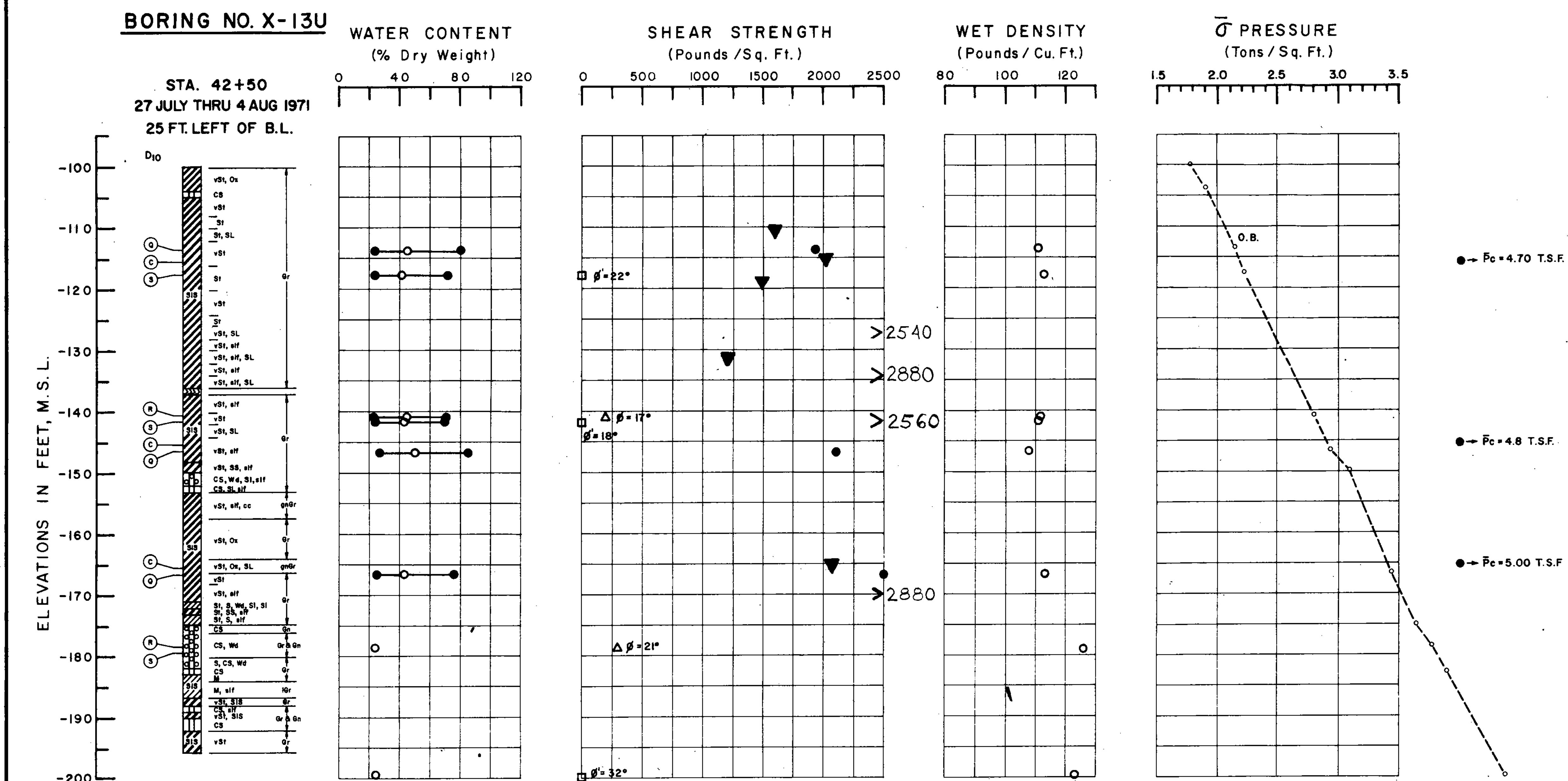
FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

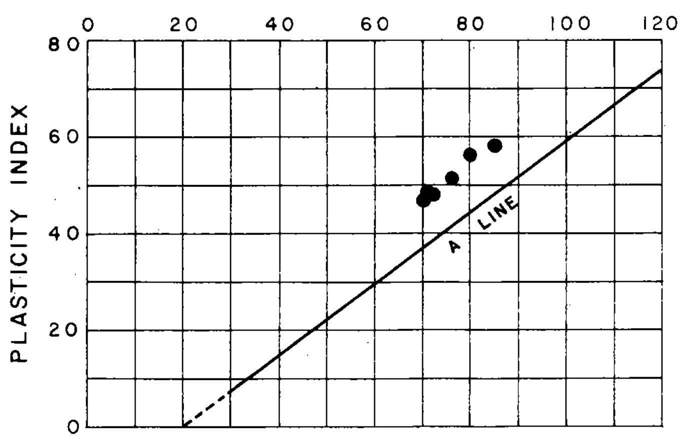
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X-13U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





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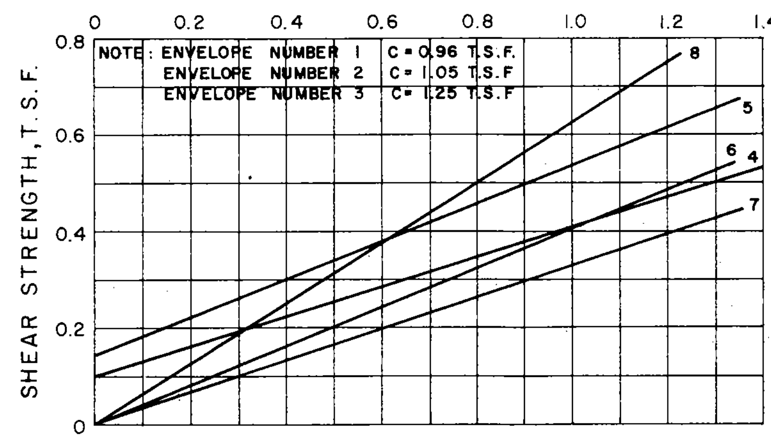


SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F.	CLASS
1	-113.9	Q-●	0	0.96	CH
2	-146.7	Q-●	0	1.05	CH
3	-166.8	Q-●	0	1.25	CH
4	-141.0	R-Δ	17	0.10	CH
5	-178.8	R-Δ	21	0.15	SC
6	-117.9	S-□	22	0	CH
7	-141.9	S-□	18	0	CH
8	-199.7	S-□	32	0	SC

For General Notes
See Plate A-2

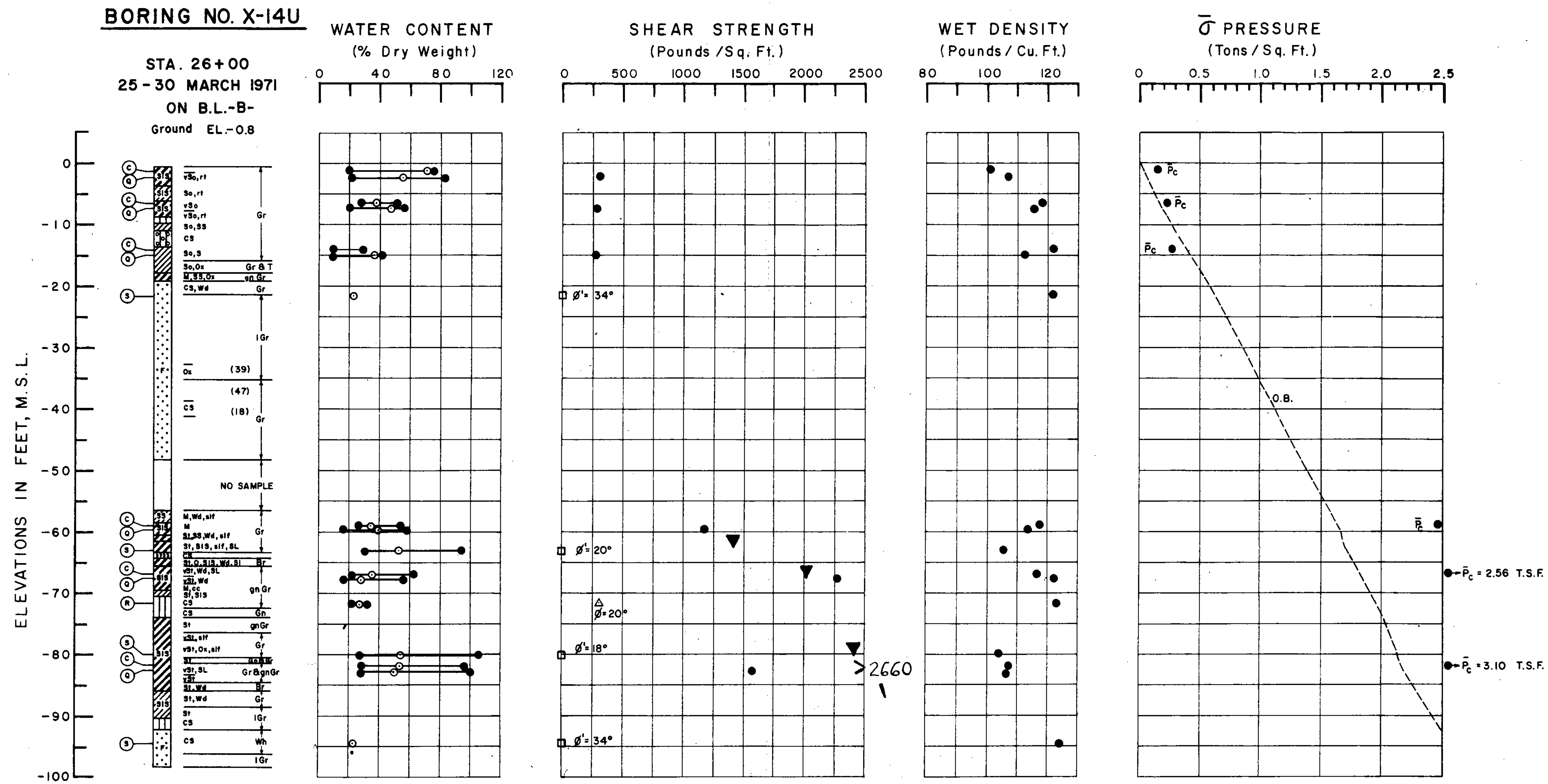
FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

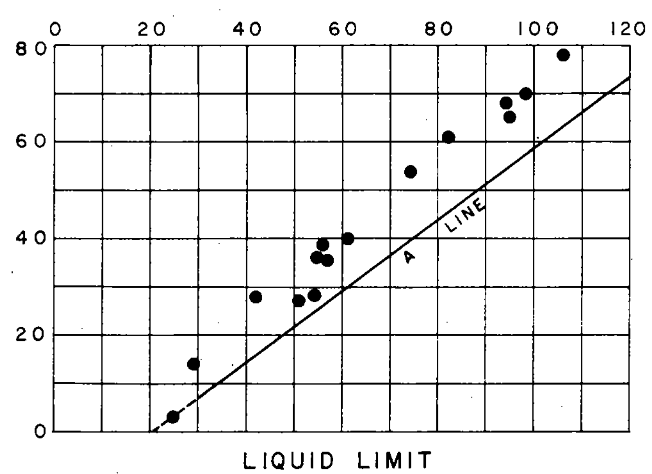
LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 — DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X-13U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417





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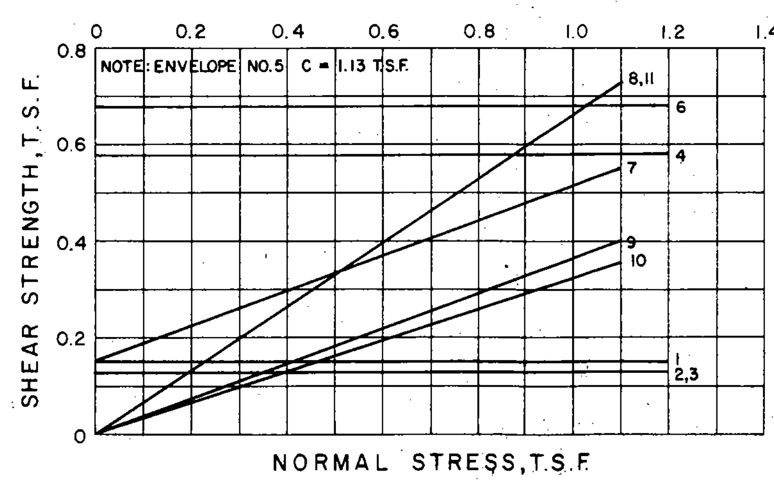


SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F	CLASS
1	-2.8	Q-●	0	0.15	CH
2	-7.6	Q-●	0	0.13	CH
3	-15.1	Q-●	0	0.13	CL
4	-60.1	Q-●	0	0.58	CH
5	-68.0	Q-●	0	0.13	CH
6	-83.0	Q-●	0	0.78	CH
7	-71.9	R-△	20°	0.15	ML
8	-22.0	S-□	34°	0	SP
9	-63.1	S-□	20°	0	CH
10	-80.1	S-□	18°	0	CH
11	-95.0	S-□	34°	0	SM

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN

RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

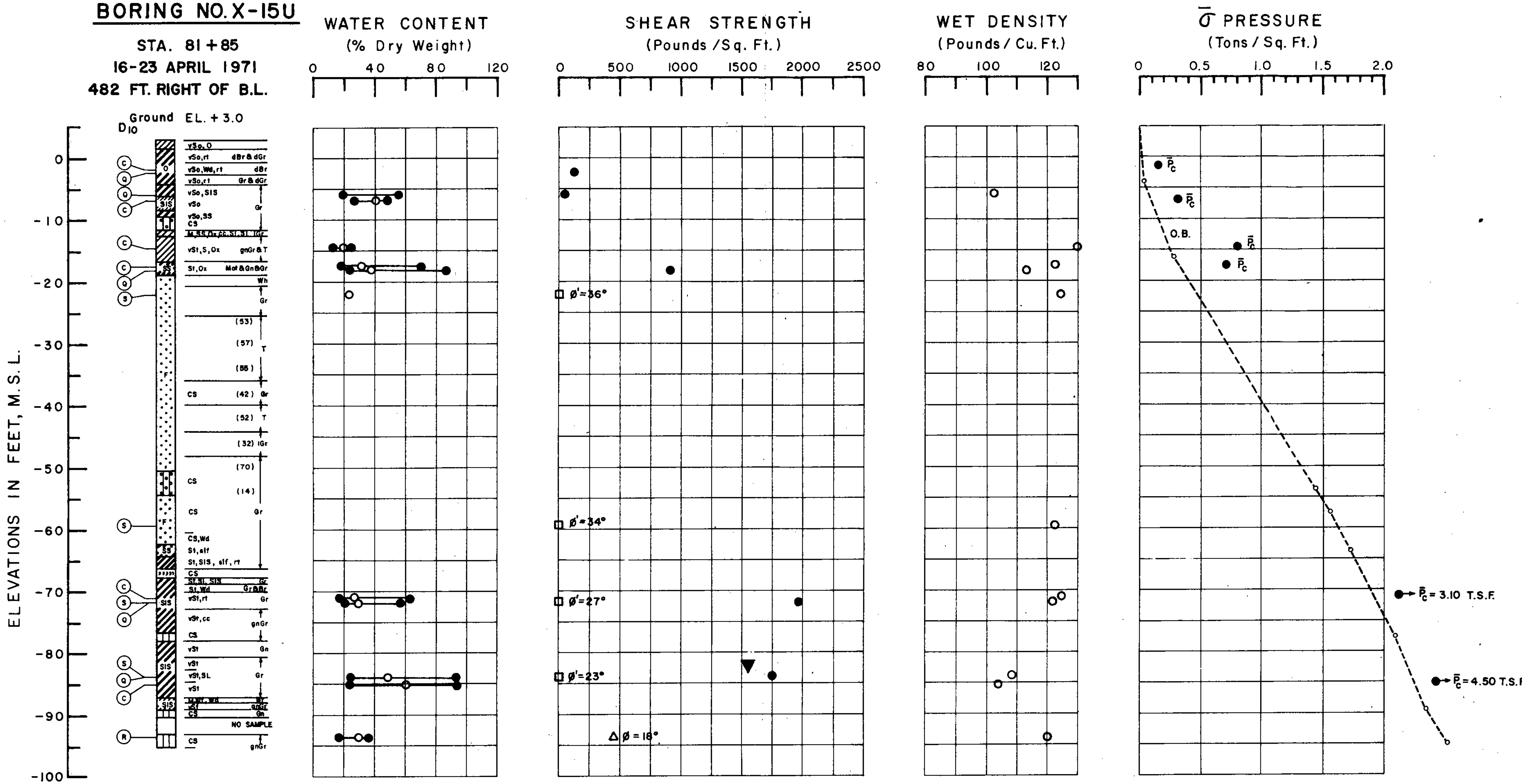
UNDISTURBED SOIL BORING
NO. X-14U, SOIL TEST DATA

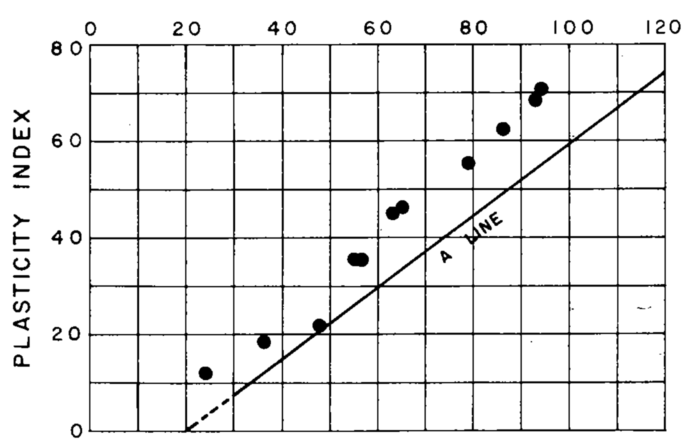
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417

BORING NO. X-15U

STA. 81+85
16-23 APRIL 1971
482 FT. RIGHT OF B.L.





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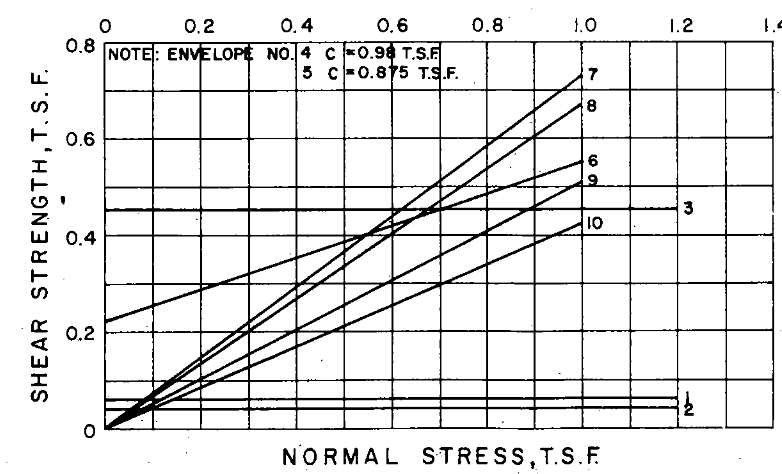


SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F.	CLASS
1	- 2.5	Q - ●	0	0.06	O H
2	- 6.0		0	0.04	C H
3	- 18.3		0	0.45	C H
4	- 71.9		0	0.98	C H
5	- 84.0		0	0.875	C H
6	-94.1	R - Δ	18°	0.22	C L
7	- 22.1	S - □	36°	0	S M
8	- 59.8		34°	0	S M
9	- 71.9		27°	0	C H
10	- 84.0		23°	0	C H

For General Notes See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

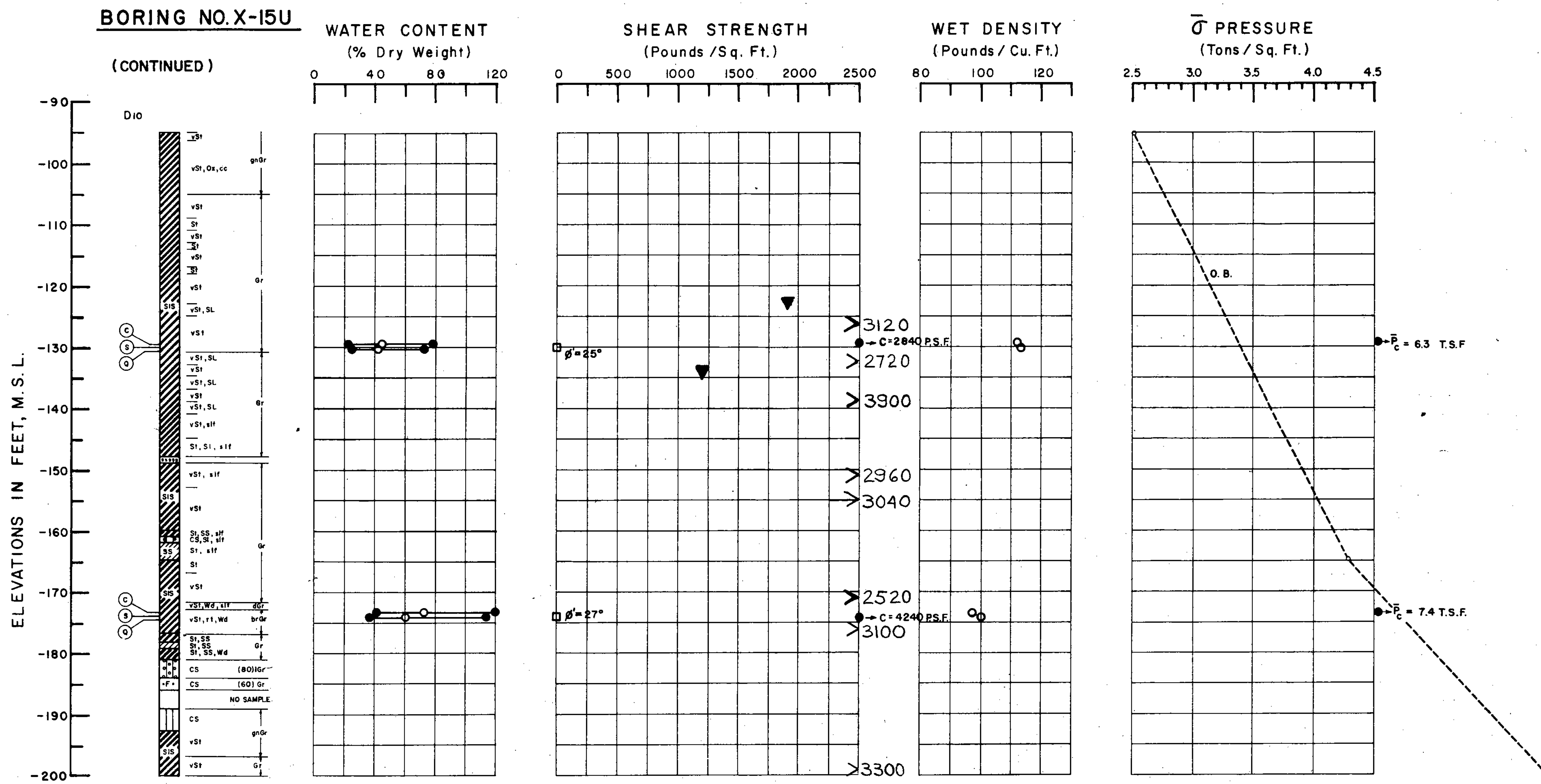
UNDISTURBED SOIL BORING NO. X-15U, SOIL TEST DATA

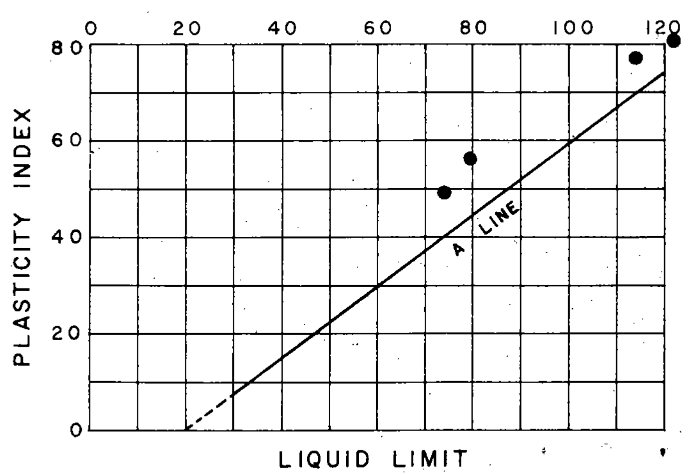
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417

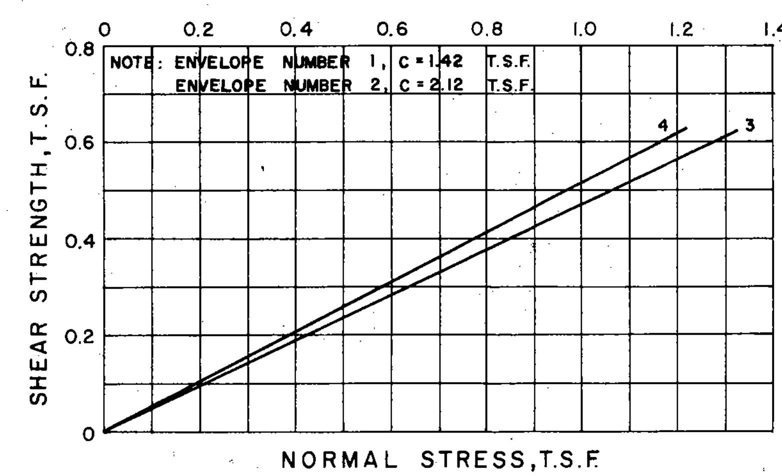
BORING NO. X-15U

(CONTINUED)





PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH ø°	STRENGTH C,T.S.F.	CLASS
1	-130.1	Q-●	0	1.42	CH
2	-174.2		0	2.12	CH
3	-130.1	S-□	25	0	CH
4	-174.2		27	0	CH

For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

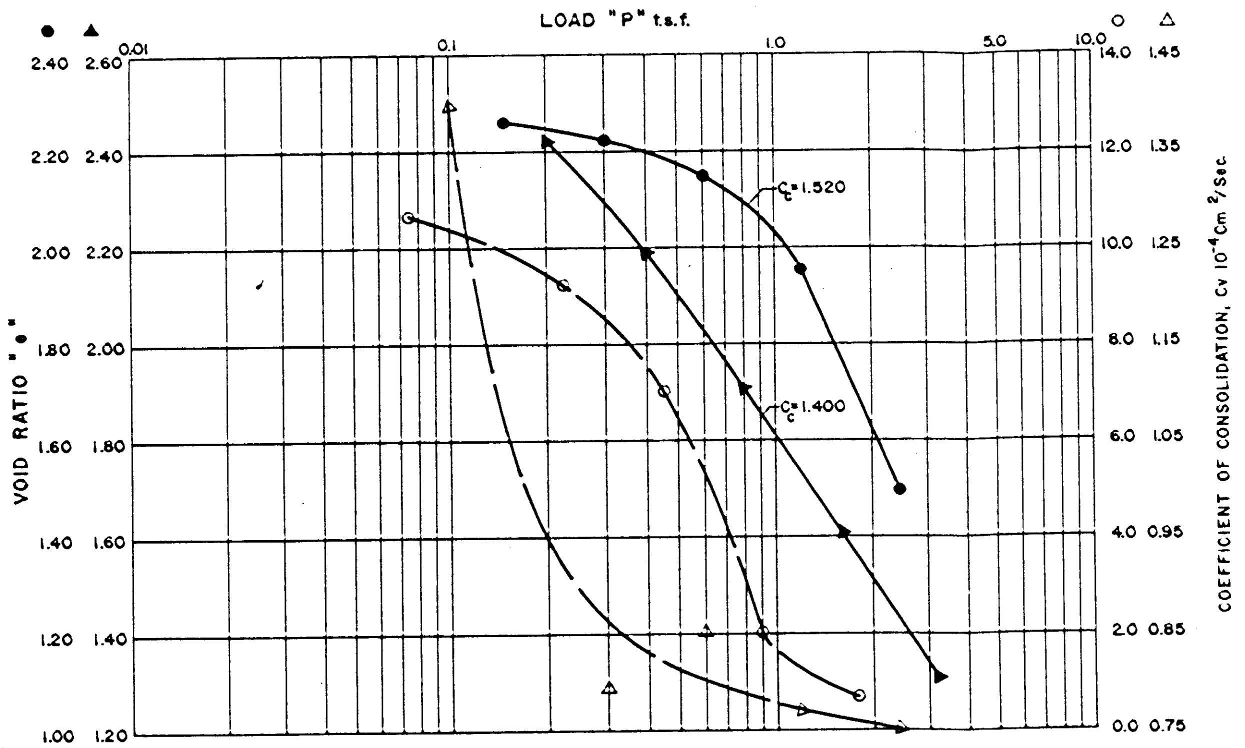
LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

UNDISTURBED SOIL BORING
NO. X-15U, SOIL TEST DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE: JULY 1972 FILE NO. H-2-24417

PLATE A-17



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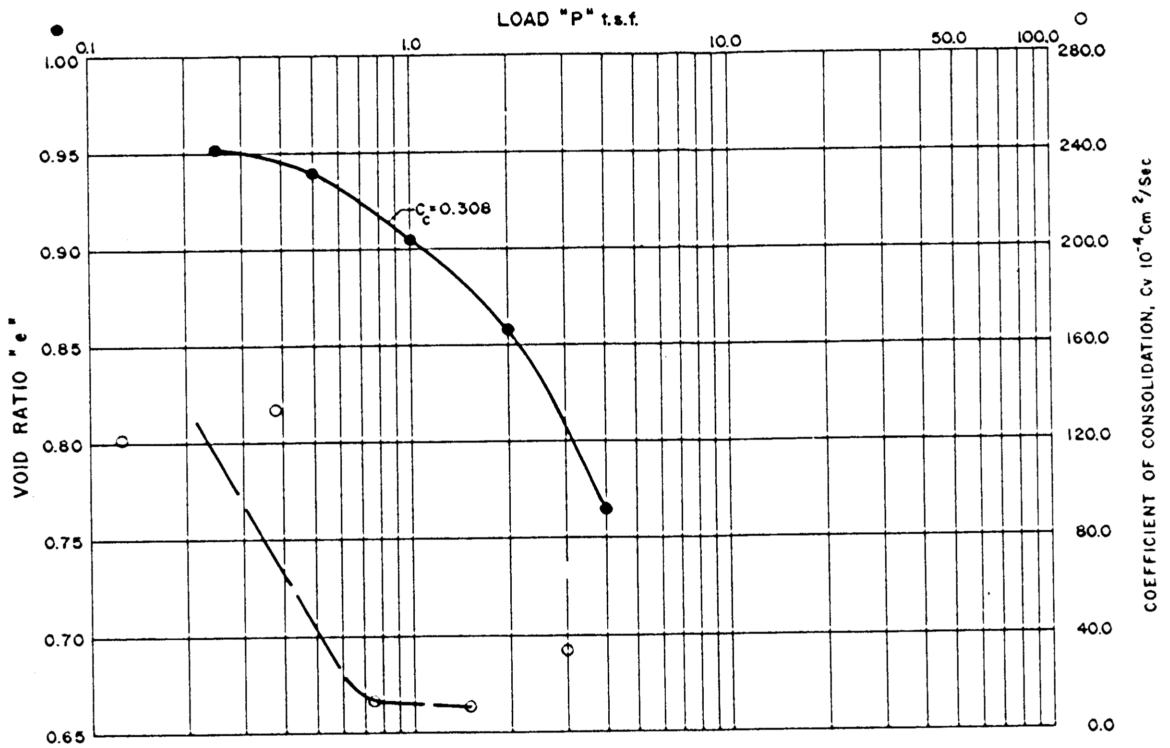

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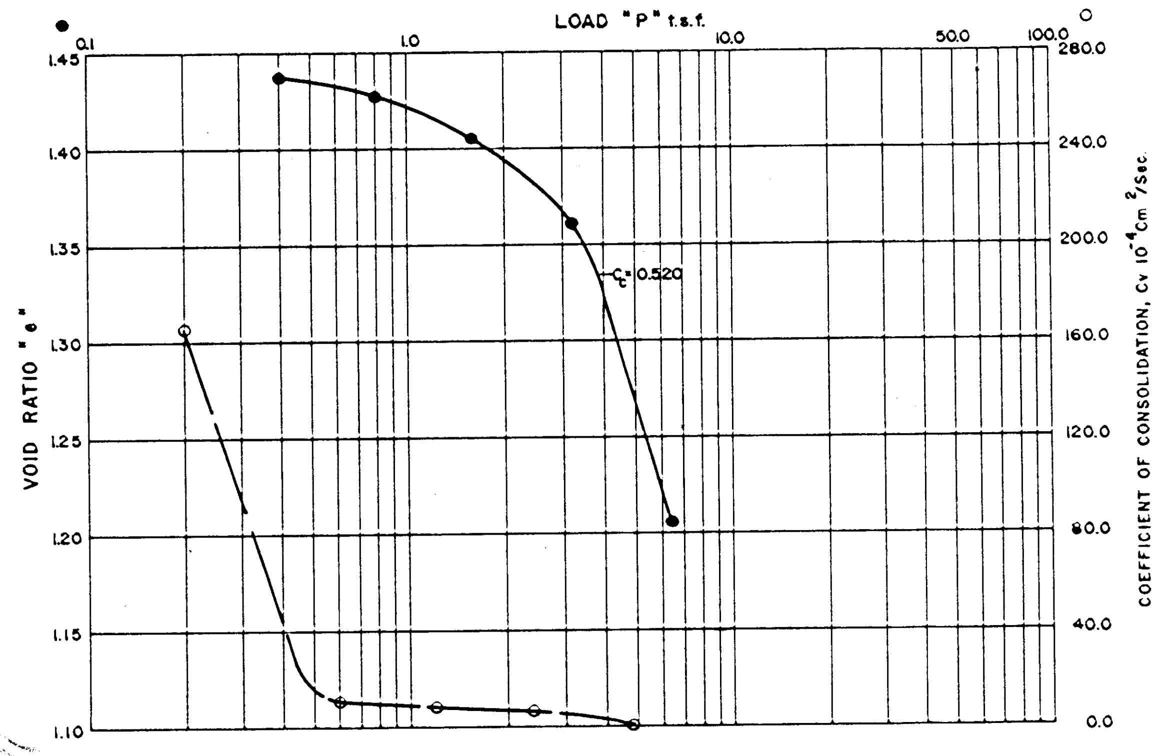

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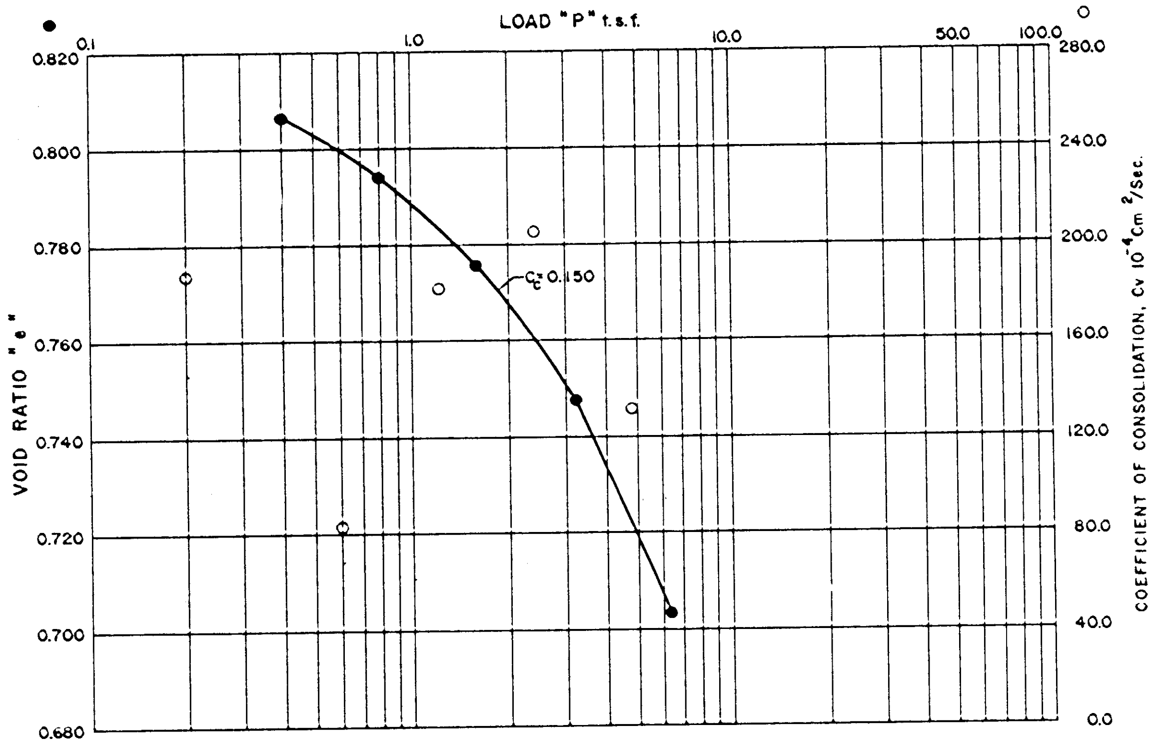


SAMPLE NO. 18-C, EL. - 90.6

For General Notes
See Plate A-2

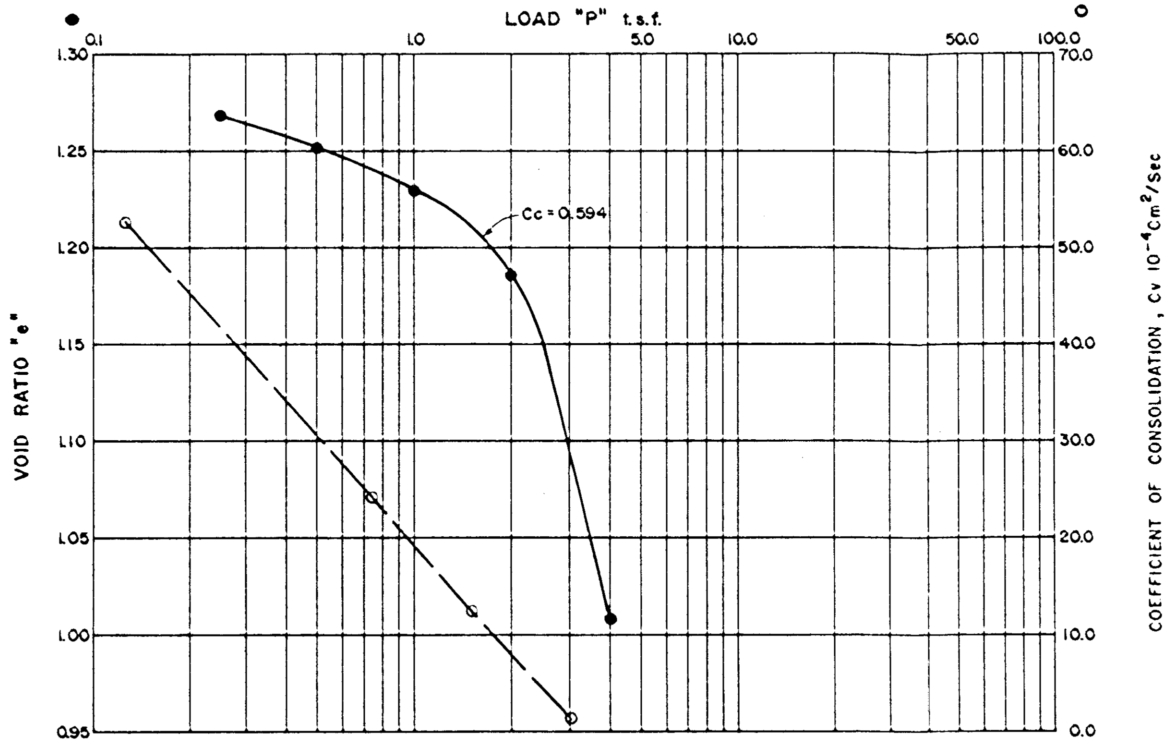
FREDERIC R. HARRIS INC
CONSULTING ENGINEERS - NEW ORLEANS LA

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 - DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

CONSOLIDATION TEST RESULTS
BORING X-1U

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972　　FILE NO. H-2-24417



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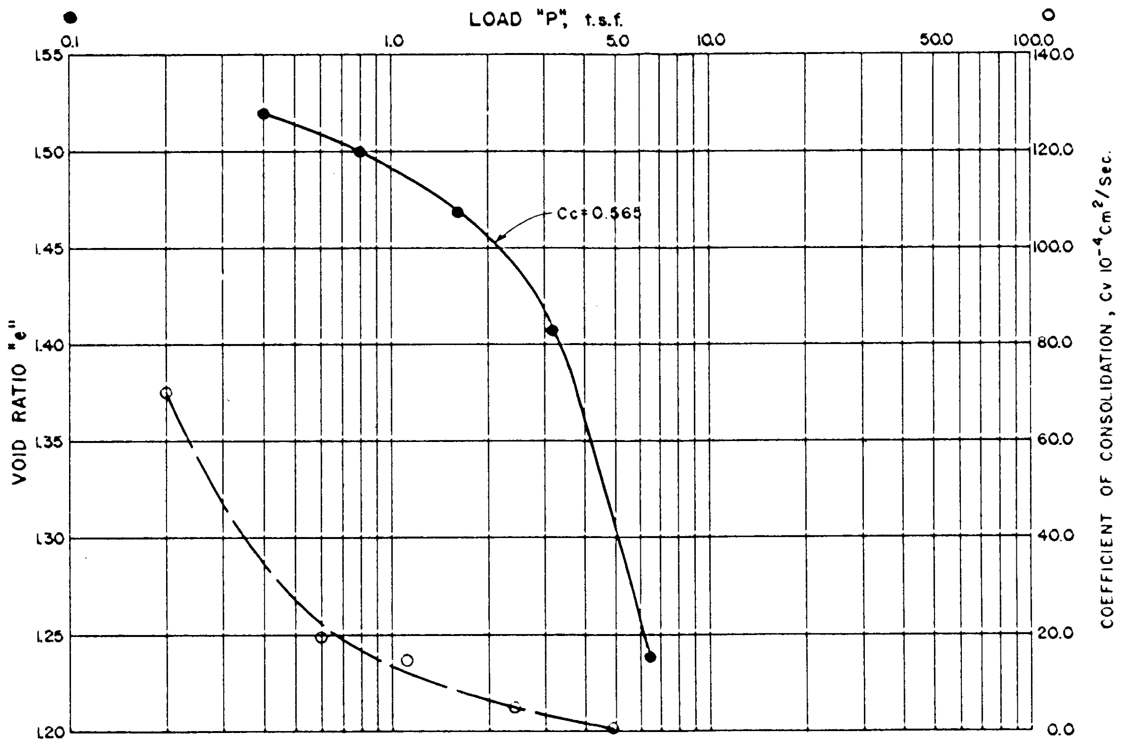

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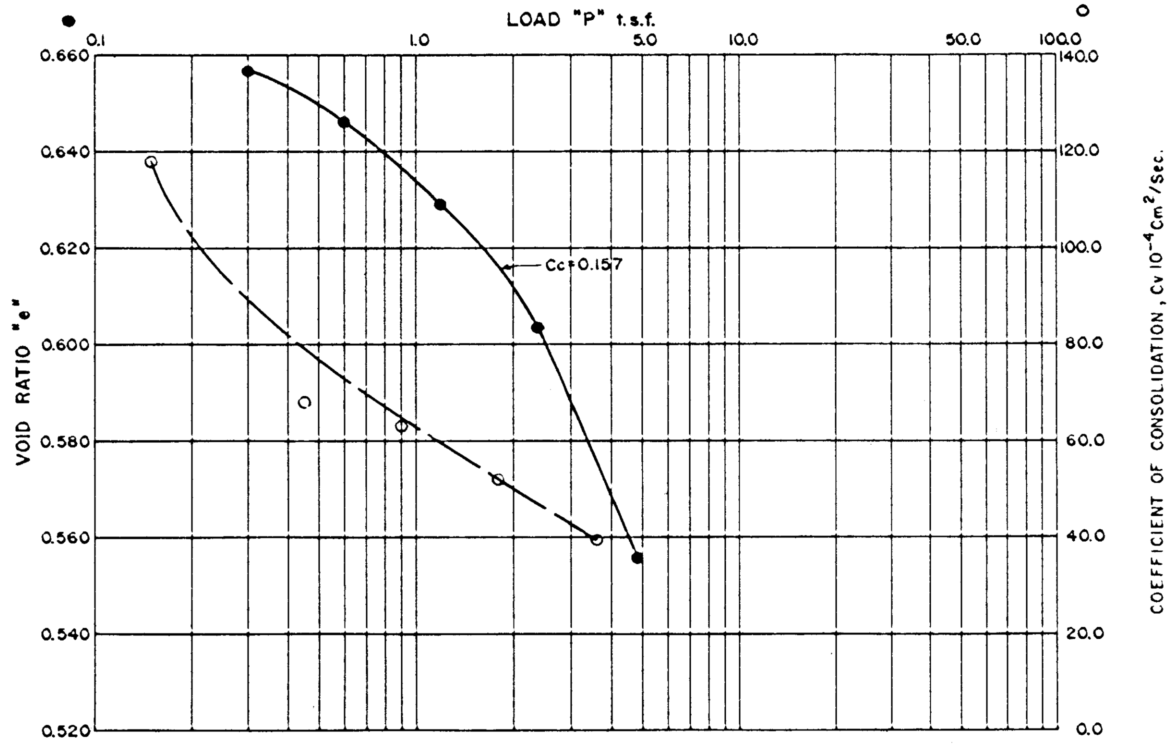

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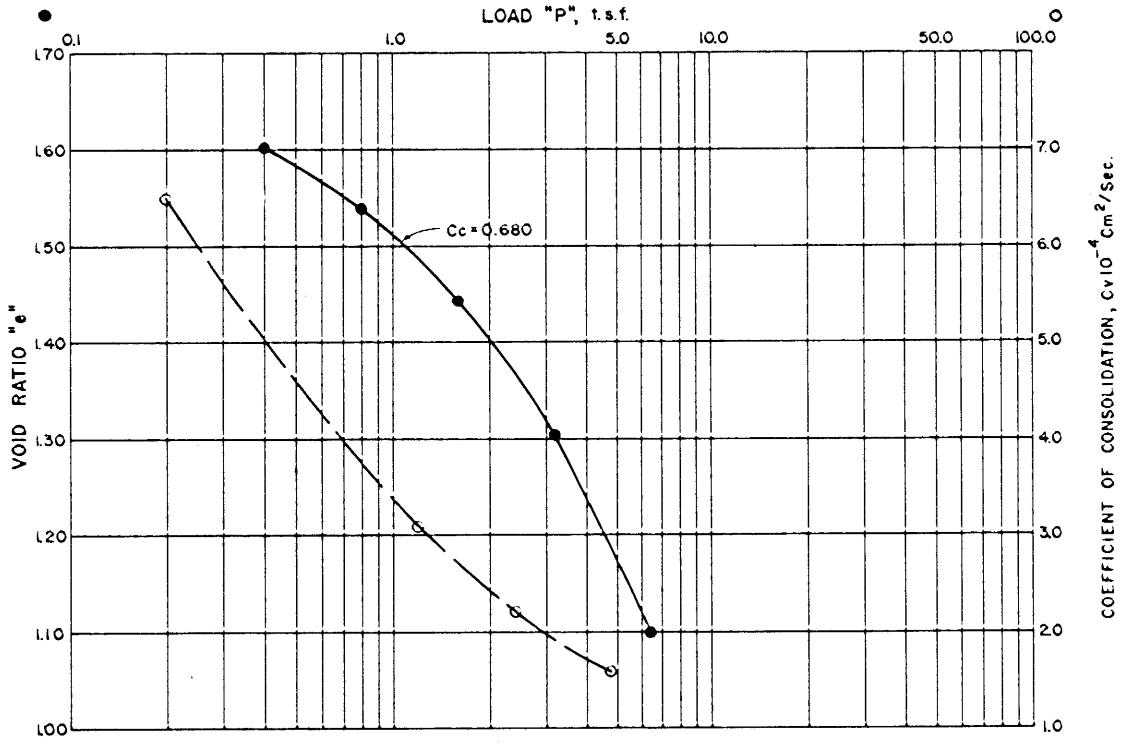


SAMPLE NO. 20-A EL. -94.0

For General Notes
See Plate A-2-

FREDERIC R. HARRIS INC.
CONSULTING ENGINEERS - NEW ORLEANS, LA.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 6 – DETAIL DESIGN

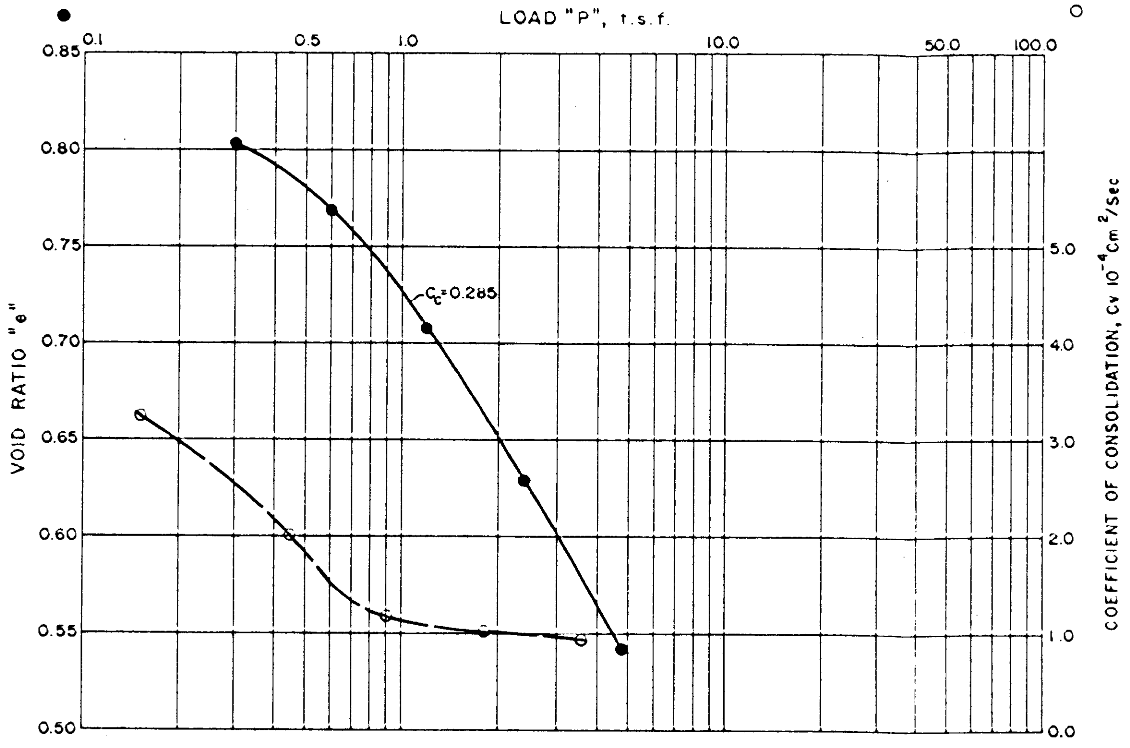
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

CONSOLIDATION TEST RESULTS

BORING X-3U

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO. H-2-24417



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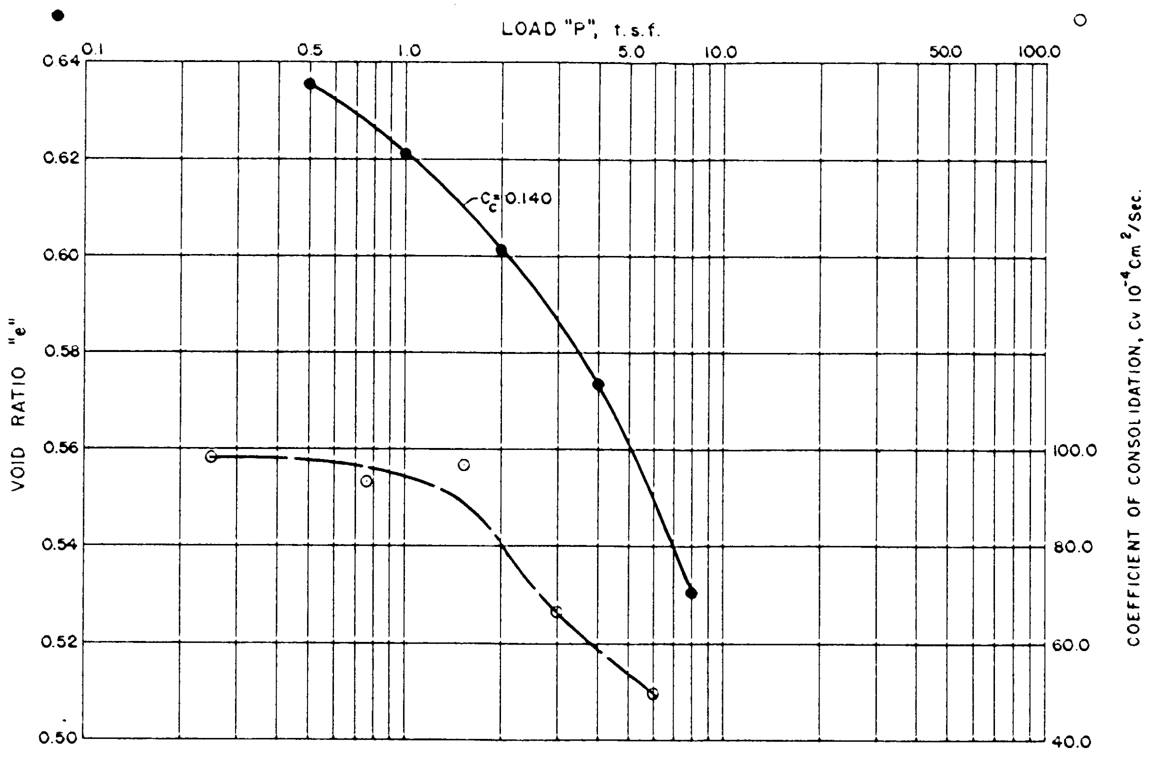

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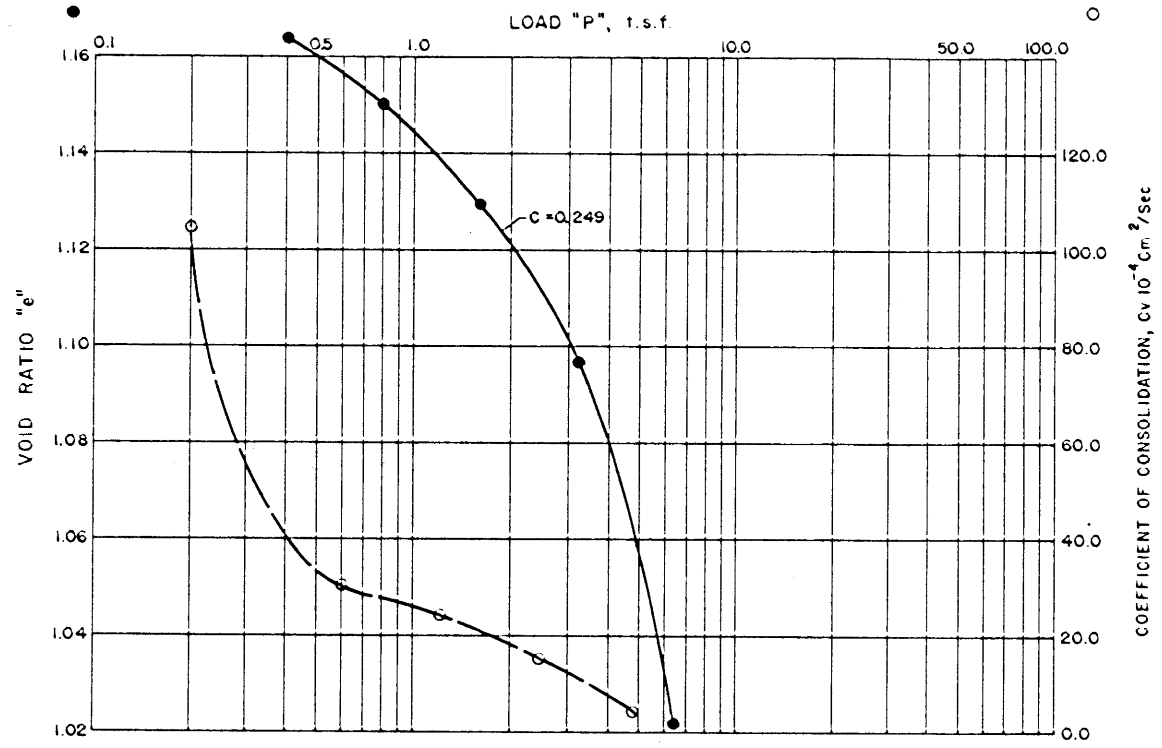
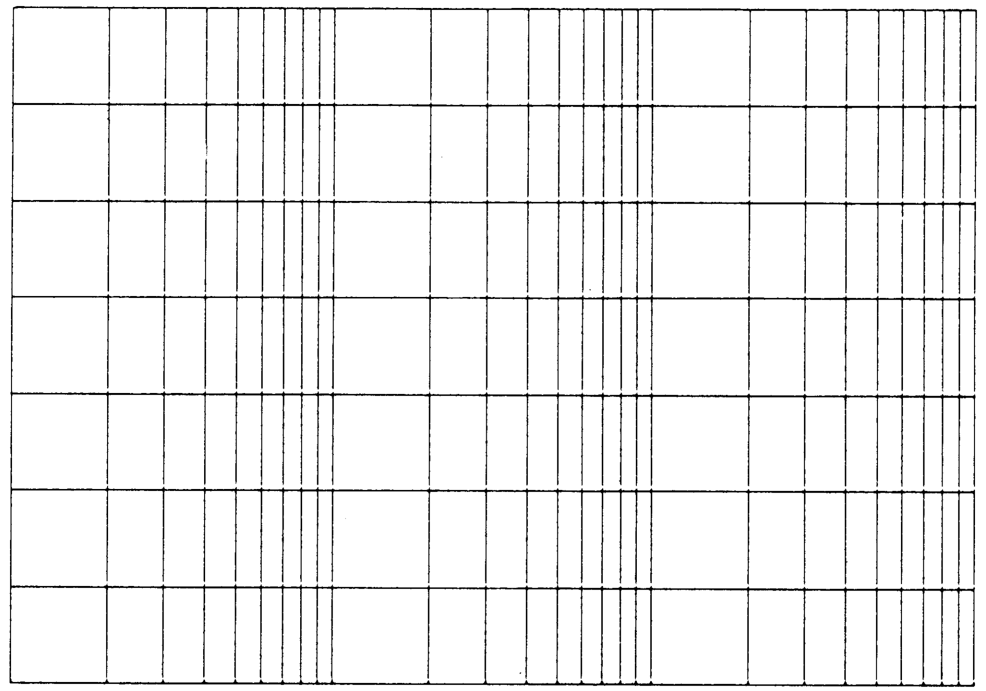


SAMPLE NO. 15-C, EL. -78.8



For General Notes
See Plate A-2

FREDERIC R. HARRIS INC
CONSULTING ENGINEERS NEW ORLEANS LA

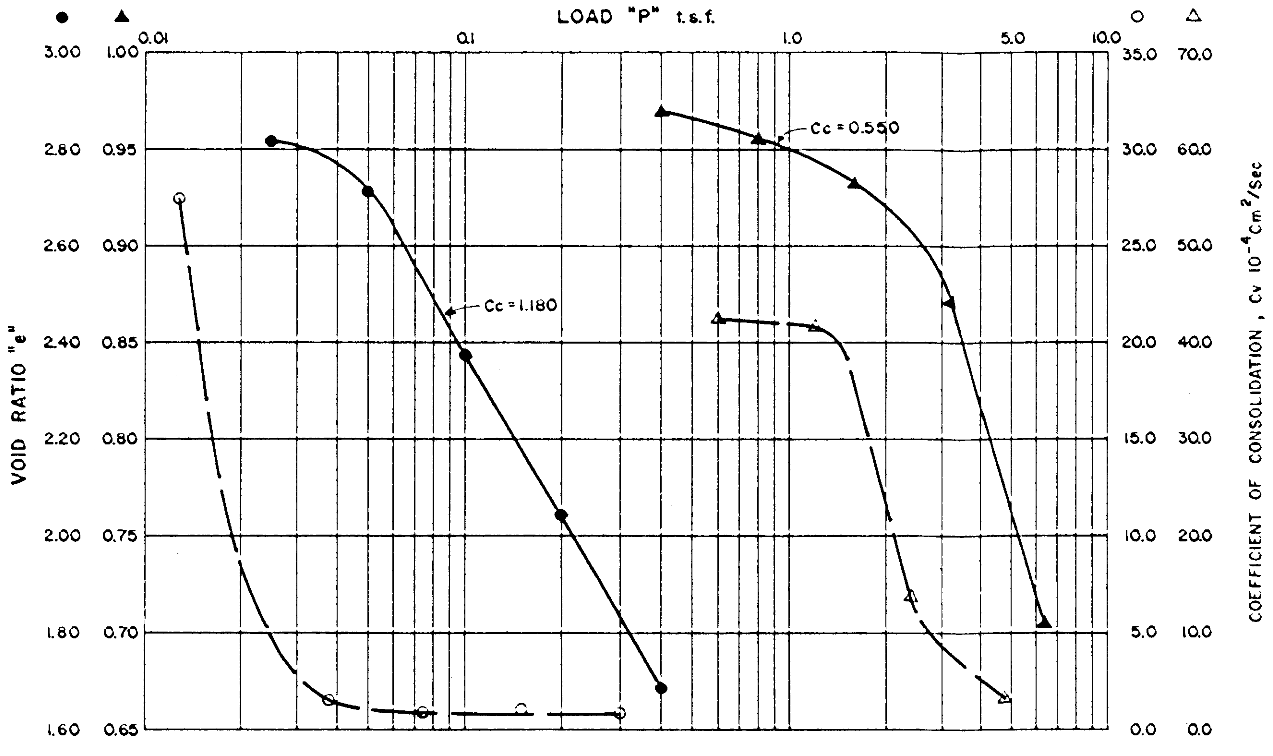
LAKE PONTCHARTRAIN LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 6 – DETAIL DESIGN
RIGOLETS CONTROL STRUCTURE
AND CLOSURE DAM

CONSOLIDATION TEST RESULTS

BORING X-5U

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

DATE JULY 1972 FILE NO H-2-24417



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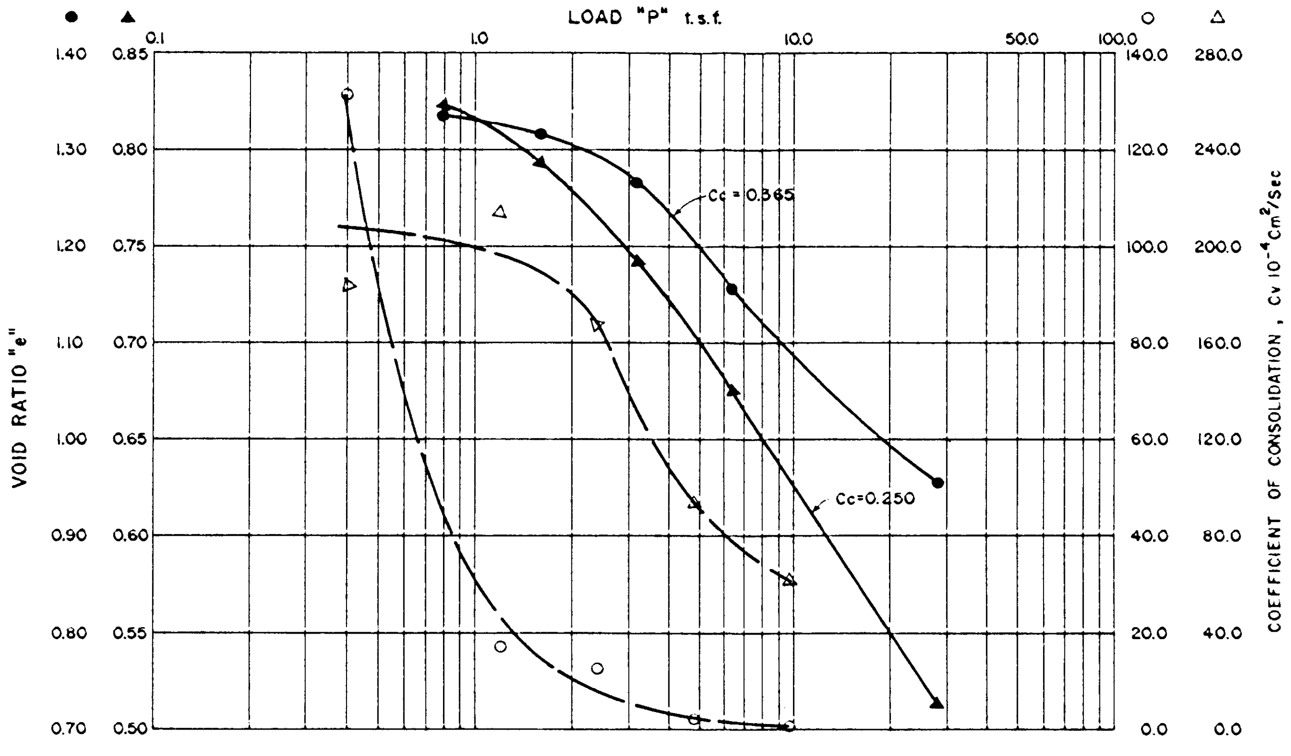

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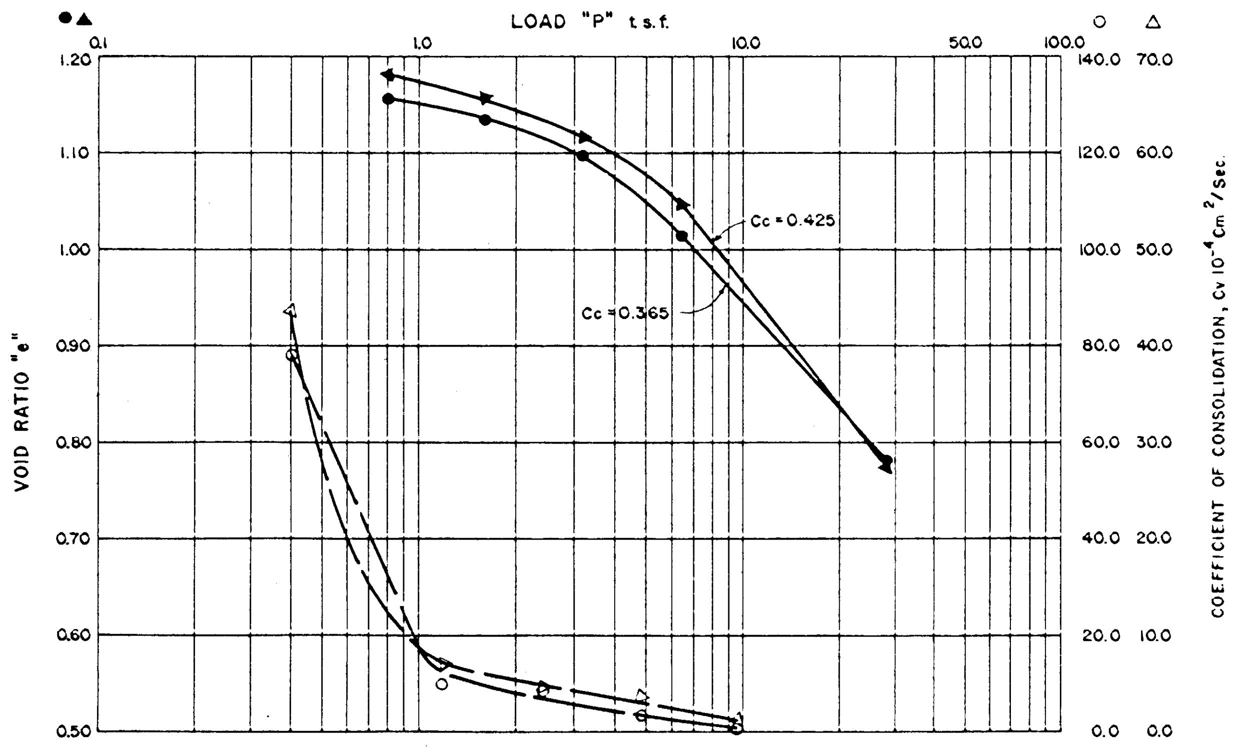

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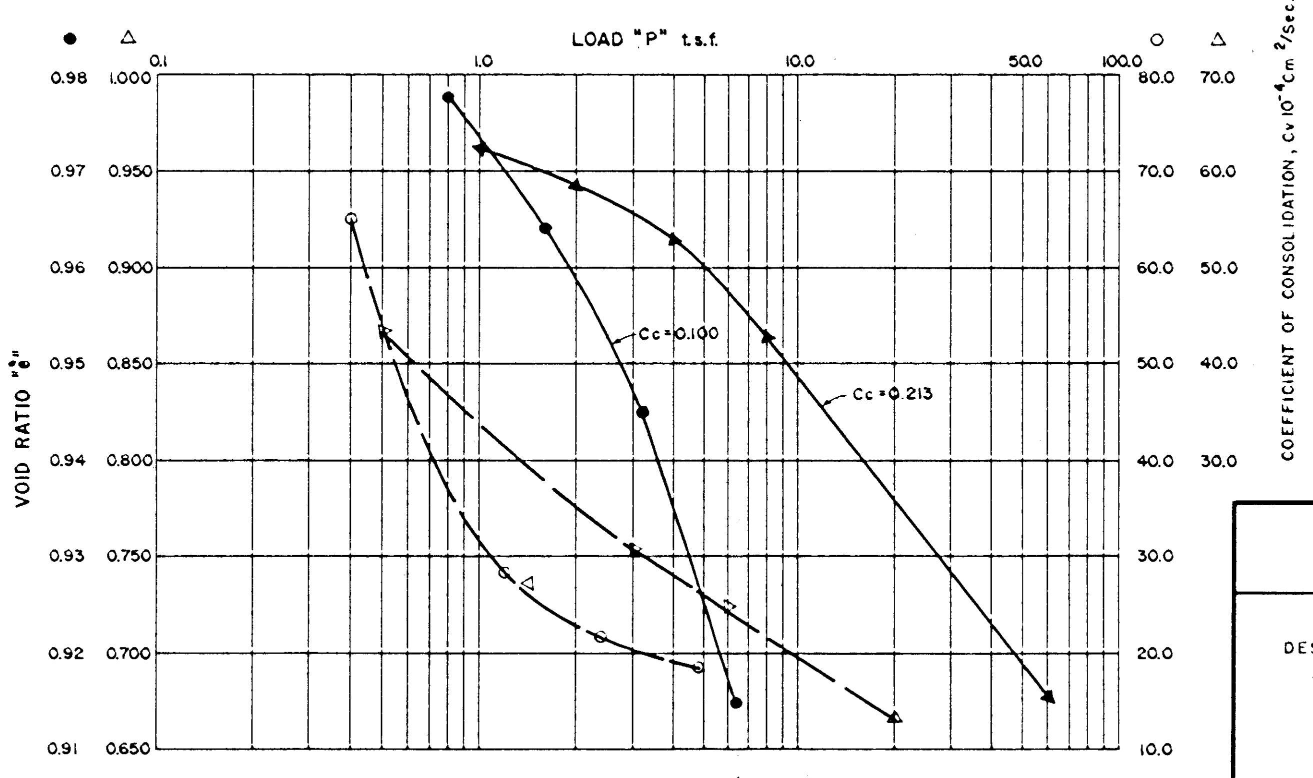

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For General Notes
See Plate A-2

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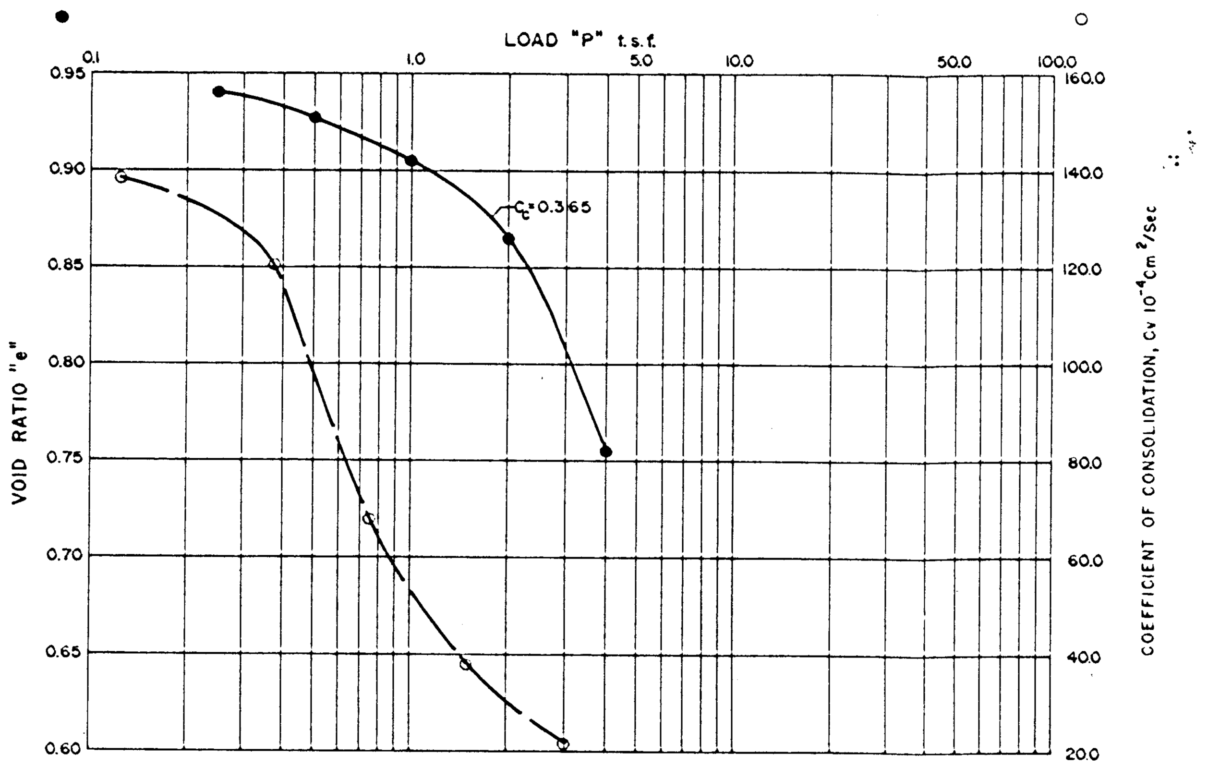

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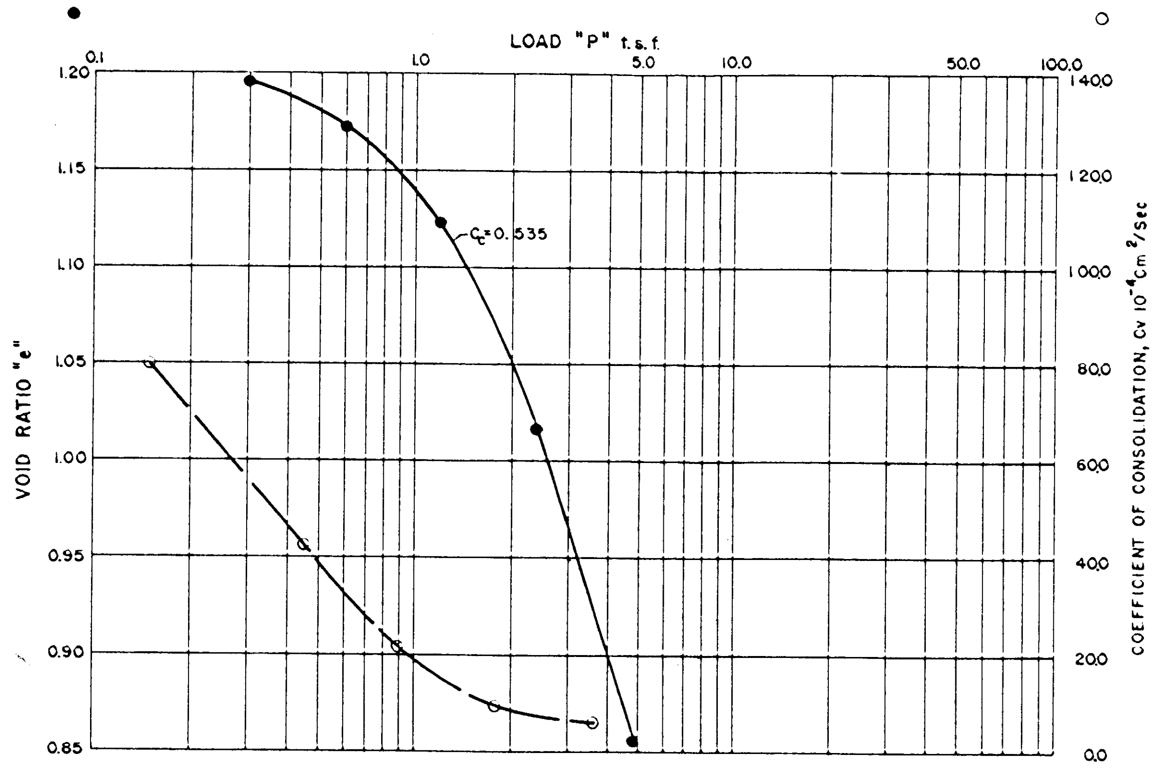

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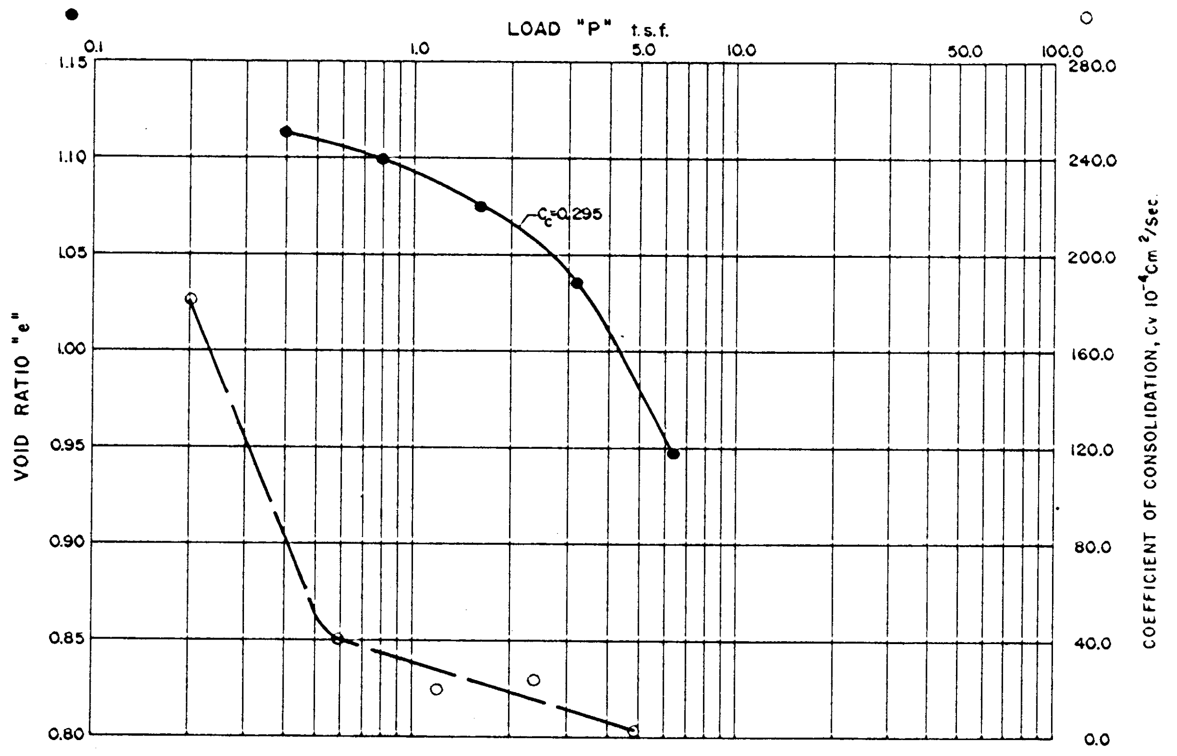

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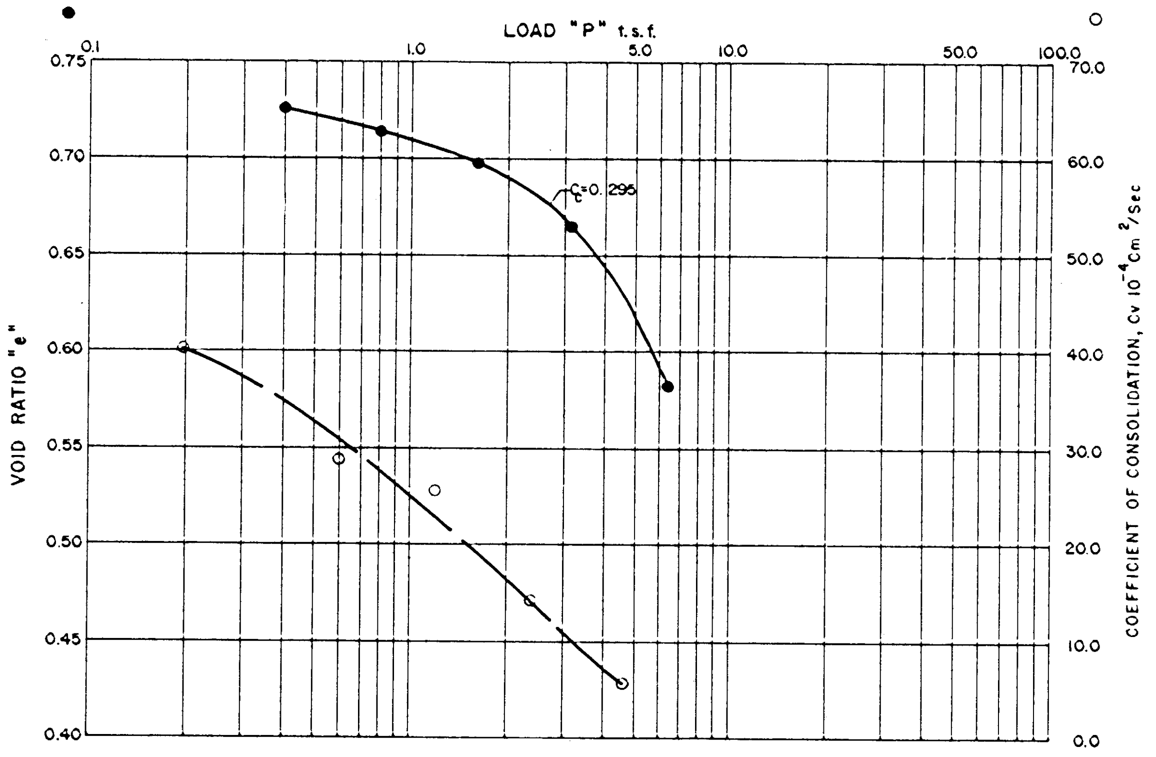

SAMPLE NO. 17-C, EL. - 87.6

For General Notes
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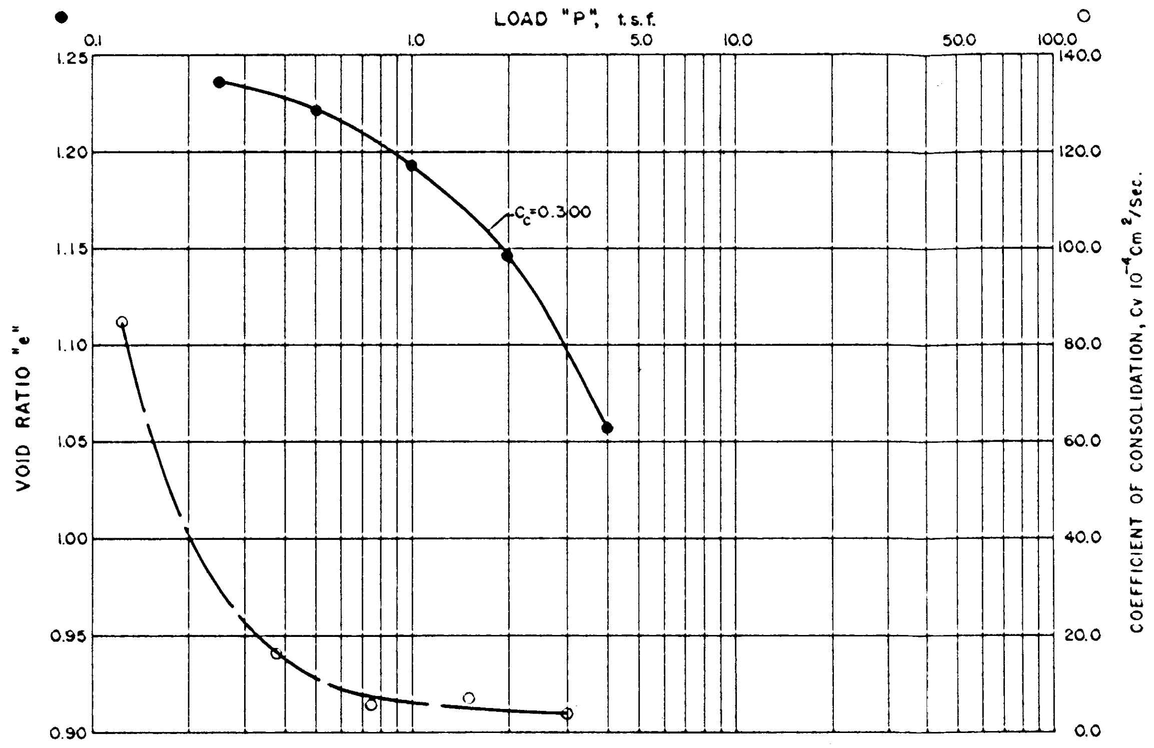

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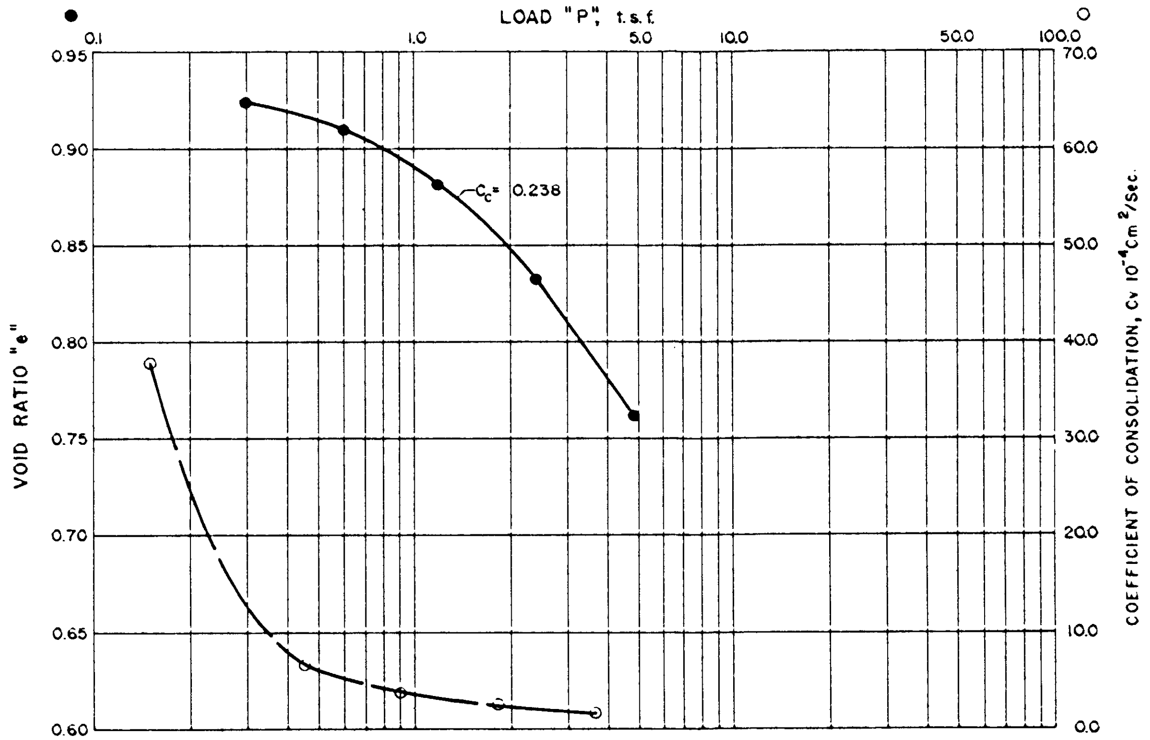

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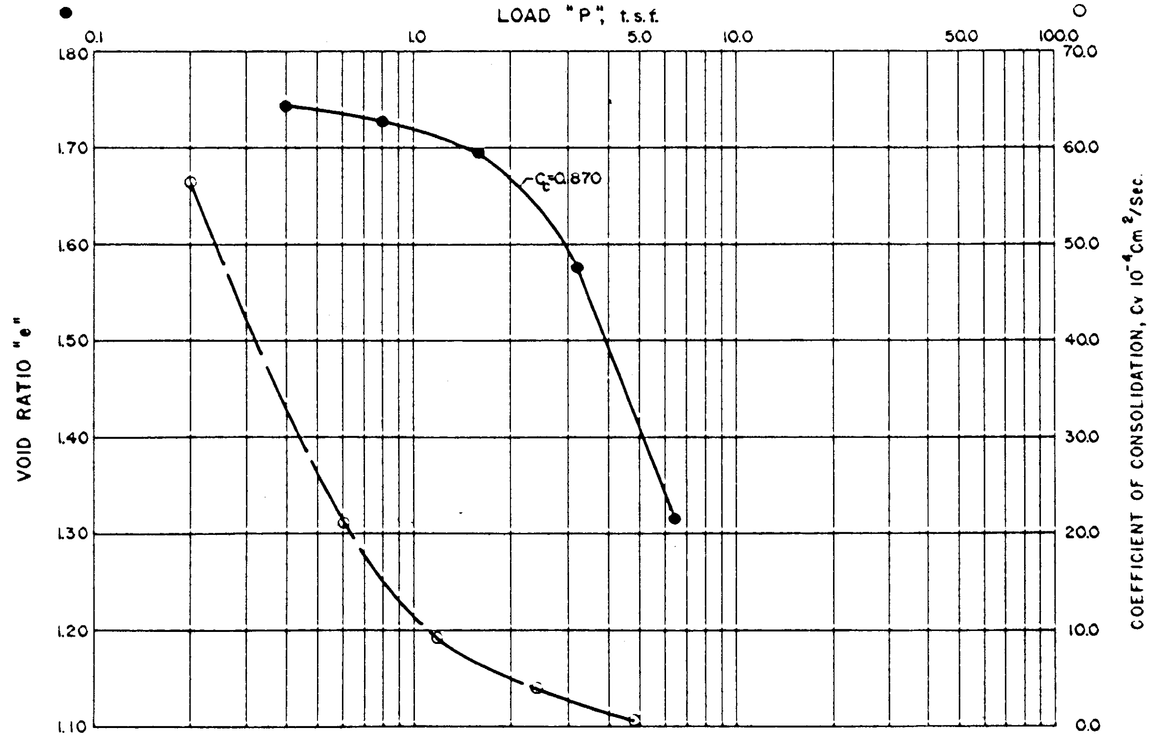

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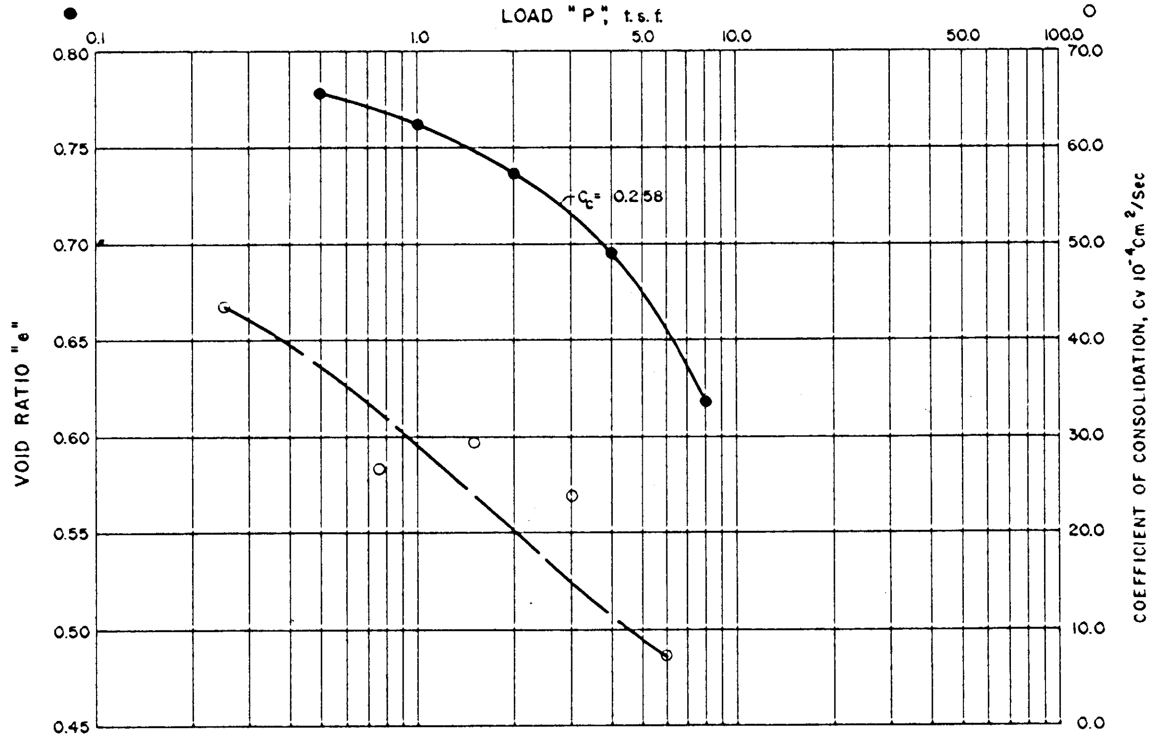

SAMPLE NO. 17 - C, EL. - 94.8

For General Notes
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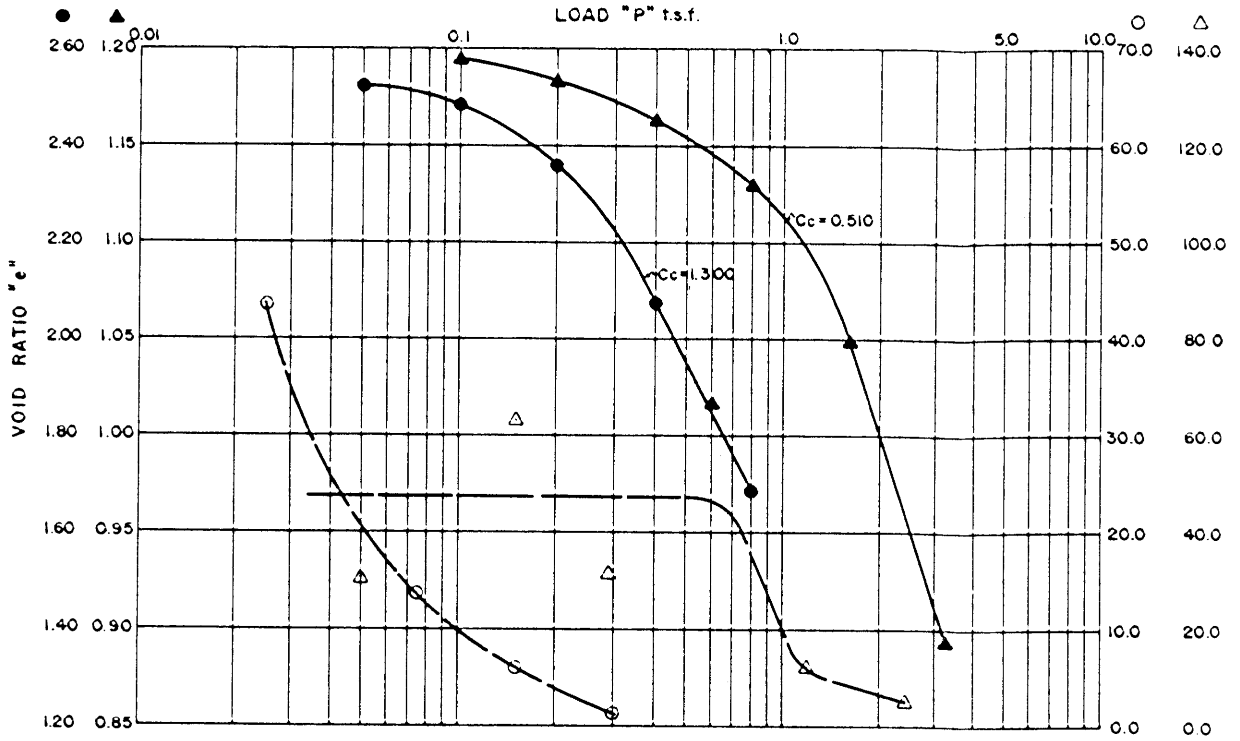

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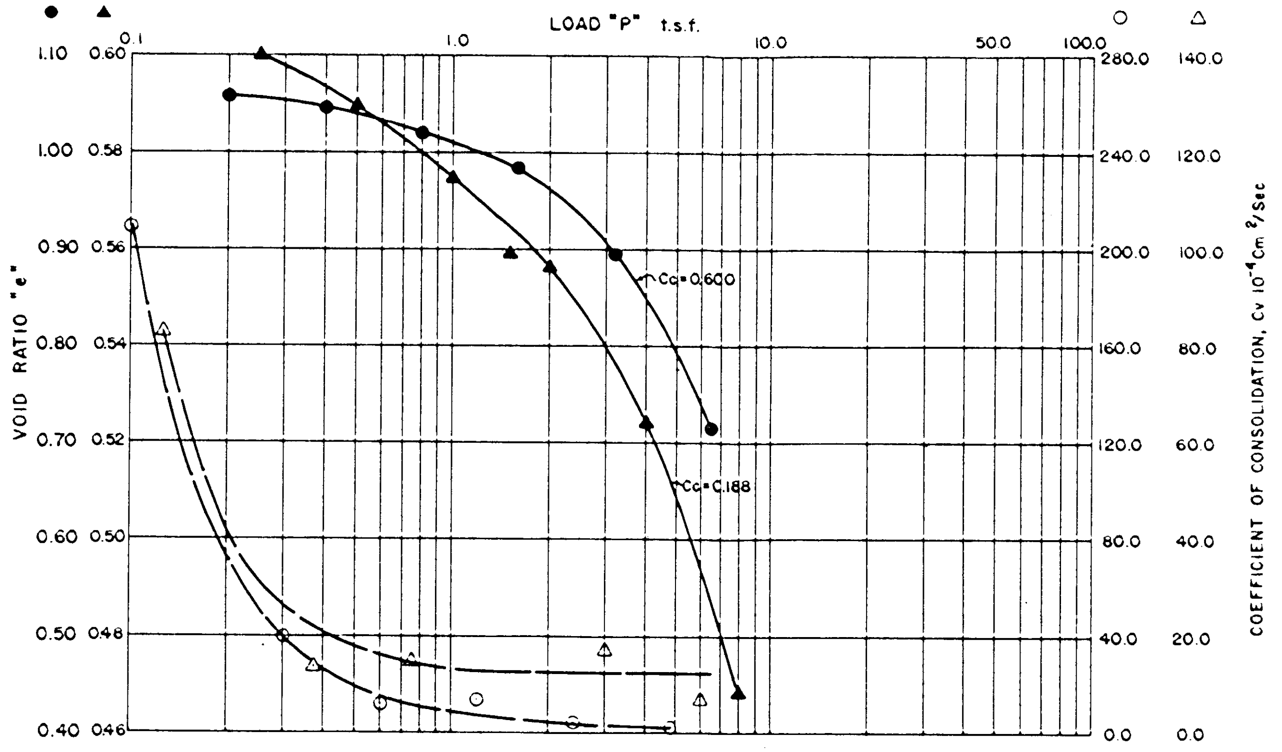
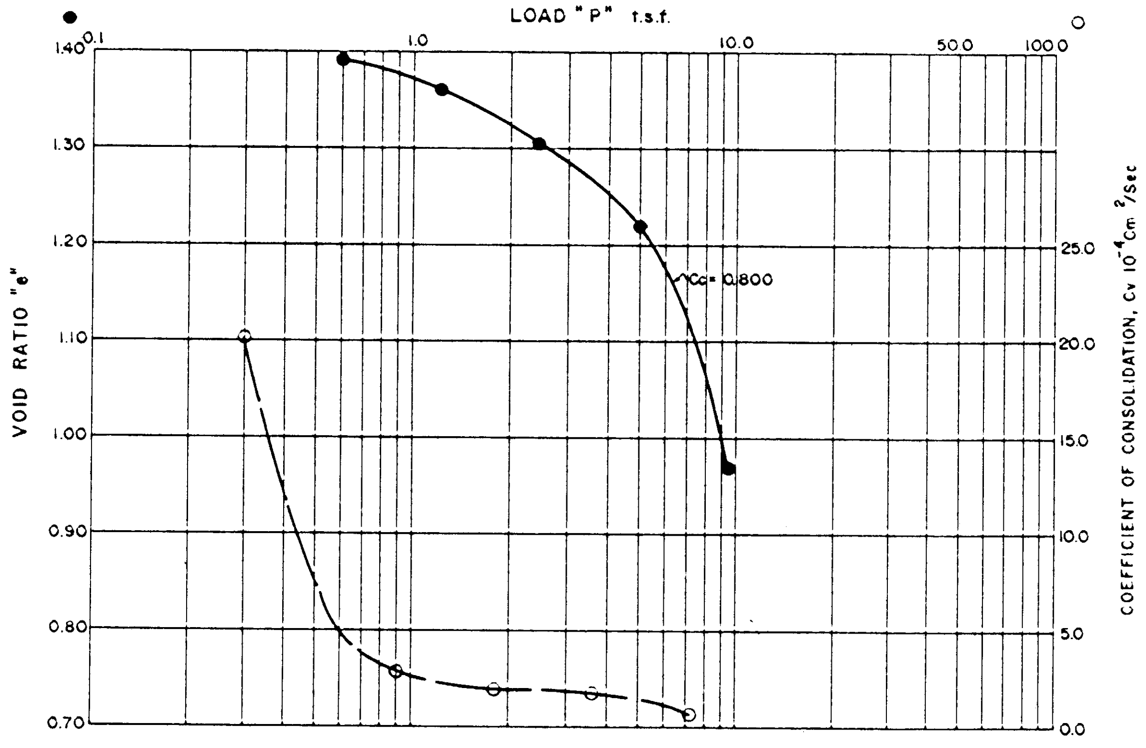


○ SAMPLE NO. 8-C EL. -58.6 △ SAMPLE NO. 10-C EL. -67.1



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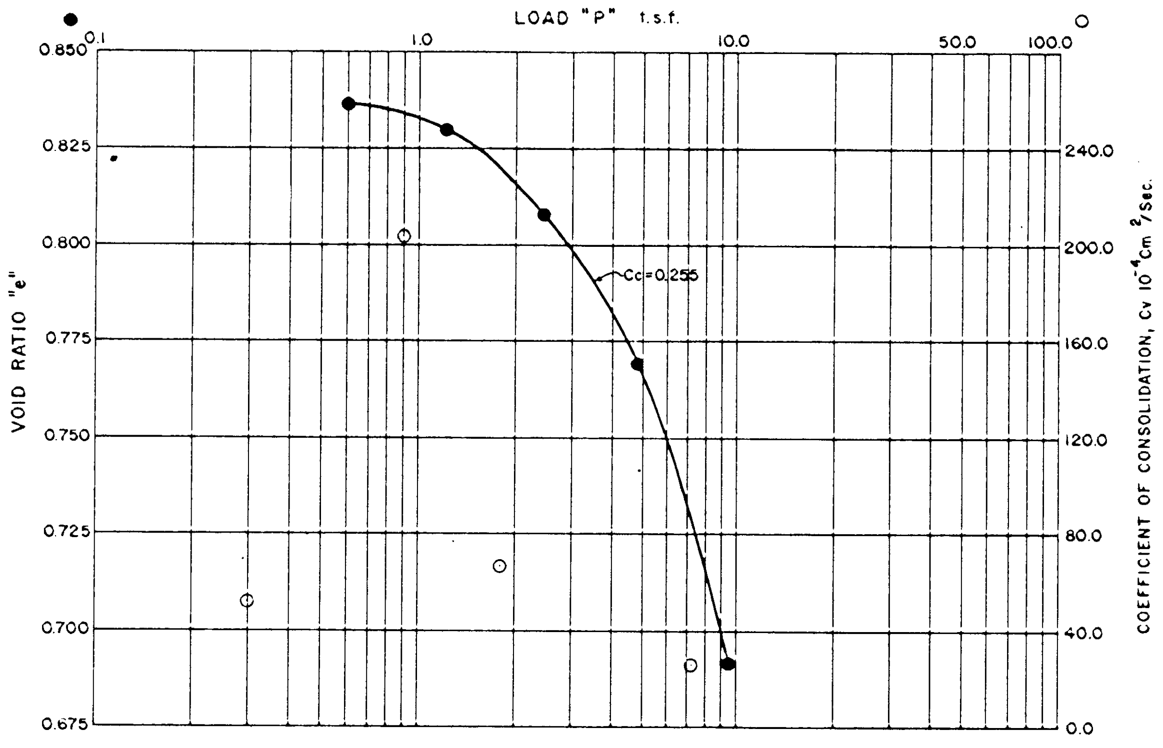

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For General Notes
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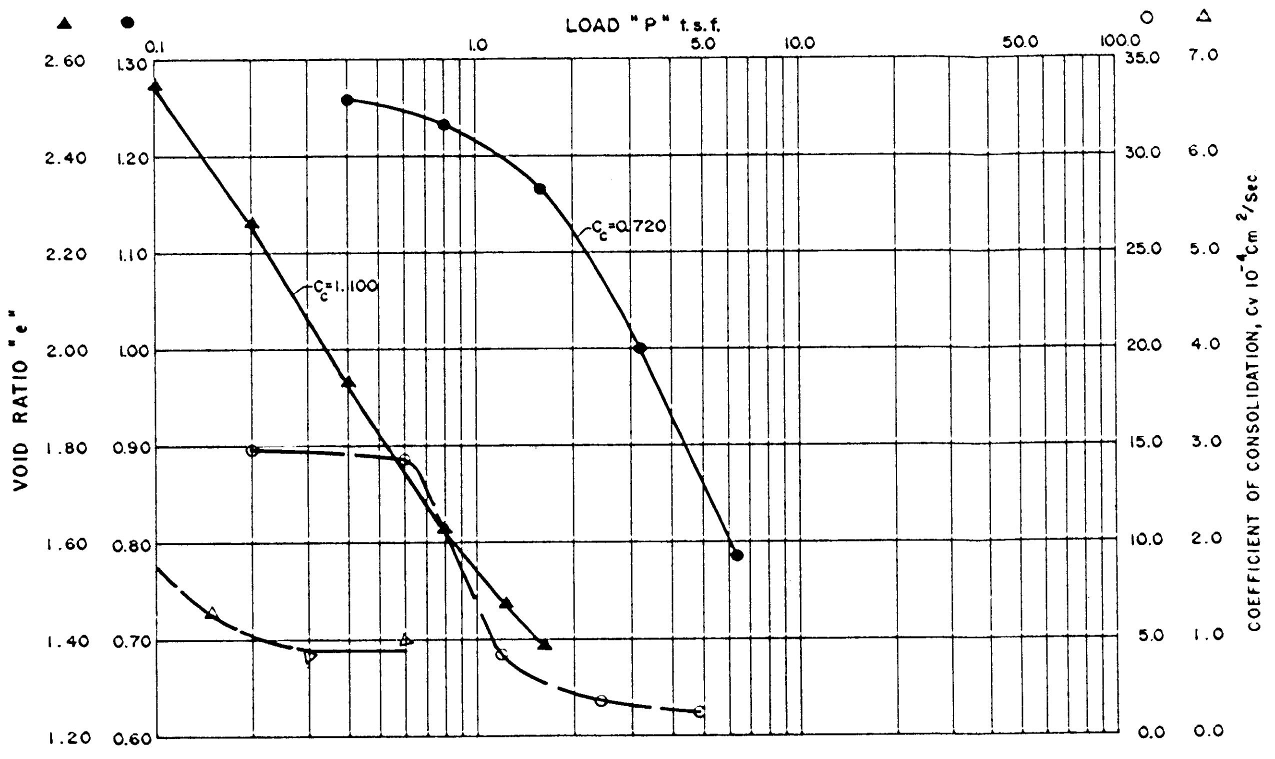

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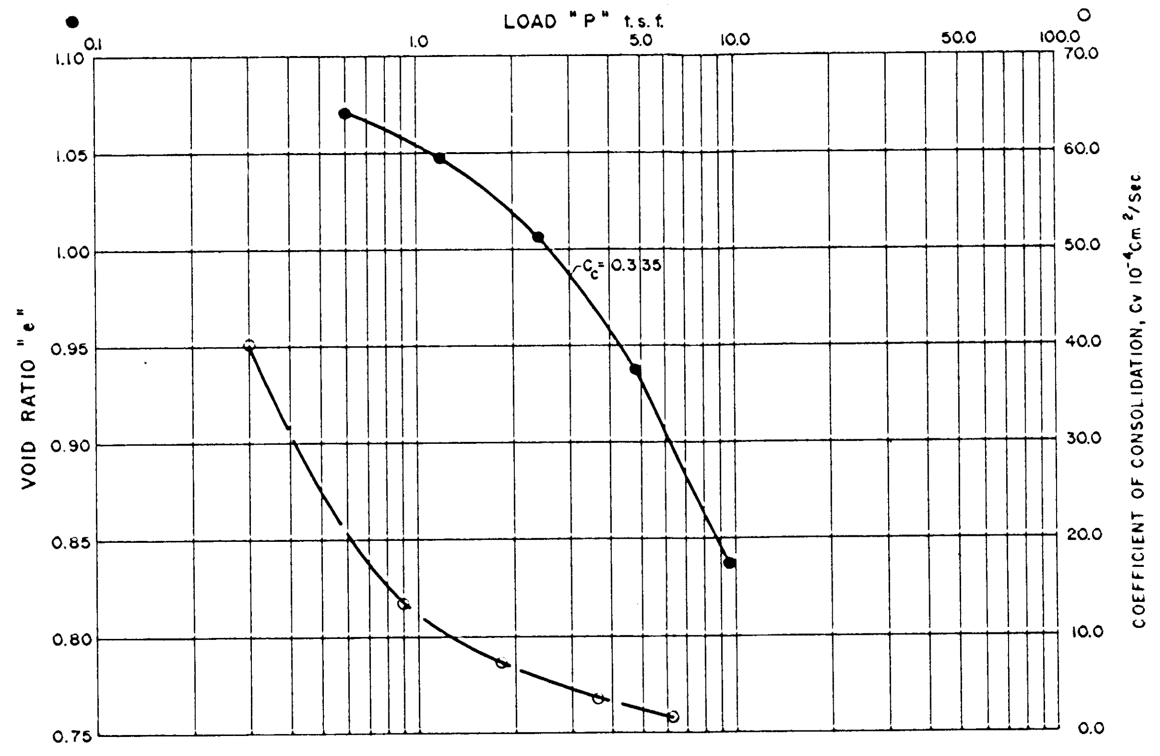

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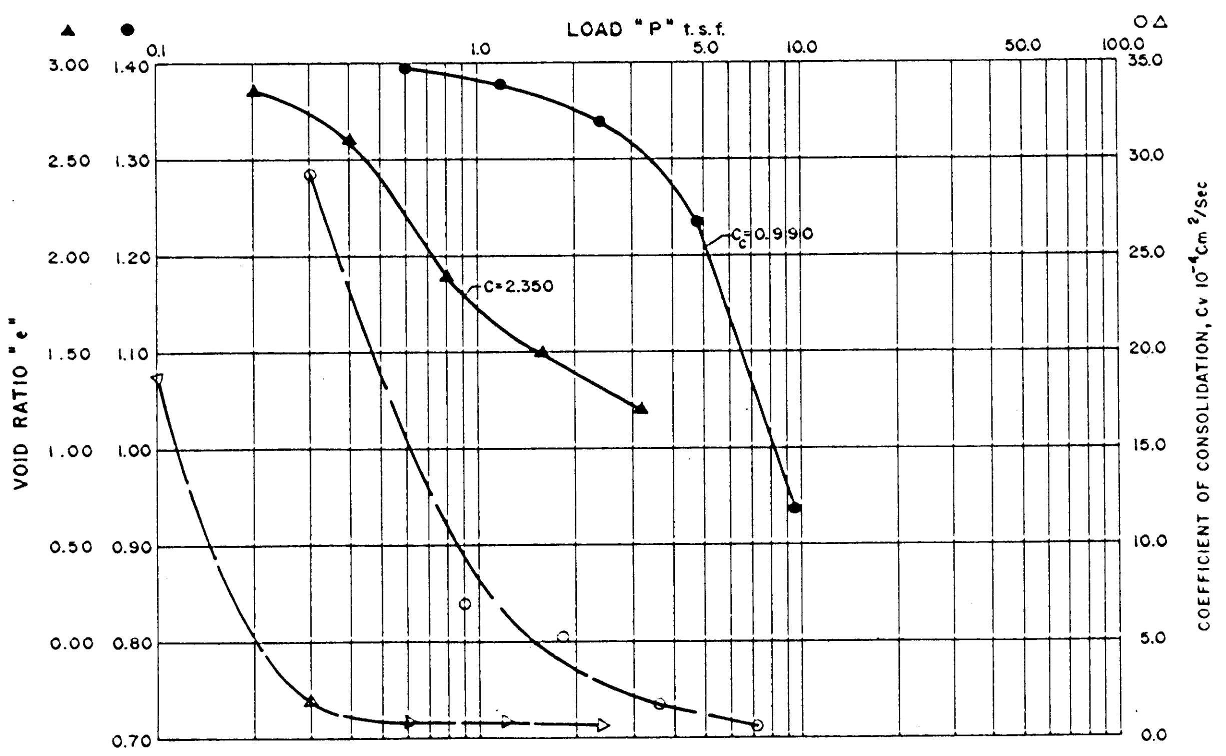

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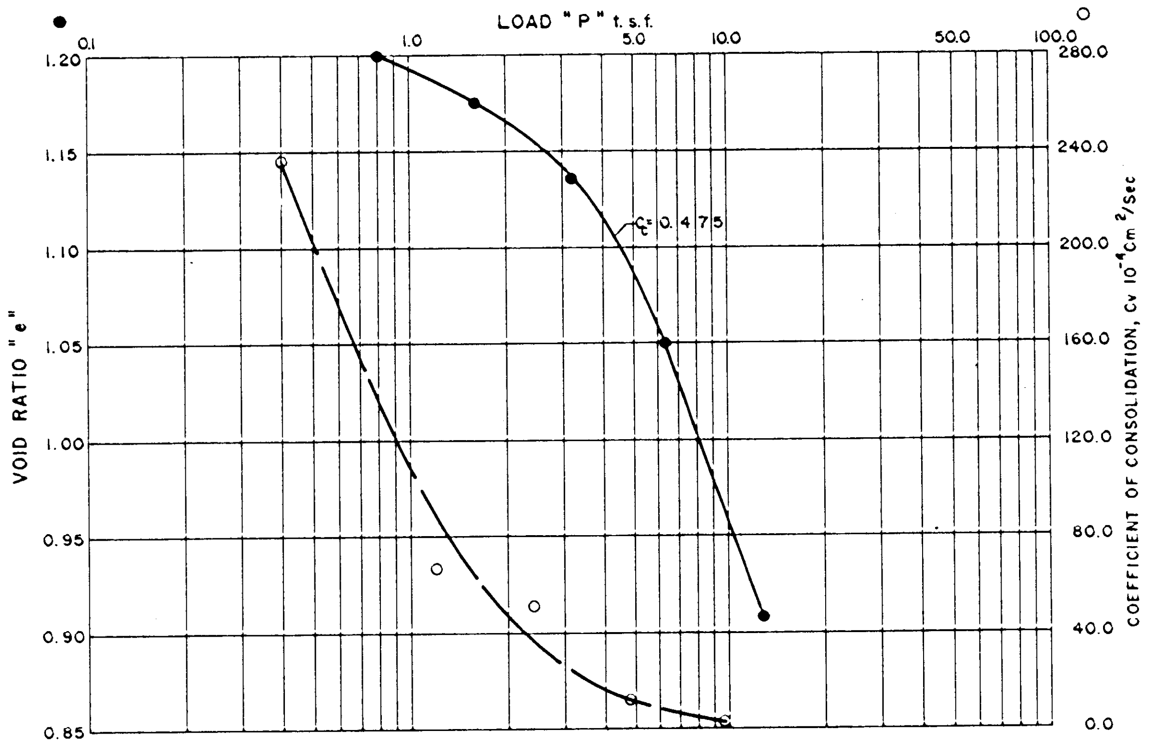

SAMPLE NO. 16-B, EL.-92.8

For General Notes
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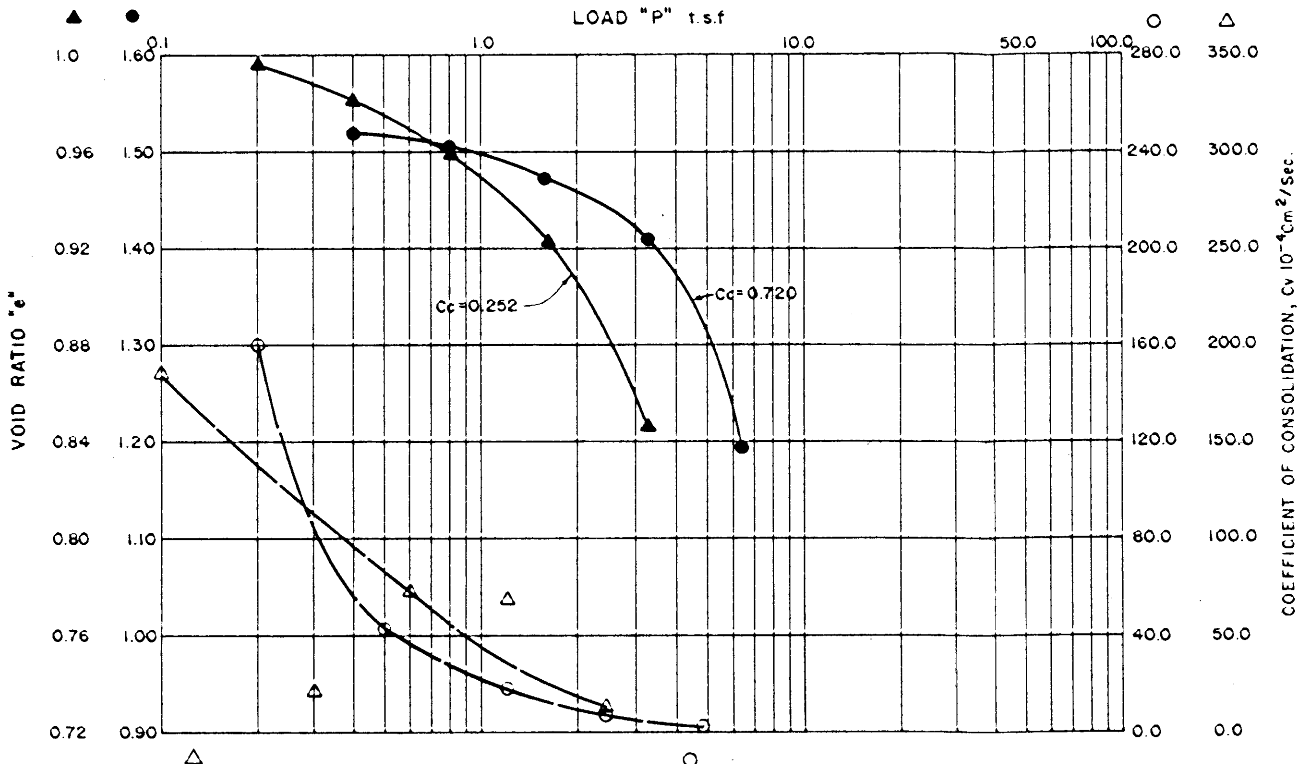

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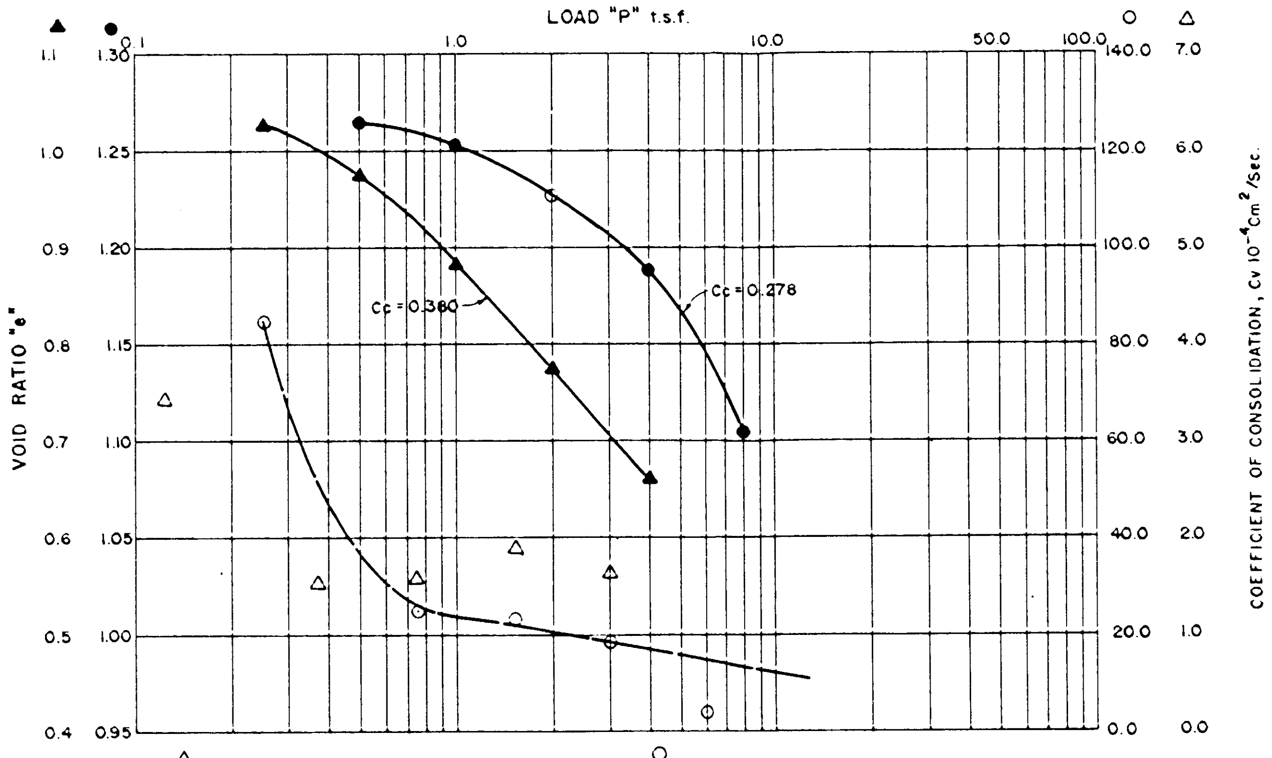
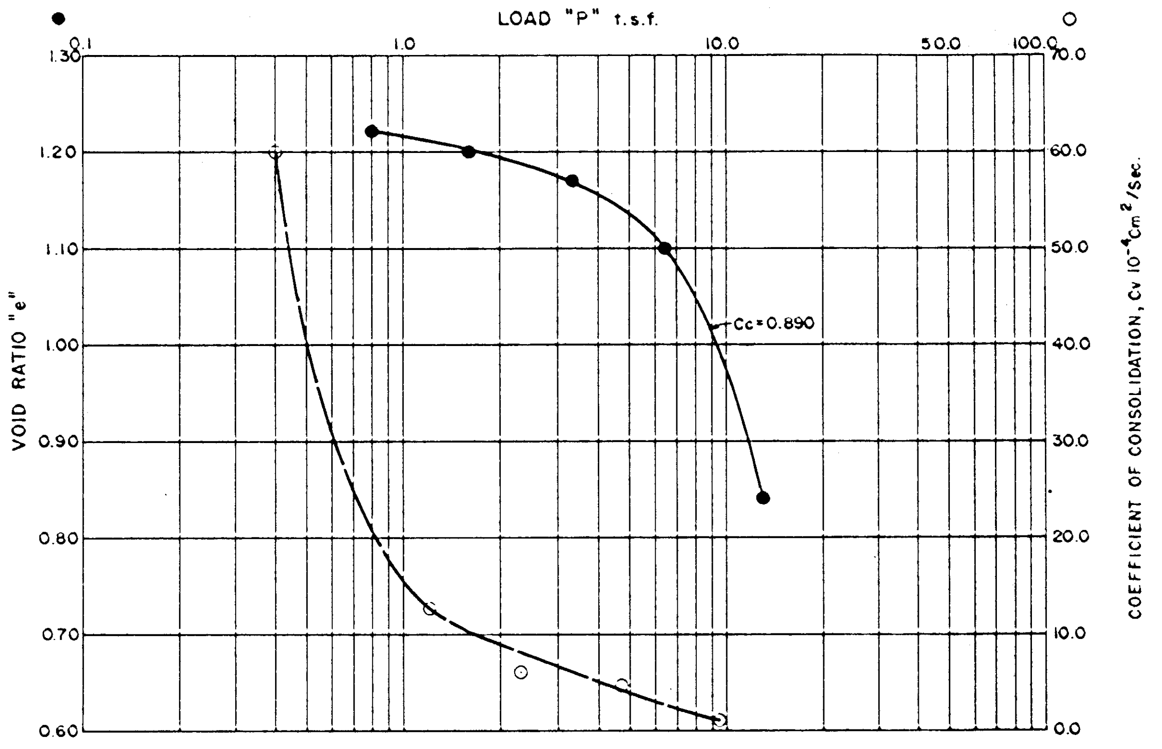


SAMPLE NO. 9-B, EL.-65.9 SAMPLE NO. 21-B, EL.-114.0



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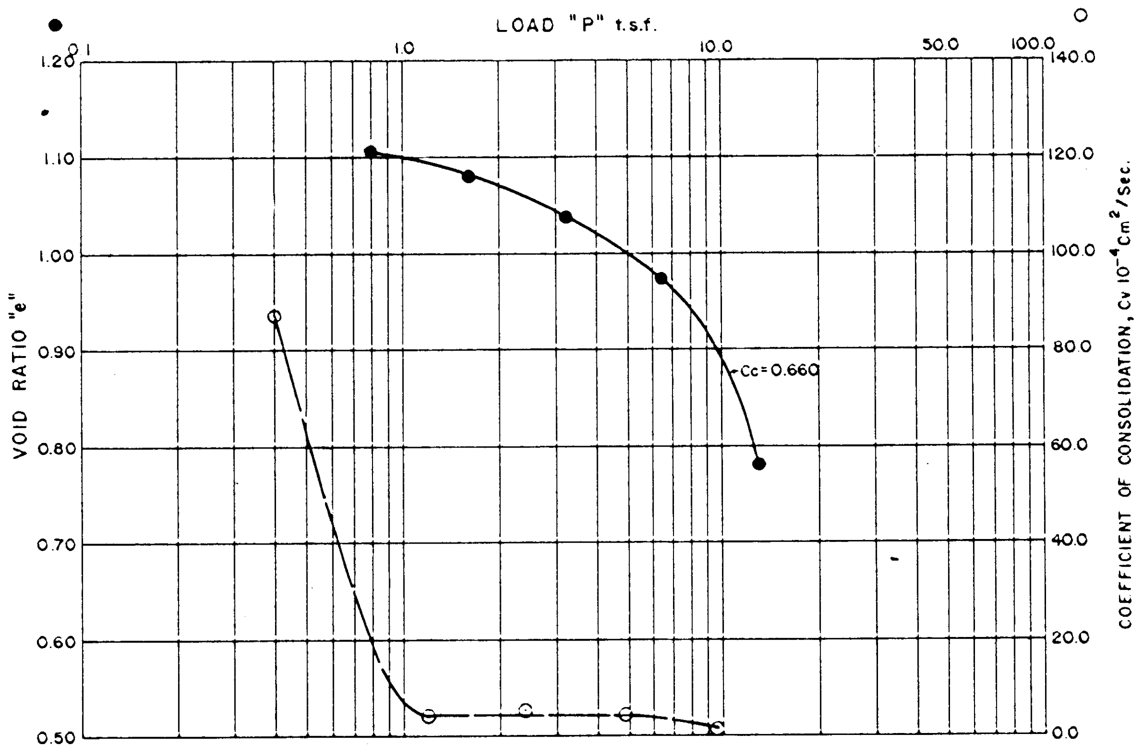

SAMLE NO. 34-D EL. -167.9

For General Notes
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U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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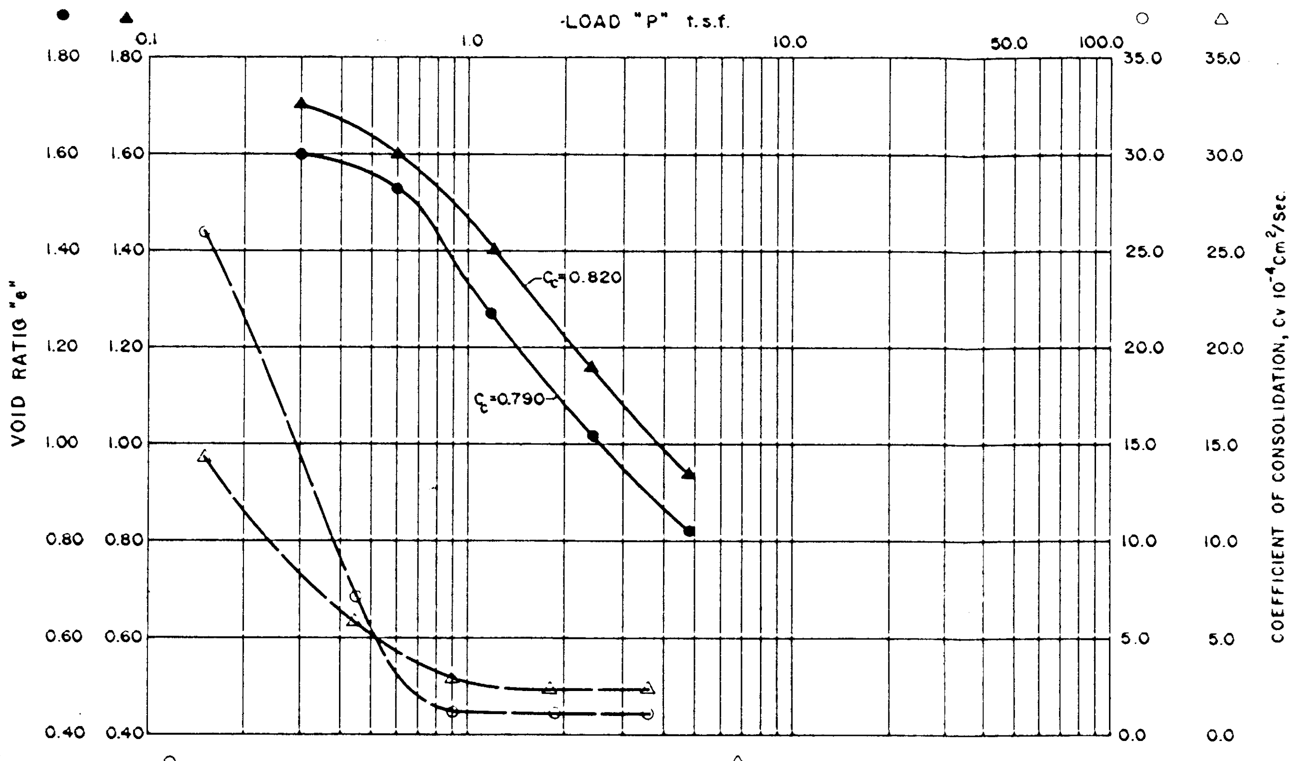

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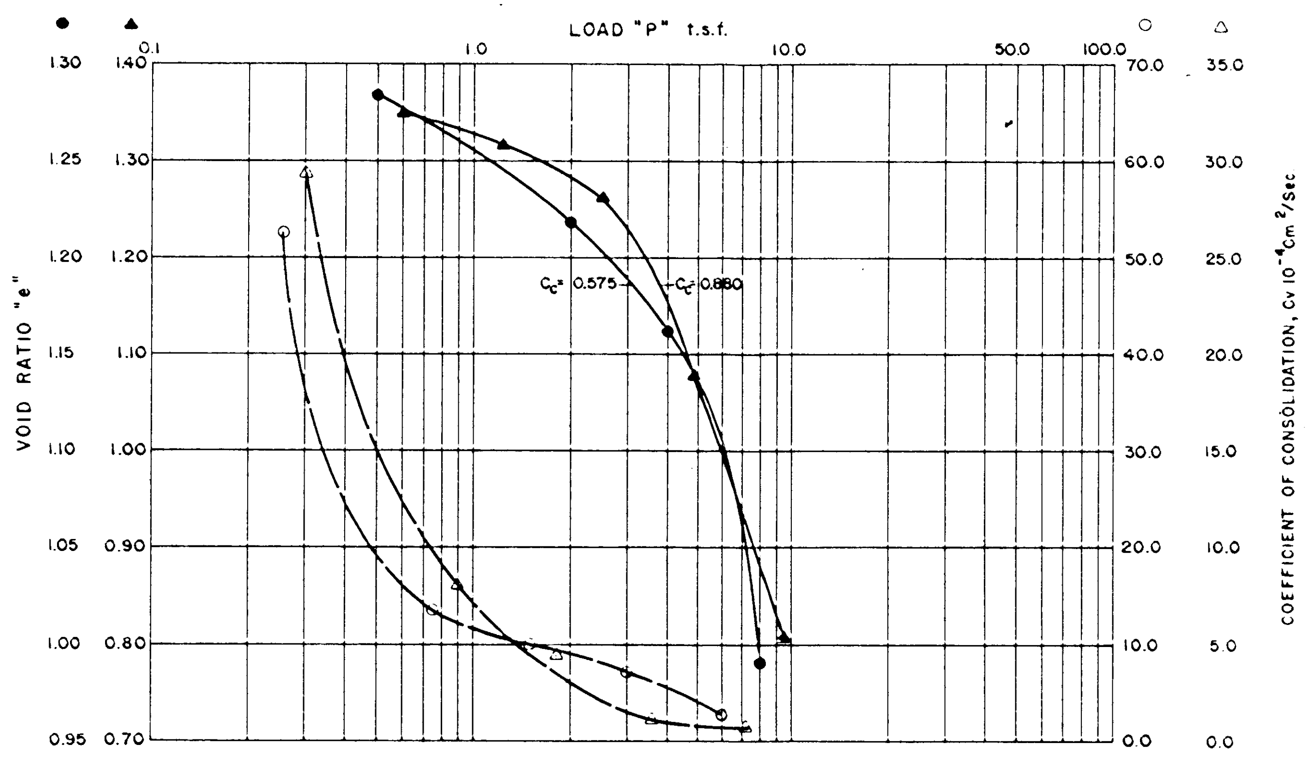
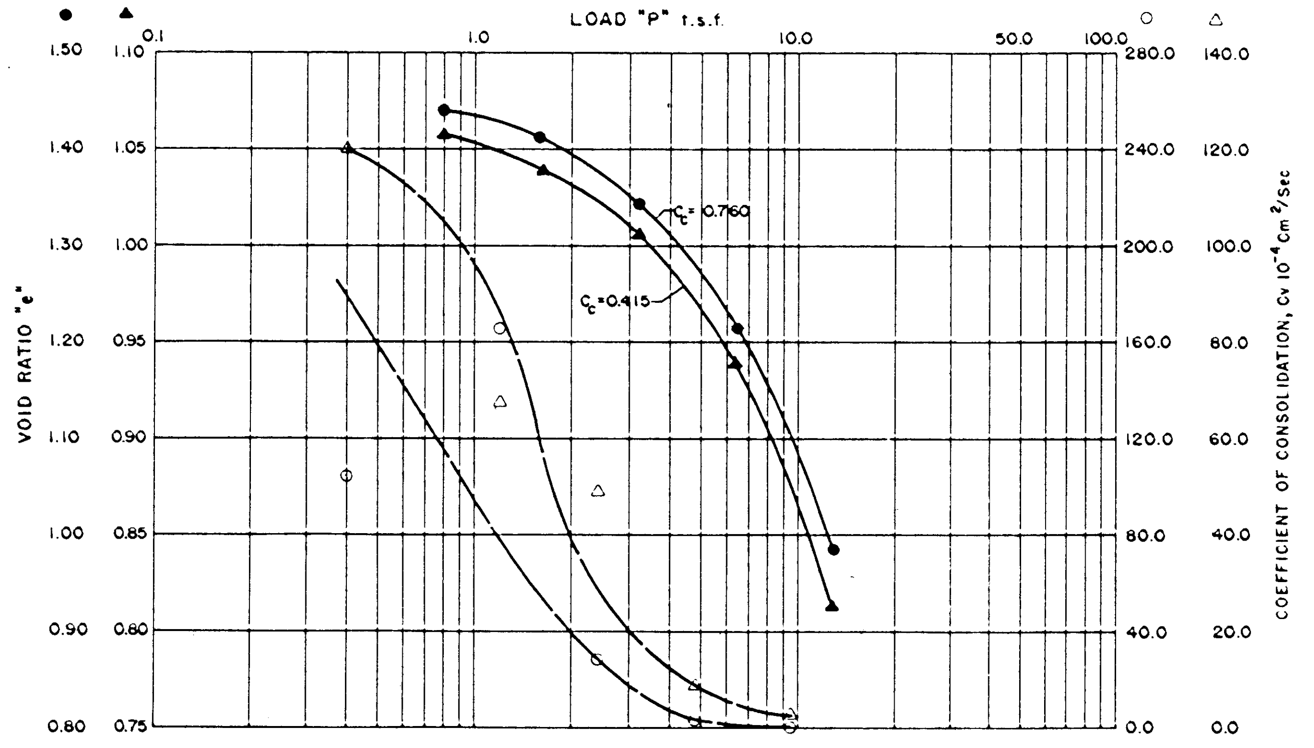
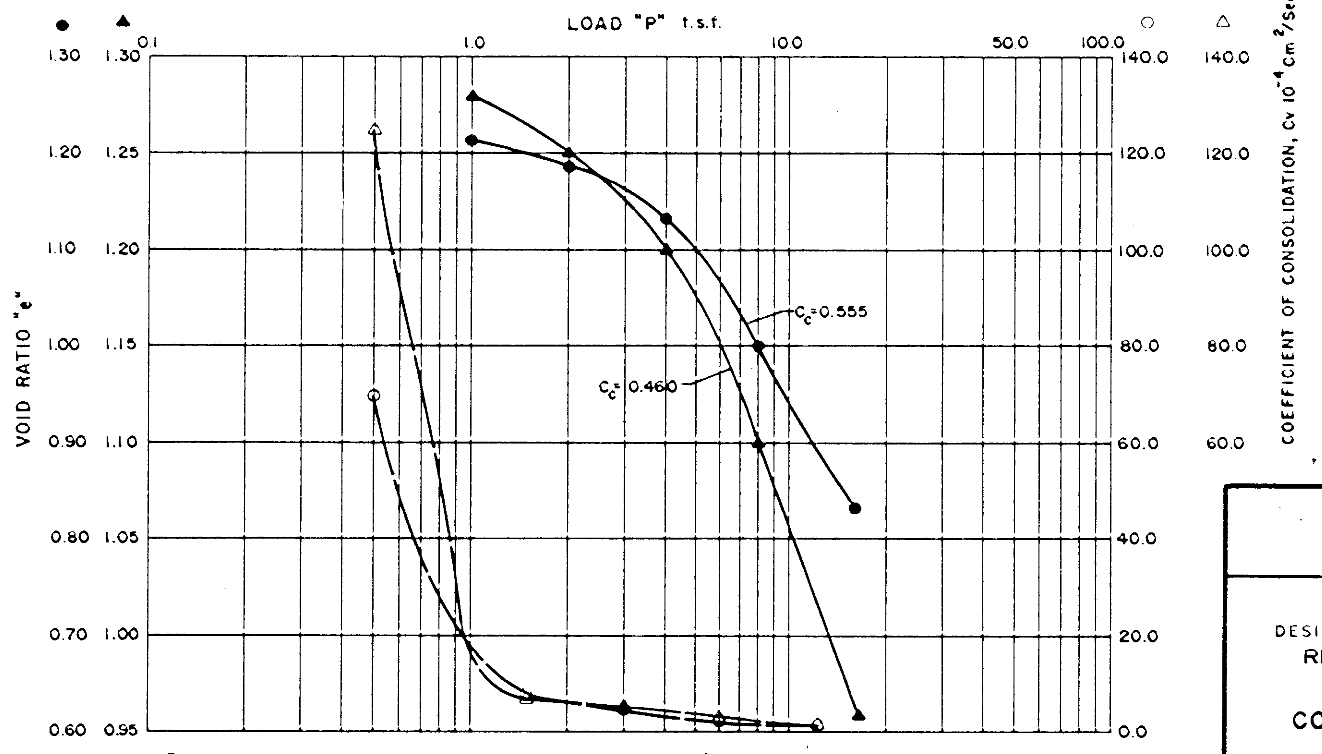


○ SAMPLE NO. 10-D1, EL.-63.2 △ SAMPLE NO. 14-D, EL.-79.3



○ SAMPLE NO. 18-C, EL.-94.2 △ SAMPLE NO. 23-D, EL.-115.7



○ SAMPLE NO. 31-B, EL.-145.8 △ SAMPLE NO. 36-B, EL.-165.4

For General Notes See Plate A-2

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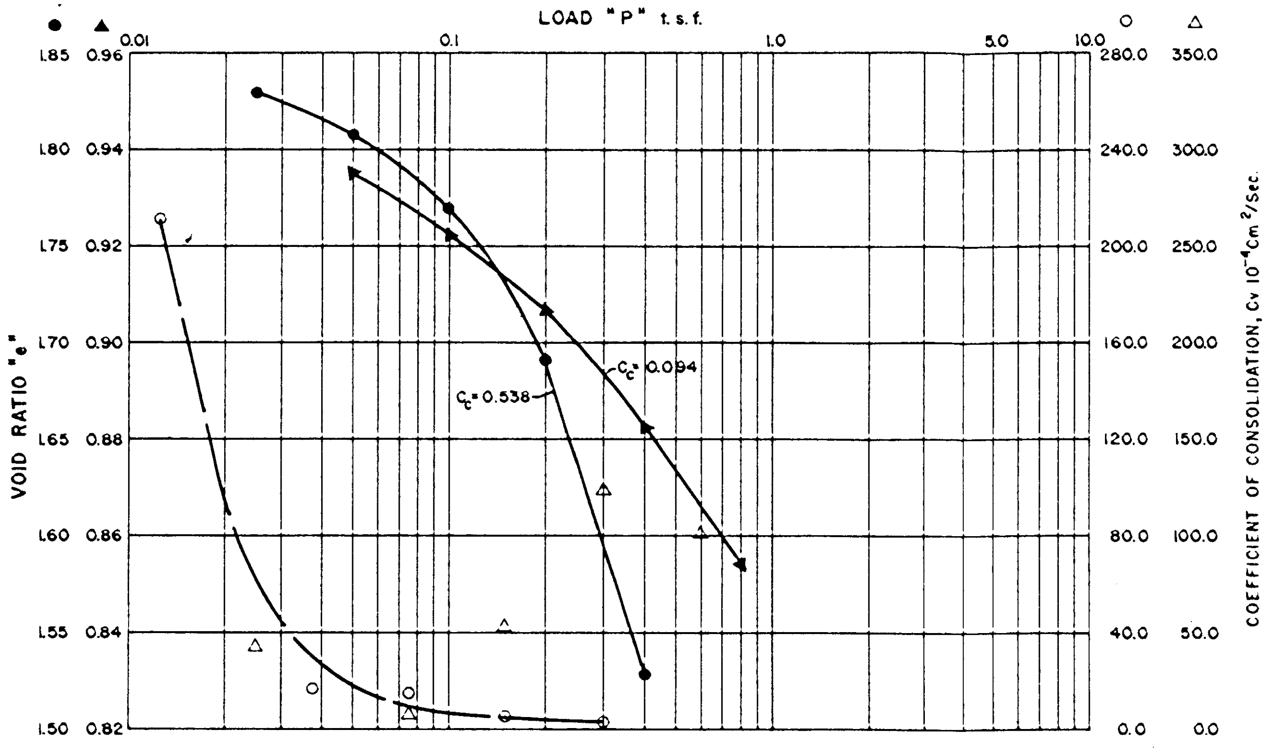
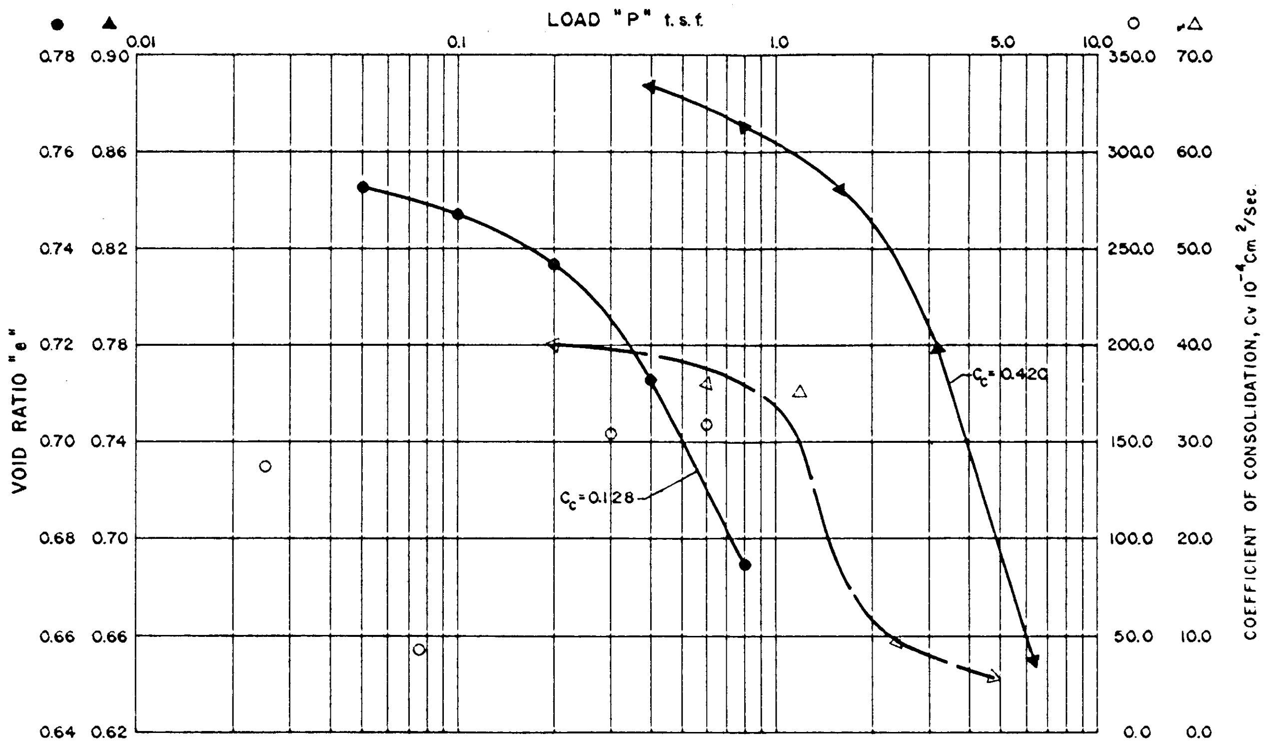
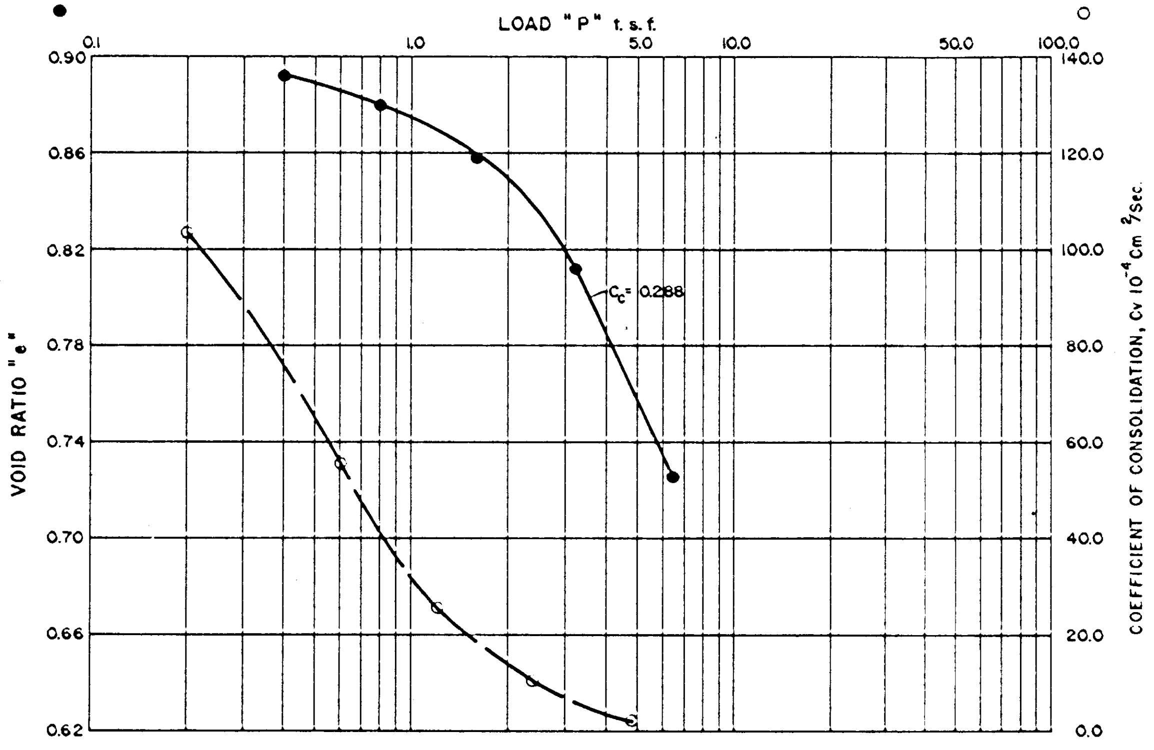
DATE JULY 1972 FILE NO. H-2-24417



SAMPLE NO. 1-B_1, EL. -1.4 SAMPLE NO. 2-C_2, EL. -6.7



SAMPLE NO. 4-B_2, EL. -14.2 SAMPLE NO. 15-C, EL. -59.2



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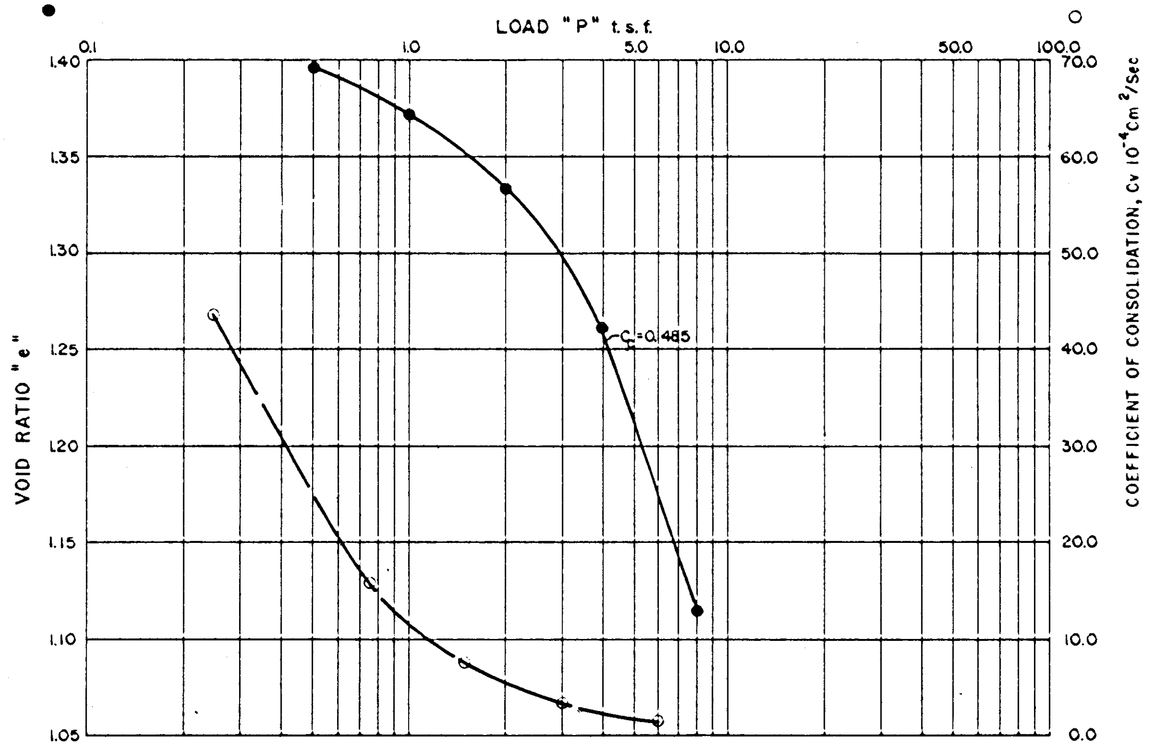

SAMPLE NO. 21-B, EL. -81.6

For General Notes
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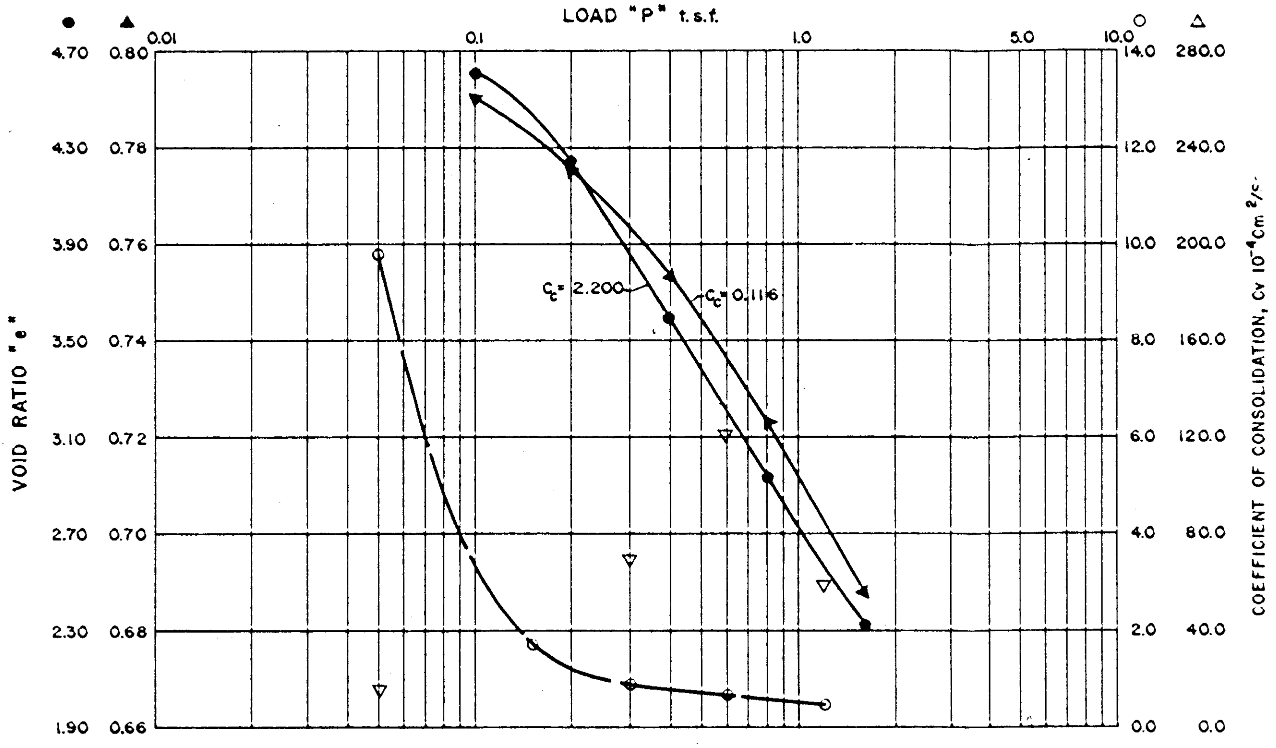
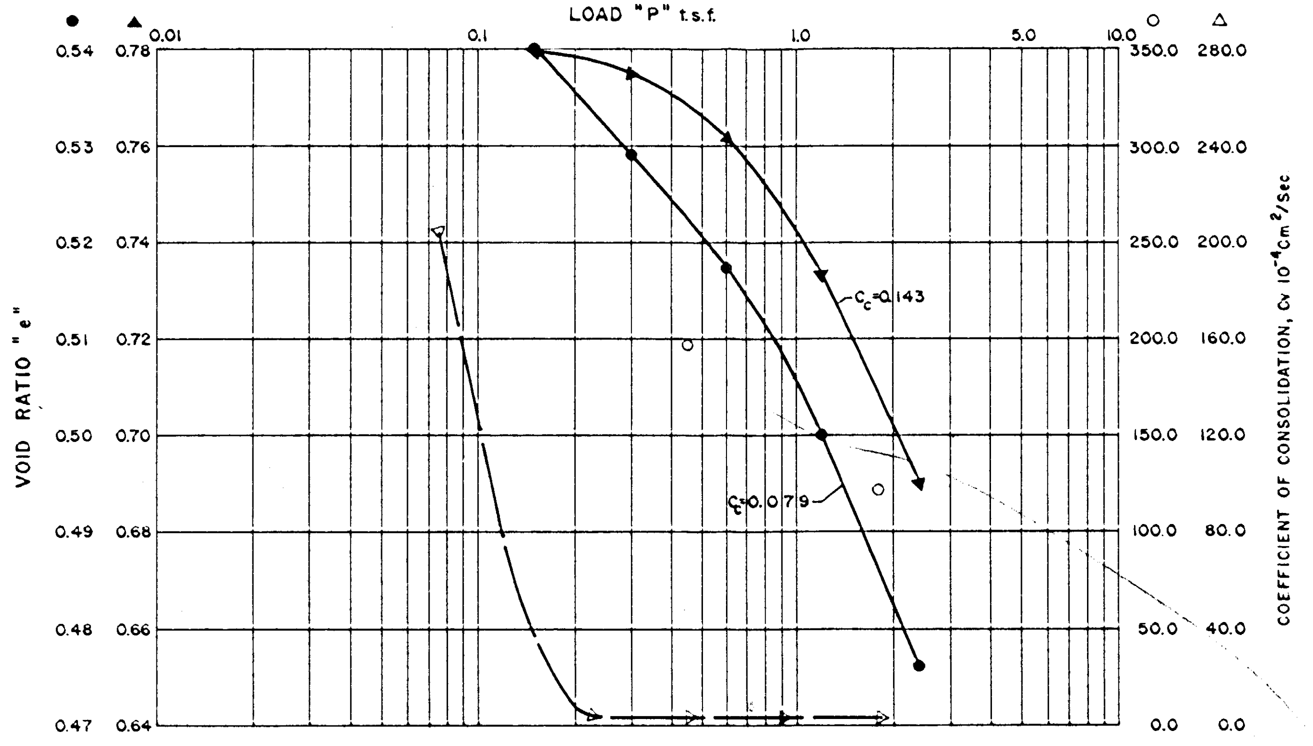


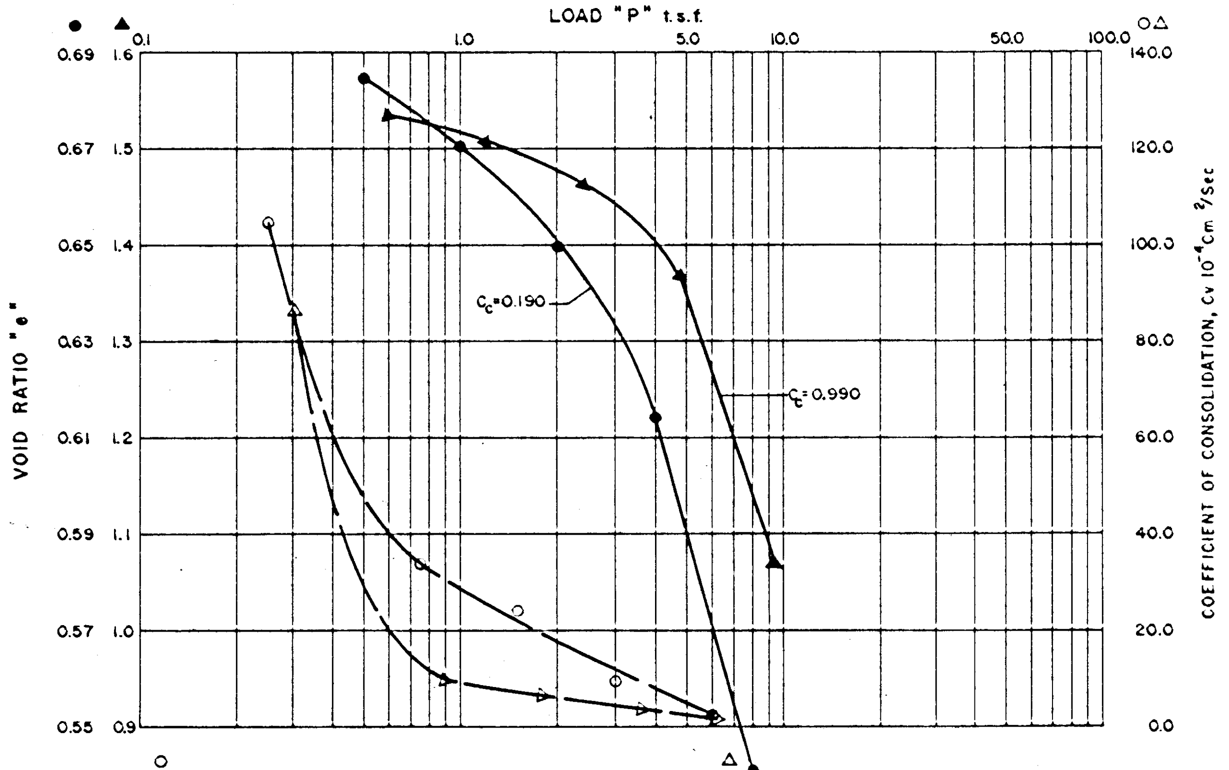
○ SAMPLE NO. 2-A, EL. -1.6

△ SAMPLE NO. 3-C, EL. -6.9



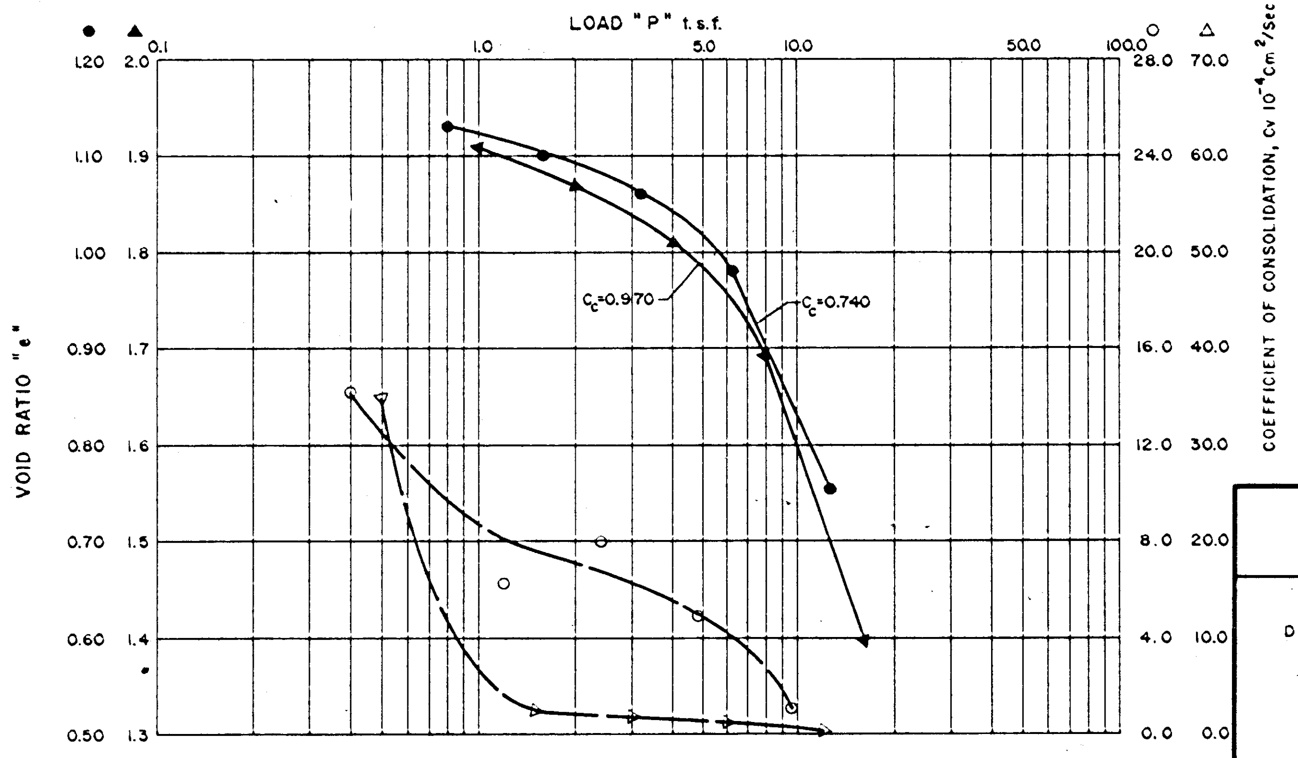
○ SAMPLE NO. 5-B, EL. -14.5

△ SAMPLE NO. 6-A, EL. -17.4



○ SAMPLE NO. 19-C, EL. -71.3

△ SAMPLE NO. 23-A, EL. -85.2



○ SAMPLE NO. 34-A, EL. -129.5

△ SAMPLE NO. 45-A, EL. -173.6

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PLATE A-29

ASSUMPTIONS

(1) STEEL SHEET PILE CIRCULAR TYPE COFFERDAM IS TO BE USED.

(2) COFFERDAM TIP SHALL CUT INTO THE IMPERMEABLE SOIL LAYER TO PREVENT THE SEEPAGE.

(3) CELL FILLING MATERIAL TO BE SAND FROM THE BORROW AREA AS INDICATED ON PLATE

(4) BERM MATERIAL TO BE FREE DRAINING GRANULAR MATERIAL

(5) COULOMB'S THEORY IS USED TO FIND EARTH PRESSURE AGAINST THE COFFERDAM.

(6) 4.0" DIA. WEEP HOLES ARE PROVIDED AT ELEVATION -13.0 ON INBOARD SIDE OF CELL.

(7) HIGHEST WATER ELEVATION = +6.0, AND NORMAL WATER ELEVATION = +1.5

(8) CUMMING'S METHOD IS USED TO DESIGN THE CELL AGAINST HORIZONTAL SHEAR.

DESIGN DATA

D = DIAMETER OF CELL = 63.66 FT.

B = EFFECTIVE WIDTH OF CELL = 55.78 FT.

2L = DISTANCE BETWEEN CELLS = 68.72 FT.

γ = WEIGHT OF SATURATED SAND FILL = 122.5 #/FT.^{3}

γ_{wet} = WEIGHT OF WET SAND = 120.0 #/FT.^{3}

γ' = WEIGHT OF SUBMERGED SAND FILL = 60.0 #/FT.^{3}

t = ALLOWABLE INTERLOCK TENSION = 8,000 #/LIN. INCH

f = CO-EFFICIENT OF FRICTION, STEEL ON STEEL

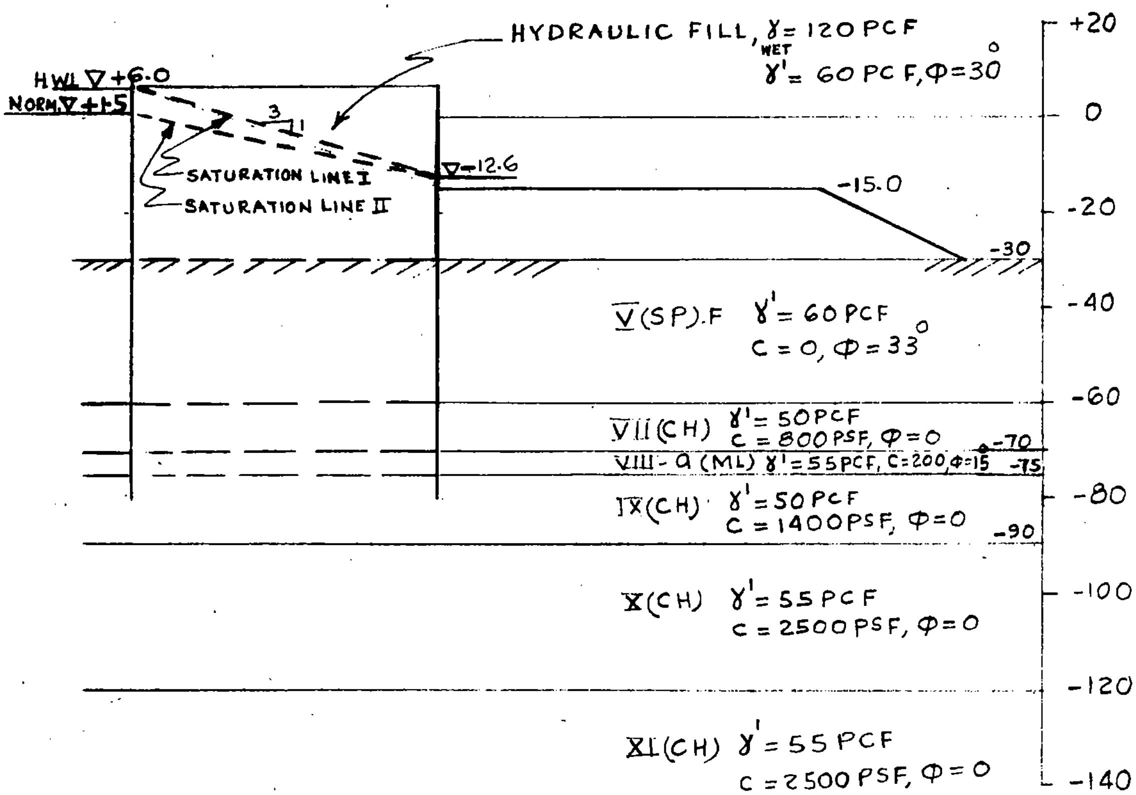
INTERLOCK = 0.3

γ_w = UNIT WEIGHT OF WATER = 62.5 #/FT.^{2}

ϕ = ANGLE OF INTERNAL FRICTION OF FILL = 30°

TANδ = CO-EFFICIENT OF FRICTION - SOIL ON STEEL = 0.25

K_A = CO-EFFICIENT OF ACTIVE EARTH PRESSURE



γ' = SUBMERGED WEIGHT OF SOIL

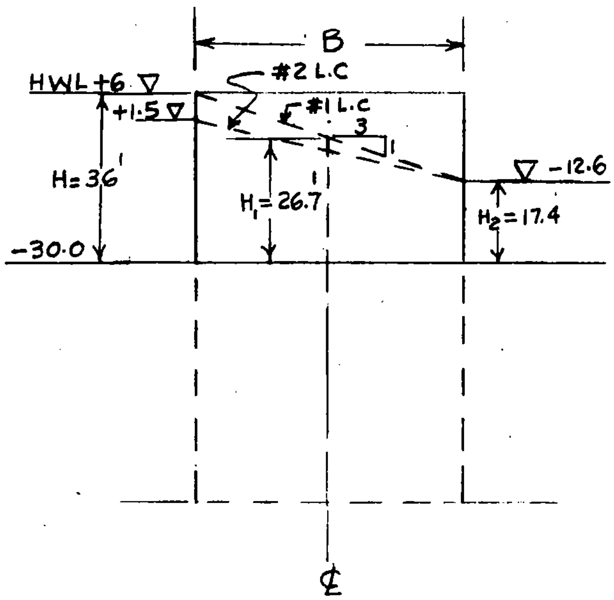
C = COHESION (FROM CONSOLIDATED UNDRAINED 'Q' TEST)

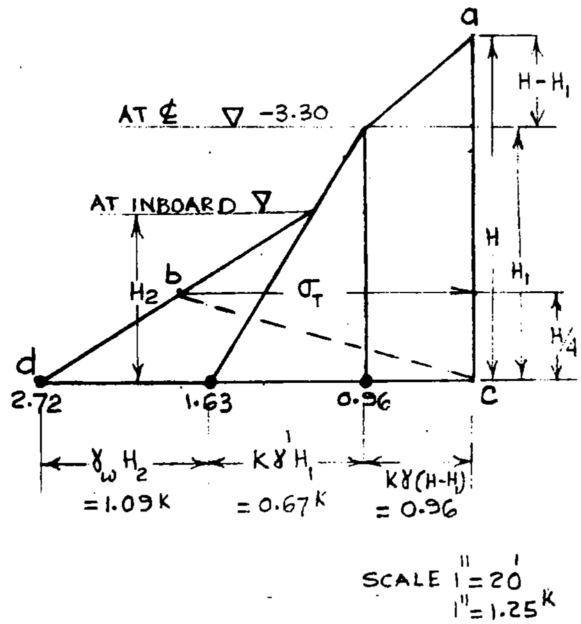
Φ = ANGLE OF INTERNAL FRICTION (FROM CONSOLIDATED UNDRAINED 'Q' TEST)

LOADING CONDITIONS

L.C #1. HIGHEST WATER ELEVATION = +6.0. SINCE WEEPHOLES ARE PROVIDED AT EL. -13.0, SO CELL FILL IS ASSUMED TO BE SATURATED FROM THE TOP OF COFFERDAM AT ONE END TO THE ELEVTION -12.6 ON THE OTHER END i.e 1 ON 3 SLOPE.

L.C #2. NORMAL WATER ELEVATION = +1.5, SO CELL FILL IS ASSUMED TO BE SATURATED FROM EL. +1.5 AT ONE END TO THE ELEVATION -12.6 ON THE OTHER END





LATERAL PRESSURE DIAGRAM

LOADING CONDITION #1 & #2

L.C #1.

H = 36'

H_1 = 26.7'

H_2 = 17.4'

WEIGHT OF CELL FILL = $\bar{W} = \frac{(36-17.4)\,55.8}{2} \times 0.12 + \frac{(17.4+36)\,55.8}{2} \times .06$

$\bar{W} = 151.6^K$

LATERAL PRESSURE ON OUTSIDE OF CELL = $P_w = \frac{1}{2} \times .0625 \times 36^2$

$= 40.5^{K/LFT}$

MOMENT DUE TO P_w AT EL -30, $M' = 40.5 \times \frac{36}{3} = 486^{'K}$

L.C #2

H = 36.0'

H_1 = 25.9'

H_2 = 17.4'

WEIGHT OF CELL FILL = $W = \frac{1}{2} \times 55.8\,(18.6+4.5) \times 0.12$

$+ \frac{1}{2} \times 55.8\,(17.4+31.5) \times 0.06$

$= 159.3^K$

MOMENT DUE TO WATER EL. +1.5 = M

$M = \frac{1}{2} \times .0625 \times (31.5)^2 \times \frac{31.5}{3}$

$= 325.5^{'K}$

BEARING CAPACITY

A. LOADING CONDITION #1

$$\text{FACTOR OF SAFETY} = \frac{q_{ult.}}{q_{max.}} = \frac{C N_c + \frac{1}{2}\gamma B N_\gamma}{\frac{6M}{B^2} + \frac{\bar{W}}{B}} > \begin{matrix} 3.0 \text{ FOR CLAY} \\ 2.0 \text{ FOR SAND} \end{matrix}$$

REF. 2, NAVFAC DM-7, P 7-11-6, FIG. 11-5, FOR 2 LAYERS

COHESIVE SOIL.

(1) ASSUME TIP EL. -65

FOR LAYER VIII - CH, $C = 800 \text{ PSF} = C_1$

IX - CH, $C = 1400 \text{ PSF} = C_2$

$$\frac{C_2}{C_1} = \frac{1.4}{0.8} = 1.75, \quad \frac{T}{B} = \frac{10}{55.8} = 0.18, \quad N_c = 8.8$$

(T = THICKNESS OF SOIL LAYER BELOW THE TIP = 75-65 = 10')

$$q_{ult} = C N_c = 0.8 \times 8.8 = 7.0^{KSF}$$

$$q_{max.} = \frac{6M}{B^2} + \frac{\bar{W}}{B} = \frac{6 \times 486}{(55.8)^2} + \frac{151.6}{55.8} = 0.93 + 2.72 = 3.65^{KSF}$$

$$F.S = \frac{7.0}{3.65} = 1.92 < 3.0 \quad \text{N.G}$$

(2) ASSUME TIP EL. -80

FOR LAYER IX - $C = 1{,}400 \text{ PSF} = C_1$

X - $C = 2{,}000 \text{ PSF} = C_2$

$$\frac{C_2}{C_1} = \frac{2.0}{1.4} = 1.43, \quad \frac{T}{B} = \frac{10}{55.8} = 0.18, \quad N_c = 7.5$$

$$F.S = \frac{1.4 \times 7.5}{3.65} = \frac{10.5}{3.65} = 2.88 < 3.0,$$ SO PROVIDE BERM

TO GET $F.S \doteq 3.0$

TRY BERM WITH TOP ELE. -15.0

$$F.S = \frac{q_{ult.} + \gamma' D}{q_{max}}$$ WHERE γ' = SUBMERGED UNIT WT. OF BERM = 60PCF

D = REQUIRED HEIGHT OF THE BERM = 15' (ASSUMED)

MOMENT DUE TO BERM AT EL. -15 = $\frac{1}{2} \times .06 \times 15^2 \times \frac{15}{3} = 33.75$ 1-K

NET MOMENT = 486 - 33.75 = 452.25 1-K

$q_{MAX.} = \frac{6M}{B^2} + \frac{W}{B} = \frac{6 \times 452.25}{(55.8)^2} + \frac{151.6}{55.8} = 0.87 + 2.72 = 3.$[illegible]

$F.S = \frac{10.5 + .06 \times 15}{3.59} = \underline{3.18 > 3.0}$ O.K, SO PROVIDE BERM AT EL. -15.0

TOP WIDTH OF THE BERM FOR FULLY MOBILIZED

PASSIVE WEDGE. $= \frac{(80-60)}{\tan 45^\circ} + \frac{(60-30)}{\tan 28.5^\circ} = 20' + 62.5' = 82.5'$

SAY 80' WIDE.

PROVIDE 1 ON 3 SLOPE.

<u>BEARING CAPACITY FOR LOADING CONDITION #2.</u>

TIP ELE. -80.0 WITH BERM AT -15.0.

$q_{ult.} = C N_c = 1.40 \times 7.5 = \underline{10.5 \text{ K/SQ FT.}}$ NET MOMENT M = 325.5 - 33.75 = 291.75

$q_{max.} = \frac{6M}{B^2} + \frac{W}{B} = \frac{6 \times 291.75}{(55.8)^2} + \frac{159.3}{55.8}$

$= 0.56 + 2.85 = \underline{3.41}$

$\gamma' D$ (DUE TO BERM) = .06 × 15 = 0.90

FACTOR OF SAFETY $= \frac{10.5 + 0.90}{3.41}$

$= 3.34 > 3.0$ O.K

FOR FURTHER ANALYSIS, LOADING CONDITION #1 WHICH IS MORE CRITICAL WILL BE USED.

INTERLOCK TENSION

$$t_{MAX.} = \frac{\sigma_t (L \sec\alpha)}{12} \text{ K/IN}, \quad \alpha = 45^\circ$$

$$= \frac{\sigma_t (34.36 \times \sqrt{2})}{12} = 4.05 \times \sigma_t \text{ K/IN}$$

(1) ACCORDING TO NAVDOCK,

CO-EFFICIENT OF EARTH PRESSURE (AT REST) $K' = \frac{\cos^2\phi}{2 - \cos^2\phi} = 0.6$

$$\sigma_T = K \times \gamma (H - H_1) + K \gamma' (H_1 - \frac{H}{4}) + \gamma_\omega (H_2 - \frac{H}{4})$$

$$= 0.60 \times 0.12 \times 9.3 + 0.6 \times .06 \times (26.7 - \frac{36}{4}) + .0625 (17.4 - \frac{36}{4})$$

$$= 0.67 + 0.71 + 0.53 = 1.81 \text{ K/LIN FT.}$$

$$t_{MAX.} = 4.05 \times 1.81 = \underline{7.33} \text{ K/IN} < 8.0 \text{ K/IN} \quad \text{O.K}$$

(2) ACCORDING TO REF. 3 P.77, USING abdc FOR PRESSURE DIAGRAM, AND $K = K_A = 0.3$

$$\sigma_T = K_A \gamma (H - H_1) + K_A \gamma' H_1 + \gamma_\omega \times H_2$$

$$= 0.3 \times 0.12 \times 9.3 + 0.3 \times .06 \times 26.7 + .0625 \times 17.4$$

$$= 0.37 + 0.48 + 1.09 = 1.94 \text{ \#/LIN. FT.}$$

$$t_{MAX.} = 4.05 \times 1.94 = \underline{7.75} \text{ K/IN} < 8.0 \text{ K/IN} \quad \text{O.K}$$

VERTICAL SHEAR

(1) DRIVING SHEAR $V_{MAX.} = \frac{3M}{2B}$

SHEARING RESISTANCE $S_T = P_S \times \tan\phi + f \times P_T$
(ALONG ℄ OF CELL)

$P_S = \frac{1}{2} K\gamma(H-H_1)^2 + K\gamma(H-H_1) \times H_1 + \frac{1}{2} K\gamma' H_1^2$, $K = 0.6$

$= \frac{1}{2} \times 0.6 \times 0.12 \times (9.3)^2 + 0.6\,(0.112) \times 26.7 + \frac{1}{2} \times 0.6 \times .06 \times (26.7)^2$

$= 3.12 + 1.79 + 128.0 = 132.9^K$

P_T = INBOARD SHEETING AREA 'abc'

$= \frac{1}{2} K_A \gamma (H-H_1)^2 + K_A \gamma (H-H_1) \times H_1 + \frac{1}{2} K_A \gamma' H_1^2 + \frac{1}{2} \gamma_w \times H_2^2$

$- \frac{1}{2}\left[\gamma_w H_2 + K_A \gamma' H_1 + K_A \gamma (H-H_1)\right] \times \left(\frac{H}{4}\right)$, $K_A = 0.33$

$= \frac{1}{2} \times 0.33 \times 0.12 \times (9.3)^2 + 0.33 \times 0.12 \times (9.3) + \frac{1}{2} \times 0.33 \times 0.06 \times (26.7)^2$

$+ \frac{1}{2} \times .0625 \times (17.4)^2 - \frac{55.8}{8}\left[.0625 \times 17.4 + 0.33 \times .06 \times 26.7 + 0.33 \times 0.12 \times 9.3\right]$

$= 154.5^K$

$S_T = 132.9 \times 0.577 + 0.3 \times 154.5 = 125.4^K$

$V_{MAX.} = \frac{3M}{2B} = \frac{3\left[486 - \frac{1}{2} \times .06 \times 15^2 \times \frac{15}{3}\right]}{2 \times 55.8} = 12.2^K$

$F.S = \frac{S_T}{V_{MAX.}} = \frac{125.4}{12.2} = 10.3 > 1.25$ O.K.

(2) ACCORDING TO TERZAGHI (REF. 3, P. 79),

$F.S = AP \times R \times f \times \left(\frac{B}{L}\right)\left(\frac{L + .25B}{L + .5B}\right)$

WHERE $\Delta P = P_T - P_P = P_T - 0 = P_T$

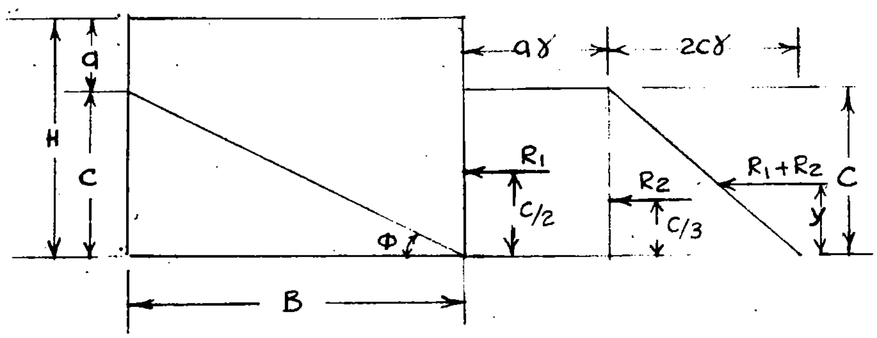
R = RADIUS OF COFFERDAM CELL = 31.83

$$F.S = \frac{154.5 \times 31.83 \times 0.30 \times \frac{55.8}{34.36} \times \frac{34.36 + 0.25 \times 55.8}{34.36 + 0.5 \times 55.8}}{452.25} = 4.12 > 1.25$$

O.K

HORIZONTAL SHEAR (CUMMING'S METHOD) SEE REF.3, P.75

$$\text{FACTOR OF SAFETY } F.S = \frac{M_r + M_i + P_P \times \frac{H_B}{3}}{\frac{1}{3} \times P_W \times H}$$



R = SHEAR RESISTANCE OF CELL = $R_1 + R_2 = ac\gamma + c^2\gamma$

M_r = MOMENT DUE TO ULTIMATE LATERAL SHEAR RESISTANCE OF CELL

$= R_1 \frac{c}{2} + R_2 \frac{c}{3}$

$$M_r = \frac{ac^2\gamma}{2} + \frac{c^3\gamma'}{3} = \frac{3.8(32.2)^2 \times .06}{2} + \frac{(32.2)^3 \times .06}{3} = 783^{1-K}$$

M_i = MOMENT DUE TO INTERLOCK FRICTION.

$M_i = P_T \times f \times B = 154.5 \times 0.30 \times 55.8 = 2600^{1-K}$

DUE TO BERM $\frac{P_P \times H_B}{3} = \frac{1}{2}\gamma' H_B^2 \times \left(\frac{H_B}{3}\right) = \frac{.06}{6} \times 15^3$ (H_B = HT. OF BERM = 15')

$= 33.75^{1-K}$

MOMENT DUE TO $P_W = M = \frac{1}{3} \times P_W \times H = 486^{1-K}$

$$F.S = \frac{783 + 2600 + 33.75}{486} = \frac{3416.75}{486} = 7.0 > 2.0 \quad O.K$$

PULLOUT RESISTANCE. (OUTER FACE SHEETING)

$$\text{FACTOR OF SAFETY} = \frac{Q_{tu}}{Q_P} \geqq 1.5$$

Q_{tu} = ULTIMATE PULLOUT CAPACITY PER LIN. FT.

Q_P = AV. PILE REACTION DUE TO OVERTURNING MOMENT

$$Q_{tu} = \left[K_R (P_o)_{AVG.} \times \tan\delta + C_A\right] A_P L$$

K_R = CO-EFF. OF EARTH PRESSURE AT REST = 0.7

Po_{AVG} = OVER-BURDEN PRESSURE (AVG.) = $\frac{0.06 \times 30}{2} = 0.09^K$

C_A = ADHESION BETWEEN SOIL AND PILE = 0.75 KSF

A_P = AREA OF PERIMETER = 1 SQ. FT

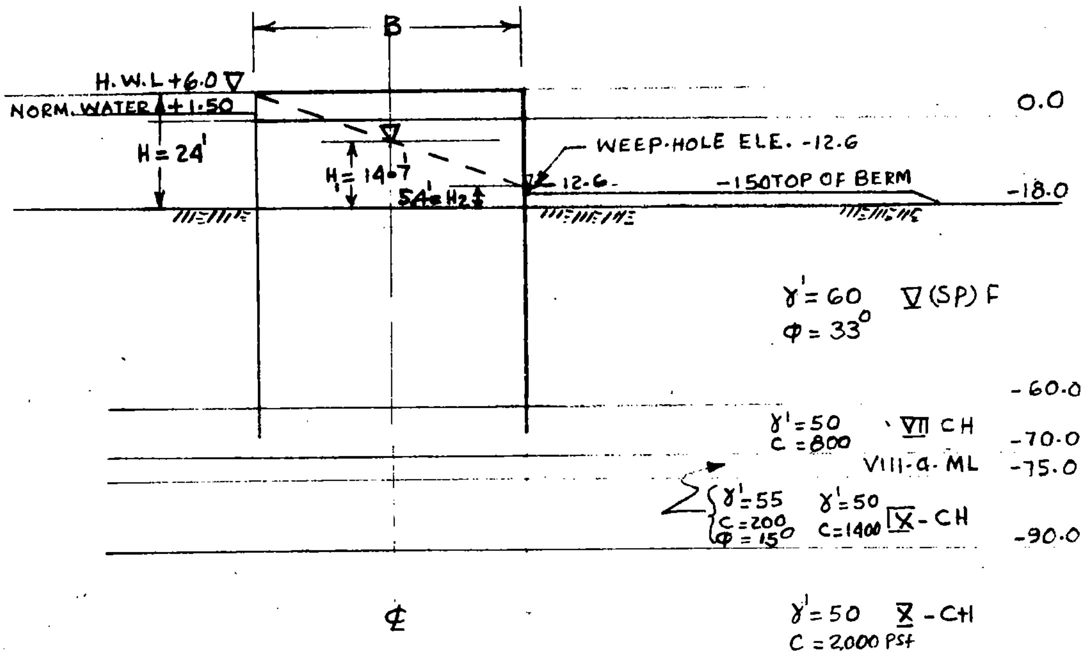
$$Q_{tu} = \left[0.7 \times 0.09 \times 0.25 + 0.75\right] \times 1 \times 30 + 0.75 \times 1 \times 20$$

$$= \underline{21.4\ K}$$

$$Q_P = \frac{0.5\,\gamma_\omega H^3}{3B\left(1+\frac{B}{4L}\right)} = \frac{0.5\,(.0625)\,(36)^3}{3\,(55.8)\left[1 + \frac{55.8}{4\,(34.36)}\right]} = 6.0^K$$

$$F.S = \frac{21.4}{6} = 3.57 > 1.5 \quad O.K$$

COFFERDAM DESIGN FOR CONSTRUCTING CONTROL STRUCTURE FOR SECTION AT EXISTIN GROUND ELE. -18.0



CONSIDERING SATURATION LINE FROM +6.0 TO -12.6

WEIGHT OF CELL FILL $\bar{W} = \frac{(24-5.4)(55.8)}{2} \times 0.12$

$$+ \frac{(5.4+24)}{2} \times 55.8 \times 0.06$$

$$= 111.5^{K}$$

MOMENT 'M' DUE TO HYDROSTATIC PRESSURE.

$$M = \frac{\gamma_\omega \times H^2}{2} \times \frac{H}{3} = \frac{.062 \times 24 \times 24}{2} \times \frac{24}{3}$$

$$M = 142.4^{1-K}$$

BEARING CAPACITY AT TIP EL. -65.0 (5' INTO VII-CH)

$$F.S = \frac{Q_{ult.}}{Q_{Max.}} = \frac{CN_c + \frac{1}{2}\gamma' B N_\gamma}{\frac{6M}{B^2} + \frac{\bar{W}}{B}} > 3.0$$

REFER TO REF. 2, NAVFAC DM-7 P. 7-11-6, FIG. 11-5 FOR 2 LAYER COHESIVE SOIL,

$$\frac{C_2}{C_1} = \frac{1400}{800} = 1.75, \quad \frac{T}{B} = \frac{10}{55.8} = 0.18, N_c = 8.8$$

$$Q_{ult.} = C N_c = 0.8 \times 8.8 = 7.04$$

$$Q_{MAX} = \frac{6 \times 142.4}{(55.8)^2} + \frac{111.5}{55.8} = 0.277 + 2.0$$

$$= 2.28$$

$$F.S = \frac{7.04}{2.28} = 3.10 > 3.0$$

SO FOR THE PORTION OF COFFERDAM PROPOSED AT EXISTING GROUND ELEVATION VARYING FROM -18.0 TO -6.0, THE TIP ELEVATION OF THE CELLS IS RECOMMENDED AT -65.0 .

REFERENCES:

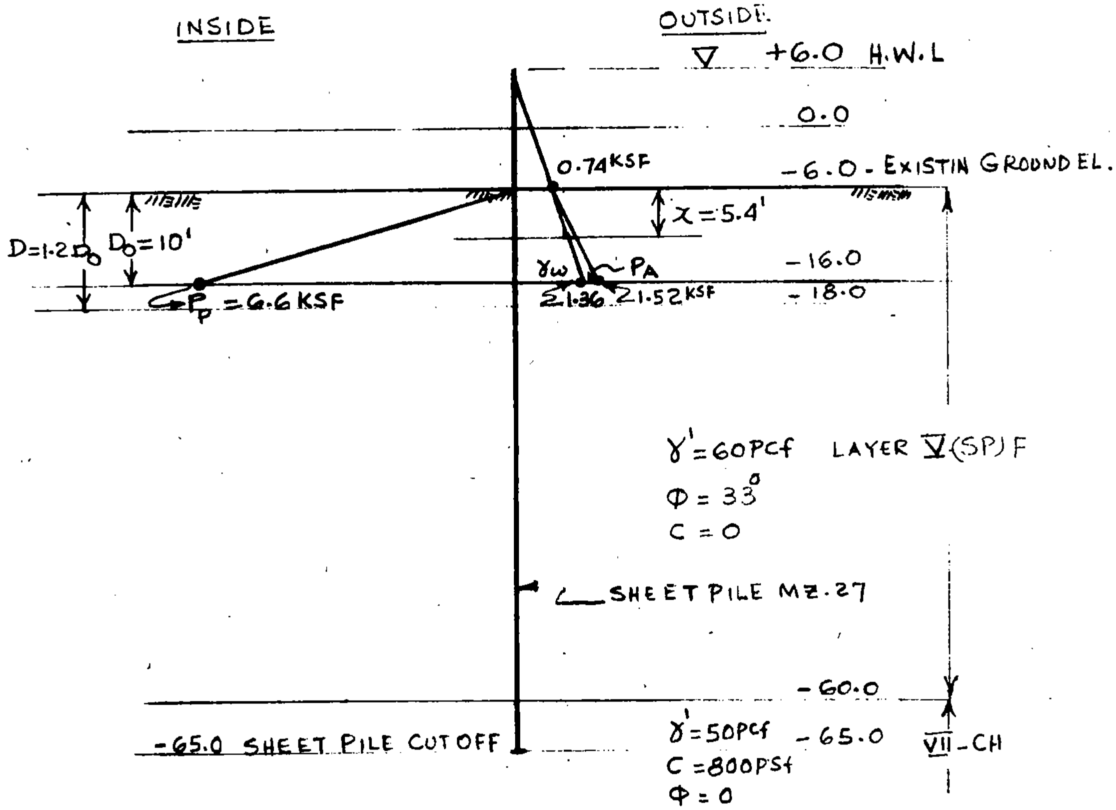
1. CORPS OF ENGINEERS, U.S ARMY, "COFFERDAMS AND CELLULAR RETAINING WALLS," EM 1110-2-2906, 1970

2. DEPARTMENT OF NAVY, "DESIGN MANUAL - SOIL MECHANICS, FOUNDATIONS, AND EARTH STRUCTURES" NAVFAC, DM-7, 1971

3. USS, "STEEL SHEET PILING DESIGN MANUAL," 1969.

4. TENG, W.C "FOUNDATION DESIGN," PRENTICE-HALL 1962.

DESIGN OF CANTILEVER STEEL SHEET PILE FOR CONSTRUCTION OF CONTROL STRUCTURE.



LATERAL PRESSURE DIAGRAM

ASSUMPTIONS

(1) THE MOST CRITICAL CONDITION FOR THE SHEET PILING WILL BE DURING ITS INSTALLATION UNDER DEWATERING WITHOUT BACKFILL INSIDE FOR HWL AT +6.0 OUTSIDE.

(2) FOR SIMPLICITY IN COMPUTATIONS, THE SOIL STRATA IS ASSUMED TO BE V(SP)F i.e FINE SAND FROM El. -6.0 TO El. -60.0.

(3) SIMPLIFIED METHOD OF ANALYSIS AS SUGGESTED BY TENG (REF. 1) IS USED.

DESIGN DATA.

ϕ = ANGLE OF INTERNAL FRICTION OF SOIL = 33°
γ' = SUBMERGED UNIT WEIGHT OF SAND = 60 PCF
γ = WET WEIGHT OF SAND = 120 PCF
δ = ANGLE OF WALL FRICTION = 14°, TAN δ = 0.25
K_A = CO-EFF. OF ACTIVE EARTH PRESSURE = 0.26 } REF. 2, P.8
K_P = CO-EFF. OF PASSIVE EARTH PRESSURE = 5.50 } REF. 2, P.8
F_a = ALLOWABLE DESIGN STRESS OF SHEET PILING = 25,000 PSI
(USING REGULAR CARBON GRADE)

DESIGN

ASSUME MINIMUM DEPTH OF PENETRATION = D_o

HYDROSTATIC PRESSURE AT D_o = $.062\,(D_o+12)$
$= 0.744 + .062\,D_o$ (KSF)

ACTIVE EARTH PRESSURE AT 'D_o' $= .06 \times D_o \times 0.26$
$= .0156\,D_o$ (KSF)

PASSIVE EARTH PRESSURE AT 'D_o' $= 0.12 \times D_o \times 5.5$
$= 0.66\,D_o$ (KSF)

TAKING MOMENTS AT 'D_o'

ACTIVE SIDE $M_1 = \frac{1}{2} \times 0.74 \times 12\,(4+D_o) + 0.74 \times D_o \times \frac{D_o}{2} + \frac{1}{2} \times (.062\,D_o + .0156\,D_o) \times D_o \times \frac{D_o}{3}$

$= 17.76 + 4.44\,D_o + 0.37\,D_o^2 + .013\,D_o^3$

PASSIVE SIDE, $M_2 = \frac{1}{2} \times 0.66\,D_o \times D_o \times \frac{D_o}{3} = 0.11\,D_o^3$

FOR $M_1 - M_2 = M = 0$, $D_o = 10'$

MINIMUM DEPTH OF PENETRATION = 10'

DESIGN DEPTH OF PENETRATION 'D' = $1.2 D_0 = 12'$

POINT OF ZERO SHEAR

ASSUME POINT OF ZERO SHEAR AT X FT. BELOW THE DREDGE LINE.

EQUATING THE SHEAR FORCES ON ACTIVE AND PASSIVE SIDES AND SOLVING FOR 'X'.

$$\frac{1}{2} \times 0.74 \times 12 + 0.74x + \frac{1}{2} \times .078 x^2 = \frac{1}{2} \times 0.66 x^2$$

$$x = 5.43'$$

MAX. BENDING MOMENT M_{MAX} WILL OCCUR AT $x = 5.43'$

$$M_{MAX.} = .013 \times (5.4)^3 + 0.37 \times (5.4)^2 + 4.44(4 + 5.4) - 0.11 \times (5.4)^3 \text{ FT. KIPS}$$

$$= 2.054 + 10.8 + 42.3 - 17.38 = 37.72^{1-K}$$

$$= 452.4 \text{ IN. K}$$

$$\text{SECTION MODULUS} = \frac{M_{MAX.}}{F_a} = \frac{452.4}{25.0} = 18.1 \text{ IN.}^3/\text{FT. OF WALL}$$

FROM REF. 1 (USS, STEEL MANUAL), SELECT SECTION MZ 27 WITH SECTION MODULUS = $30.2 \text{ IN.}^3/\text{FT. OF WALL}$.

ALTHOUGH THE DESIGN PENETRATION DEPTH IS 12' ONLY (I.E EL. -18.0), BUT TO MINIMISE SEEPAGE WATER, THE CUT-OFF LEVEL OF SHEET PILE IS RECOMMENDED TO BE -65.0 (5' INTO CLAY).

REFERENCES:

(1) TENG, W.C "FOUNDATION DESIGN" PRENTICE -HALL, INC. 1962

(2) USS, "STEEL SHEET PILING DESIGN MANUAL" 1970

ESTIMATION OF SEEPAGE QUANTITY THROUGH COFFERDAM.

SOURCES

(1) THROUGH THE INTERLOCKS IN SHEETPILE.

(2) THROUGH THE WEEP-HOLES PROVIDED IN COFFERDAM CELLS

(1) SEEPAGE QUANTITY THROUGH THE INTERLOCK Q_1

Q_1 = QUANTITY OF WATER PASSING THROUGH THE INTERLOCKS = 0.1 GPM/FT. OF WALL LENGTH FOR EACH 10 FT. DIFFERENTIAL IN HEAD ABOUT SHEETPILING (REF. 1)

$$= \left(0.1 \times \frac{36}{10} \times 3597 + 0.1 \times \frac{12}{10} \times 2125\right) \text{ G.P.M}$$

$$= 1295.0 + 255.0 = \underline{1550 \text{ G.P.M}}$$

(2) SEEPAGE QUANTITY THROUGH THE 4" DIA. WEEP HOLES PROVIDED AT EL. -13.0 IN CELLULAR COFFER-DAM

EVERY CELL HAS 9 WEEP-HOLES ON EITHER SIDE.

AREA OF 4" DIA. WEEP HOLE $= \frac{\pi \times 4^2}{4 \times 12 \times 12} = \frac{\pi}{36}$ S.F.

TOTAL NO. OF WEEPHOLES ON EACH SIDE OF THE COFFERDAM = 9 × NO. OF CELLS = 9 × 62 = 558

NET WTER HEAD = 13 + 6 = 19 FT.

THERE IS NO SIMPLE METHOD TO COMPUTE THE SEEPAGE QUANTITY THROUGH THE WEEP-HOLE. SO IT HAS BEEN ATTEMPTED TO FIND THE RATIO

BETWEEN THE AREA OF INTERLOCK AND WEEP-HOLE.

ASSUME THE AREA OF INTERLOCK = 0.25 SQ."

EVERY CELL HAS 59 SHEET PILES ON EITHER SIDE.

FOR 19 FT. OF WATER, THE TOTAL AREA OF

INTERLOCK OPENING FOR EACH CELL = $59 \times 19 \times \frac{0.25}{12}$
= 23.4 SQ.'

FOR 19 FT. OF WATER, THE AREA OF WEEP HOLES

IN ONE CELL = $9 \times \frac{\pi}{36}$ = 0.79 SQ.'
(CONTINUE SHEET)

WEEP-HOLE AREA = $\frac{0.79}{23.4}$ x AREA OF INTERLOCK OPENING

≈ 3.5 % x AREA OF INTERLOCK OPENING

BUT USE 10% TO BE ON SAFE SIDE

Q_2 = QUANTITY OF WATER PASSING THROUGH WEEP-HOLES
= $10\% \times 0.10 \times 4115 \times \frac{19}{10} \approx 80$ G.P.M

TOTAL QUANTITY OF WATER PASSING THROUGH THE

COFFER-DAM AND SHEETPILING = $Q_1 + Q_2$ = 1550 + 80 = 1630 G.P.M

REFERENCE:

(1) DEPARTMENT OF NAVY, "DESIGN MANUAL - SOIL MECHANICS, FOUNDATIONS AND EARTH STRUCTURES" NAVFAC, DM-7, 1971

ESTIMATION OF RUN-OFF QUANTITY

QUANTITY OF RUN-OFF Q (CUB. FT/SEC.) $= C \times I \times A$

WHERE C = CO-EFFICIENT OF RUN-OFF = 0.50 (REF. 1)

I = RAINFALL INTENSITY IN INCHES PER HOUR FOR THE TIME OF CONCENTRATION OF RUNOFF

= 2.50" TO BE EXPECTED ONCE IN 2 YEARS (REF. 2)

A = DRAINAGE AREA IN ACRES

AREA $A = A_1 + A_2 + A_3 + A_4 + A_5 + A_6$

$A_1 = \frac{1}{2} \times (630 + 900) \times 138$ SFT = 105,570 SFT

$A_2 = 1245 \times 900$ SFT = 1,120,500

$A_3 = \frac{1}{2} \times (900 + 405) \times 165$ SFT = 107,745

$A_4 = \frac{1}{2} \times (405 + 143) \times 660$ SFT = 180,840

$A_5 = \frac{1}{2} \times 45 \times 143$ S.FT = 3,220

$A_6 = \frac{1}{2} \times 60 \times 30$ S.FT = 900

TOTAL AREA A = 1,518,775 SFT

= 34.86 ACRES

$Q = 0.5 \times 2.5 \times 34.86$ = 43.6 CU. FT/SEC.

= 19,600 G.P.M

REFERENCES

(1) ELWYN E. SEELYE "DATA BOOK FOR CIVIL ENGINEERS - DESIGN VOLUME ONE - SECOND EDITION" NEW YORK - JOHN WILEY AND SONS, INC., SEPT. 1957

(2) DEPARTMENT OF ARMY, NAVY, AIRFORCE "DEWATERING AND GROUNDWATER CONTROL FOR DEEP EXCAVATIONS" TM 5-818-5, NAVFAC P-418, AFM 88-5, CHAP 6

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT RIGOLETS COMPLEX
DEWATERING SYSTEM
COMPUTED BY SA CHECKED BY JCL

SHEET NO. 1 OF 8
JOB NO. 18-111-62
DATE 6-15-72

DESIGN DATA

(1) THE RESULTS OF FIELD PUMPING TEST FOR TEST WELL 2W ARE ASSUMED TO DESIGN THE DEWATERING SYSTEM REQUIRED FOR CONSTRUCTING THE CONTROL STRUCTURE.

(2) CO-EFFICIENT OF HORIZONTAL PERMEABILITY $K_H = 109 \times 10^{-4}$ CM/SEC.
(SOIL STRATA V-SP F)

(3) SPECIFIC YIELD = 4 G.P.M PER FT OF DRAWDOWN
(SOIL STRATA V-SP F)

(4) RANGE OF GRAIN SIZE AT 10 % FINER BY WEIGHT D_{10} = .08 - .168 MM
(SOIL STRATA V-SP F)

CONDITIONS

(1) THE DEEP WELLS ARE TO BE LOCATED ON THE TOP OF THE BERM AT El.-15, AT A DISTANCE OF 5' FROM THE EDGE OF THE BERM

(2) THE WATER TABLE IS TO BE LOWERED DOWN TO A MINIMUM ELEVATION -32 i.e 2' BELOW THE EXISTING GROUND LEVEL FOR MOVEMENT OF TRAFFIC AND INSTALLATION OF WELLPOINT SYSTEM AROUND THE CONTROL STRUCTURE.

(3) THE WATER LEVEL IN THE WELL IS -53.0 i.e 5' ABOVE THE BOTTOM (El.-58) OF THE AQUIFER (SOIL STRATA V SP-F)

DESIGN OF DEEP WELLS

INSIDE DIA. OF WELL FOR MAX. CAPACITY OF 450 G.P.M = 10" (REF.1)

(1) DRAWDOWN CURVE :

BASED UPON THE PUMPING TEST DATA, DRAWDOWN CURVE (FIG.1) FOR 115 G.P.M IS PLOTTED.

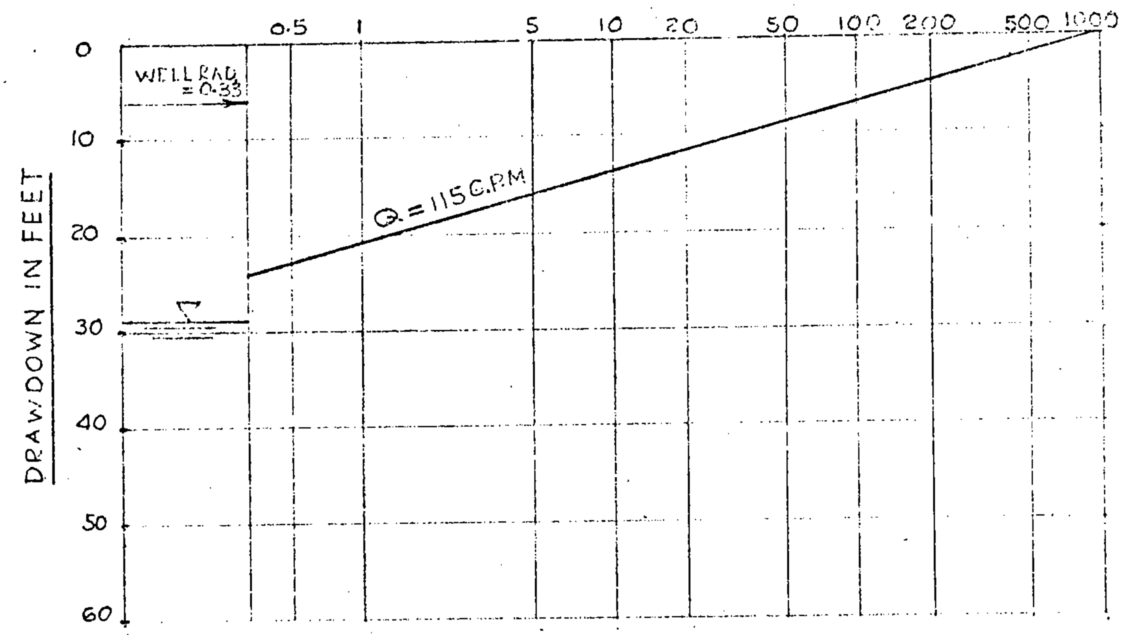


FIG. 1

DESIGN OF DEEP WELLS (CONTD.)

(2) SPACING BETWEEN WELLS

USING THE DRAWDOWN CURVE FOR Q = 115 G.P.M (FIG. 1),
TRY WELL SPACING = 150'

(A) AT WELL

DISTANCE FROM THE CENTER OF WELLS (FT.)	DRAWDOWN (FT.)	
(IN THE WELL) 0	29.0	
150	5.0	
150	5.0	
300	3.0	
300	3.0	Σ DRAWDOWN
450	2.0	AT WELL ≈ 53'
450	2.0	O.K (CONDITION 3)
600	1.2	
600	1.2	
750	0.6	
750	0.6	
	52.6	

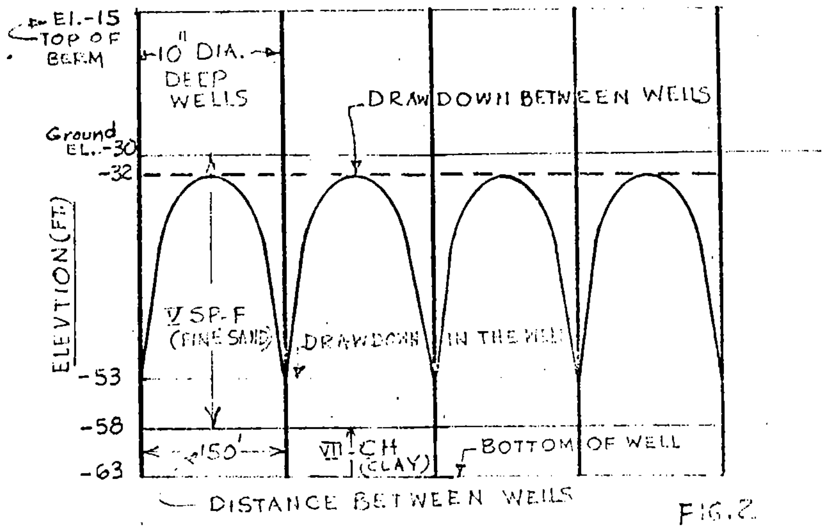


FIG. 2

CONES OF DEPRESSION BETWEEN WELLS

(B)
DRWDOWN BETWEEN WELLS FOR 150 SPAC.

DIST. FROM THE MID. PT. BETWEEN THE ADJACENT WELLS TO THE CENTER OF WELLS	DRAWDOWN (FT)
75'	7.0
75	7.0
225	4.0
225	4.0
375	2.5
375	2.5
525	1.5
525	1.5
675	1.0
675	1.0
825	0.3
825	0.3

Σ DRAWDOWN BET. WELLS = 33.6 O.K
SATISFIES CONDITION (2)

SO USE DEEP WELLS WITH A PUMPING RATE OF 115 G.P.M AT 150' C.T.C.

DESIGN OF DEEPWELLS (CONTD.)

(3) DESIGN OF WELL SCREEN AND SLOT SIZE

DATA

COEFFICIENT OF HORIZONTAL PERMEABILITY OF AQUIFER (LAYER V SP)

$= 103 \times 10^{-4}$ CM/SEC

ASSUME THE PERMEABILITY OF FILTER MATERIAL (GRAVEL PACK)

$= 300 \times 10^{-2}$ CM/SEC

A. LENGTH OF SCREEN (WITHOUT FILTER)

$$L_S = \frac{Q}{7.48 \times A_O \times V_C} \quad \text{(REF. 2)}$$

WHERE L_S = OPTIMUM LENGTH OF SCREEN, IN FT.

Q = DISCHARGE IN G.P.M = 450 (USE MAX. CAPACITY FOR 10" DIA. WELL)

A_O = EFFECTIVE OPEN AREA PER FT. OF SCREEN, IN SQ. FT

≃ 50% OF THE AREA OF SCREEN, PER FT. OF SCREEN, IN SQ. FT

$= 0.50 \times \pi \times \frac{10''}{12} = 1.3$ SQ. FT.

V_C = ENTRANCE VELOCITY IN FT. PER MIN

$= 0.20 \times 60 = 12$ FT. PER. MIN. (REF. 1)

$$\therefore L_S = \frac{450}{7.48 \times 1.3 \times 12} = 3.85'$$

B. DESIGN OF SLOT (WITHOUT FILTER)

WIDTH OF SLOT = D_{50} OF THE AQUIFER = .006" – .014" (FIG. 3)

= .010" (AVERAGE)

SO USE #10 SLOT.

C. DESIGN OF SLOT (WITH FILTER)

SCREEN - FILTER CRITERIA (REF. 1)

SLOTS: $\frac{\text{MIN. FILTER } D_{85}}{\text{SLOT WIDTH}} \geq 1.2$, FROM FIG. 3, SLOT WIDTH $= \frac{7.0}{1.2} = 5.8$ MM $= 0.23''$

SO USE #100 SLOT SCREEN

Fig. III - 25

FREDERIC R. HARRIS, INC.

CONSULTING ENGINEERS

SUBJECT: RIGOLETS COMPLEX
DEWATERING SYSTEM

COMPUTED BY: SA CHECKED BY: JCL

SHEET No. 5 OF 8

JOB No. 18-111-02

DATE 6-15-72

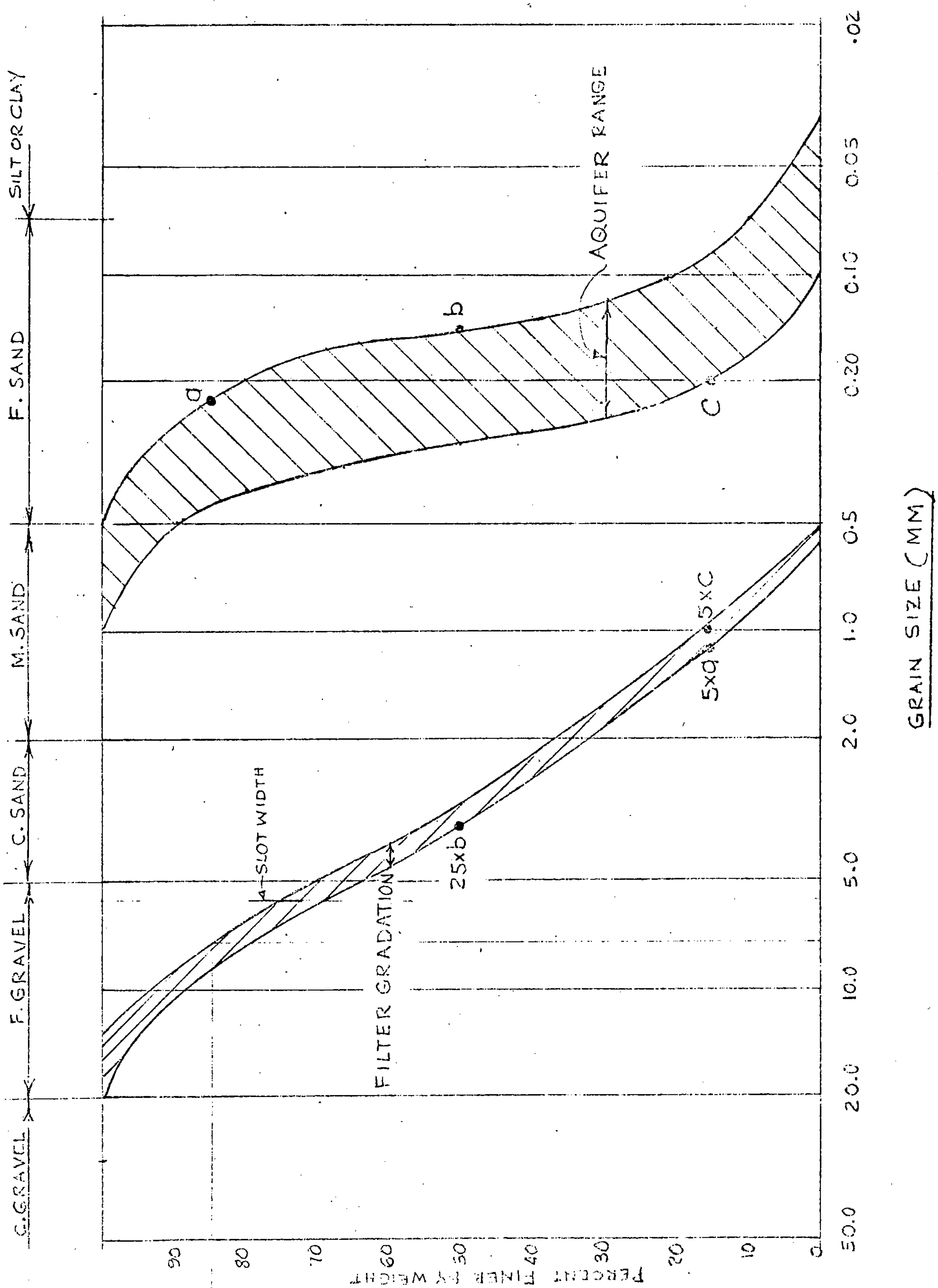


FIG. 3

(4) DESIGN OF FILTER

A GRADATION

(1) $\frac{\text{MAX. FILTER } D_{15}}{\text{MIN. AQUIFER } D_{85} = a} \leq 5.0$ MAX. FILTER $D_{15} = 5 \times a$

(2) $\frac{\text{MAX. FILTER } D_{50}}{\text{MIN. AQUIFER } D_{85} = b} \leq 25,$ MAX. FILTER $D_{50} = 25 \times b$

(3) $\frac{\text{MIN. FILTER } D_{15}}{\text{MAX. AQUIFER } D_{15} = c} \leq 5.0$ MIN. FILTER $D_{15} = 5 \times c$

(B) THICKNESS OF FILTER FOR LARGE-DIAMETER WELLS, 6-8" THICK FILTER IS REQUIRED (REF. 1)
SO PROVIDE 7" THICK FILTER AROUND THE PERFORATED PIPE.

EFFECTIVE WELL RADIUS = 5" (RADIUS OF WELL) + 7" (FILTER) = 12"

SUMMARY

(1) PROVIDE 10" I.D CLOSED-END CASING FITTED WITH PERFORATED PIPE OF 10" I.D OF METAL, SURROUNDED BY 7" THICK FILTER ACCORDING TO THE GRADATION SPECIFIED IN PLATE .....

(2) ALTHOUGH, THE PUMPING RATE OF 115 G.P.M IS DESIRED TO PULL DOWN THE WATER TABLE TO EL. -32.0 (AS SHOWN IN FIG. 2 FOR CONE OF DEPRESSION), IT IS RECOMMENDED TO USE PUMPS WITH A MAXIMUM CAPACITY OF 450 G.P.M.

(3) THE BOTTOM ELEVTION OF THE WELL IS RECOMMENDED AS -63.0 (i.e 5' INTO THE CLAY LAYER AND 10' BELOW THE WATER LEVEL) TO ACCOMMODATE THE PUMP BOWL, STRAINER AND 3' OF CLEARANCE AS SHOWN IN FIG. .

(4) PROVIDE PIEZOMETER WITH AUTOMATIC SWITCH FOR EVERY 4 PUMPS TO AVOID EXCESSIVE DRAWDOWN.

DESIGN OF WELLPOINTS

CONDITIONS

(1) THE BOTTOM ELEVATION OF THE FOOTING OF THE CONTROL STRUCTURE = -38.0. IN ORDER TO CONSTRUCT THE FOOTING, IT IS NECESSARY TO KEEP THE WATERTABLE BY ABOUT 5' BELOW THE BOTTOM OF THE FOOTING i.e TO AN ELE. -43.0 INSIDE THE CONTROL STRUCTURE CONSTRUCTION AREA.

(2) THE WELLPOINTS WILL BE INSTALLED ON THE MID. PT. OF THE SLOPE OF EXCAVATION AT A DISTANCE OF 4' FROM THE EDGE OF THE FOOTING AROUND THE CONTROL STRUCTURE AND AT AN EL. -35.0

DESIGN :

SPACING OF WELLPOINTS - CONVENTIONAL TYPE

ACCORDING TO REF. 1 PAGE 49, FOR GROUND-WATER LOWERING FROM EL. -32.0 (OBTAINED FROM DEEP-WELL PUMPING) TO EL. -43.0 i.e 11'.0 AND ADDING ABOUT 2'.0 TO BE ON THE SAFE SIDE THE REQUIRED SPACING OF WELLPOINTS IS 7' CENTER TO CENTER (APPROXIMATELY)

USE 2½" INCH DIA. CONVENTIONAL TYPE WELLPOINTS AT 7' C.T.C

WELLPOINT SCREEN

SLOT: FILTER MAY NOT BE REQUIRED FOR WELLPOINT SYSTEM ASSUMING CORROSIVE WATER, THE SLOT WIDTH = D_{50} OF THE AQUIFER (REF. [illegible])

AVERAGE 'D_{50}' = .010" (FROM FIG. 3)

SO USE #10 SLOT, 2¼" OUTSIDE DIA. AND 30" LONG STAINLESS STEEL SCREEN.

SUCTION LIFT

THE GREATEST SUCTION LIFT THAT CAN BE DEVELOPED IS ABOUT 25 FT., AND FOR MOST PUMPS THE LIMIT IS AROUND 20' (REF.), SO BOTTOM OF WELLPOINT IS -35-15.0 = -50.0

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT RIGOLETS COMPLEX
DEWATERING SYSTEM
COMPUTED BY SA CHECKED BY JCL

SHEET NO. 8 OF 8
JOB NO. 18-111-02
DATE 6-15-72

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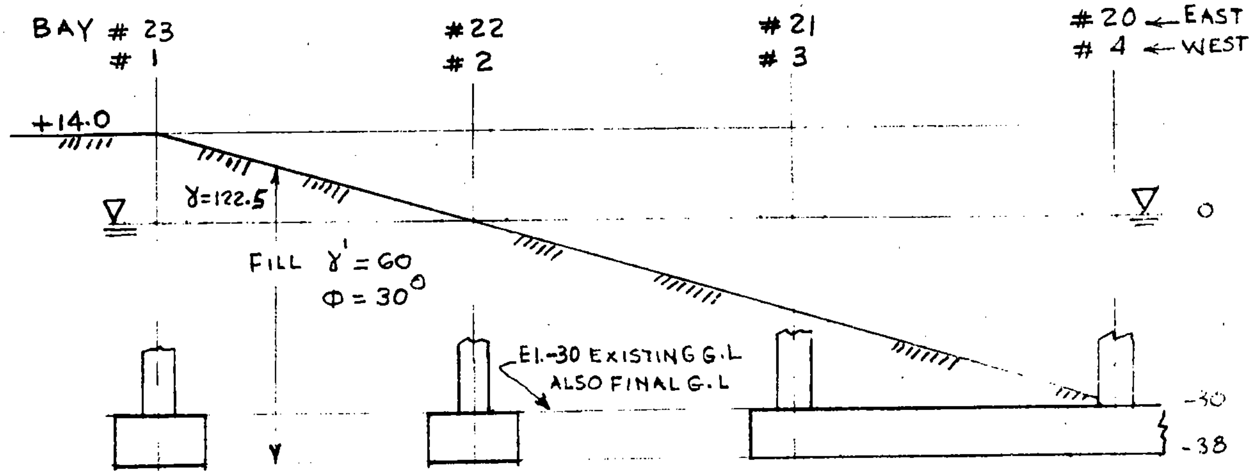
2. WILLIAM C. WALTON "SELECTED ANALYTICAL METHODS FOR WELL AND AQUIFER EVALUATION" BULLETIN 49 STATE OF ILLINOIS, DEPARTMENT OF REGISTRATION AND EDUCATION.

3. EDWARD E. JOHNSON INC. "GROUND WATER AND WELLS", EDWARD E. JOHNSON, INC. SAINT PAUL, MINNESOTA 55104.

CONTROL STRUCTURE - PILE DESIGN



V SP(F), $\gamma' = 60$, $K_R = 0.7$, $C_A = 0$
$\Phi = 33^\circ$ (OR 30°), $\tan\delta = 0.25$,

-60

VII (CH) $\gamma' = 50$, $C_A = 0.75C = 0.6$ KSF, $c = 800$, $e_o = 1.15$, $C_c = 0.35$, $C_r = .05$, $P_c = 5.6$ KSF

-70

VIII-a (ML), $\gamma' = 55$, $\Phi = 15^\circ$, $c = 200$, $C_A = C = 0.2^{KSF}$, $K_R = 0.7$, $\tan\delta = 0.2$

-75

IX (CH) - $\gamma' = 50$, $c = 1400$, $C_A = 0.7$ KSF, $e_o = 1.45$, $C_c = 0.87$, $C_r = .07$, $P_c =$ [illegible]

-90

X (CH) $\gamma' = 50$, $c = 2000$, $C_A = 0.75$ KSF, $e_o = 1.11$, $C_c = 0.42$, $C_r = .05$, $P_c = 8.2$ KSF

-120

XI (CH) $\gamma' = 55$, $c = 2500$, $C_A = 0.75^{KSF}$, $e_o = 1.26$, $C_c = 0.46$, $C_r = 0.06$, $P_c = 10.0$ KSF

-173

XII (SP) F $\gamma' = 60$, $\Phi = 33^\circ$

-180

XIII (ML), $\gamma' = 55$, $\Phi = 15^\circ$, $c = 200$

-186

XIV (CH), $\gamma' = 55$, $c = 3000$, $e_o = 0.99$, $C_c = 0.21$, $P_c = 10.0$ KSF

FROM LAB. Q-TEST: γ' IN PCF
c IN PSF

SCALE 1" = 30'

Fig. III - 30

PILE DESIGN FOR CONTROL STRUCTURE (FOR DESIGN DATA SEE SHEET 1.)

COMPUTATIONS

$$Q_{ult.} = F.S \times Q_p = A_p \Sigma SZ \quad (A.)$$

$$S = K_R (P_o)_{AVG.} \times \tan\delta + C_A \quad (A.S.)$$

LET $Q' = \Sigma SZ$ in K/LF.

$Q_{ult.}$ = ULTIMATE CAPACITY OF PILE (K)

Q_p = DESIGN CAPACITY OF PILE (K)

A_p = PILE PERIMETER (LF)

S = SHEAR RESISTANCE (KSF)

Z = THICKNESS OF SOIL STRATUM (FT.)

ΣZ = DEPTH OF PENETRATION (FT.)

$(P_o)_{AVG.}$ = AVERAGE OVERBURDEN PRESSURE (KSF)

K_R = CO-EFFICIENT OF EARTH PRESSURE

$\tan\delta$ = CO-EFFICIENT OF FRICTION

C_A = ADHESION (KSF)

LIMIT DEPTH FOR DRAG FORCE

REFERRING SHEET 1 AND CONSIDER THE EFFECT OF FILL UPTO +14.0 FOR BAY #1.

ASSUME SOIL STRATUM VIII-a (ML) SIMILAR TO VII (CH)

[$S = P_o \tan 15^\circ + 0.2$, P_o AT -70.0 (El. OF TOP OF STRATUM VIII-a ML)

$= 0.06 \times 8 + .06 \times 22 + .05 \times 10.0 = 2.2$ KSF

$S = 2.2 \tan 15^\circ + 0.2 = 0.59 + 0.2 = 0.79$ KSF

AND FOR VII-CH $S = C = 0.80$ KSF $\simeq$ SHEAR STRENGTH OF VIII-a (ML)]

SETTLEMENT OF STRATUM VII (CH) AND VIII-a (ML)

PRESENT OVERBURDEN P_o AT $-67.5 = 0.48 + 1.32 + (0.05) \times 7.5$

$P_o = 2.175$ KSF

WEIGHT OF FILL 'ΔP' $= 14 \times (0.122) + 30 \times .06$

$\Delta P = 3.5$ KSF

$P_o + \Delta P = 5.68$ KSF =

FOR LAYER VII-(CH), $P_c \doteq 5.60$ KSF

SINCE $P_o + \Delta P \simeq P_c$, THE SETTLEMENT IS ANTICIPATED TO BE OF MINOR ORDER OR NEGLIGIBLE.

SETTLEMENT OF STRATUM IX-(CH)

AT EL. -82.5 (MID. PT. OF STRATUM IX-CH)

$P_o = 2.175 + (.05) \times 7.5 + (.05) \times 7.5 = 2.93$ KSF

ΔP (DISPERSED) $= 3.5 \times 0.95 \simeq 3.33$

$P_o + \Delta P = 2.93 + 3.33 = 6.26$

AND $P_c = 6.2$ KSF

i.e $P_o + \Delta P \simeq P_c$, SO NEGLIGIBLE SETTLEMENT TO CAUSE DRAG FORCE.

SETTLEMENT OF STRATUM X-(CH)

AT EL. -105.0, $P_0 = 2.93 + (.05) \times 7.5 + (.05) \times 15$

$= 2.93 + .375 + .75$

$= 4.05$ KSF

DISPERSED $\Delta P = 3.5 \times 0.92 = 3.22$ KSF.

$P_0 + \Delta P = 7.27$ KSF

AND $P_c = 8.2$ KSF i.e $P_c > P_0 + \Delta P$.

SO, NEGLIGIBLE SETTLEMENT TO CAUSE DRAG.

BESIDES, THE GEOLOGY OF THE AREA INDICATES THE PLEISTOCENE DEPOSITS OF STIFF TO VERY STIFF CLAY DEPOSITS WITH HIGH PRECONSOLIDATION PRESSURE EXIST BELOW EL. -75.0 (APPROXIMATELY). HENCE IT IS RECOMMENDED TO ASSUME THE LIMIT DEPTH OF DRAG FORCE UPTO EL. -75.0.

SELECTION OF TYPE AND SIZE OF PILES

ASSUME RANGE OF PILE DESIGN LOAD = 50 - 100 TON | PILE

STRUCTURAL CAPACITY OF PILE = $Q_{MAX.} = A_S (0.35) F_y$

(FROM AISI CODE $F_y = 36$ KSI)

$Q_{MAX.} = 12.6 \, A_S \; (K)$

ALLOWABLE PILE DESIGN LOAD = $Q_P = Q_{MAX.} - Q_D \; (K)$

DRAG FORCE ON PILE $= Q_D = Q'_D \times A_P$

$Q'_D = \Sigma S Z \; (K/LF)$ AT DRAG FO[illegible]

THEREFORE, REFER to FIG. 1, 2, 3 & 4, THE DRAG FORCE FACTORS ARE DETERMINED AS FOLLOWS:

BAY No.	1-2 & 22-23	2-3 & 21-22	3-4 & 20-21	4 TO 20
Q'_D (K/LF)	29	21	15	0

IT IS RECOMMENDED:

1. USE NON-DISPLACEMENT, STEEL H-PILES, DRIVEN INTO CLAY.
2. USE FULL CROSS-SECTION AREA (A_S) AND FULL PERIMETER AREA (A_P) ASSUMING THAT CORROSION EFFECT IS NEGLIGIBLE AND ADHESION GOVERNS THE FRICTION PILE LOAD.
3. ASSUME THE DRAG FORCE DUE TO HIGH FILL OCCURS AT ABUTMENTS ONLY.

FREDERIC R. HARRIS, INC.
CONSULTING ENGINEERS

SUBJECT: RIGOLETS COMPLEX
PILE DESIGN FOR CONTROL STRUCTURE
COMPUTED BY: J.C.L. CHECKED BY: SA

JOB NO. 18-111-02
DATE: 5-24-72

PILE LENGTH AND DESIGN CAPACITY

$$Q_{ult.} = (Q' - 2Q'_D)\,A_P \quad (K)$$

$$Q_P = \frac{Q_{ult.}}{F.S} = \frac{(Q' - 2Q'_D)\,A_P}{F.S} \quad (K)$$

FOR COMPRESSION PILE

$$F.S. = 1.75$$

$$Q_P = \frac{(Q' - 2Q'_D)\,A_P}{1.75} \quad (K)$$

$$Req.\ Q' = \frac{1.75\,Q_P}{A_P} + 2Q'_D \quad (K/LF)$$

FOR TENSION PILE

$$F.S = 2.0, \quad Q'_D = 0$$

$$Q_P = \frac{Q' A_P}{2} \quad (K)$$

$$Req.\ Q' = \frac{2\,Q_P}{A_P} \quad (K/LF)$$

Assume: I. USING 12BP STEEL H-PILE, $A_P \simeq 6.0$ FT

FOR TENSION, REQUIRED $Q' = \frac{2}{6} Q_P = 0.33\,Q_P$ (K/LF)

FOR COMPRESSION REQUIRED $Q' = \frac{1.75\,Q_P}{6} + 2Q'_D = 0.292\,Q_P + 2Q'_D$ (K/LF)

SEE TABLES I-A, I-B, I-C, I-D for result of Z (Pile Penetration, cont'd)

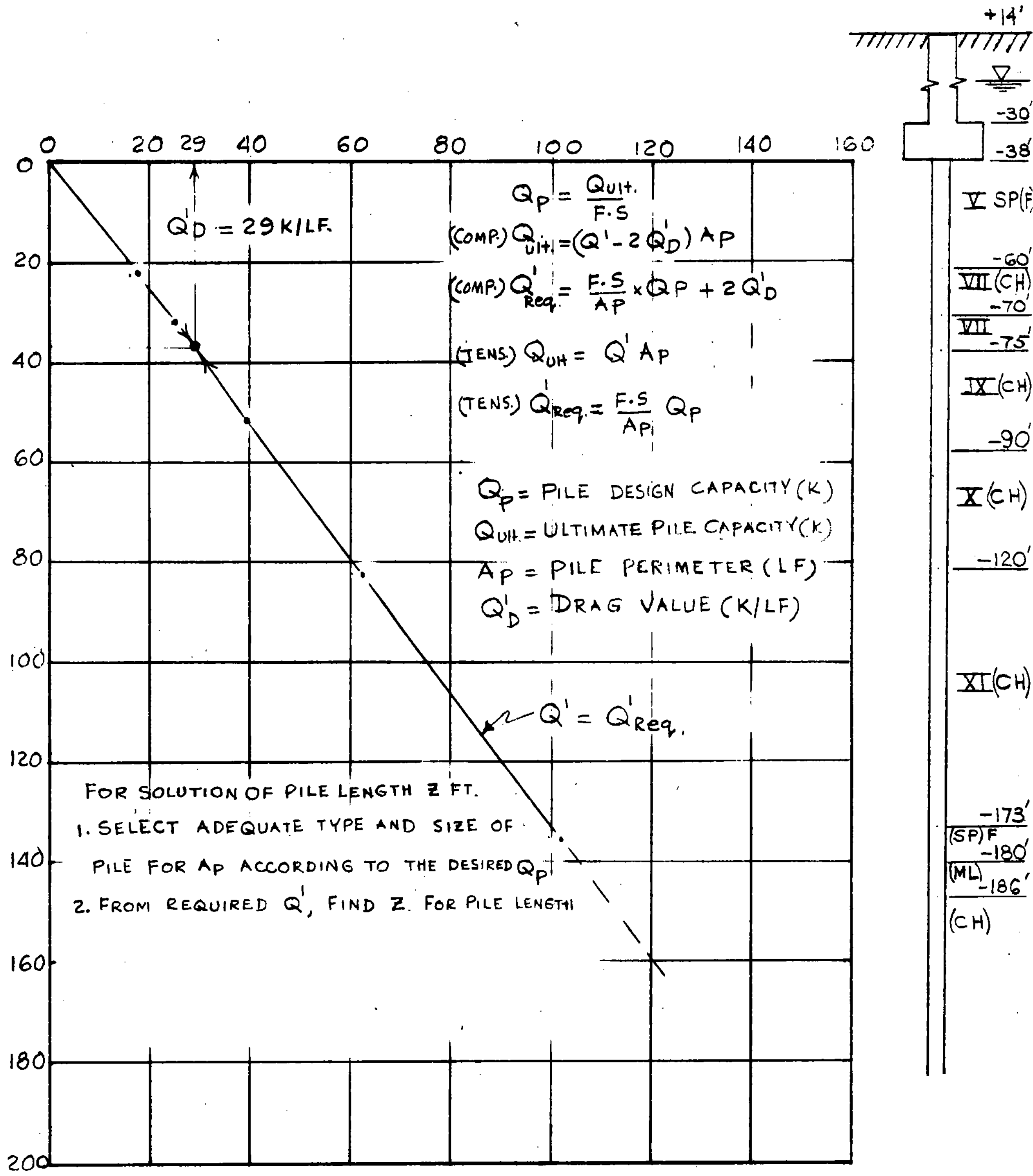
II. USING 14BP STEEL H-PILES, $A_P \simeq 7.0$ FT.

FOR TENSION, REQUIRED $Q' = \frac{2}{7} Q_P = 0.285\,Q_P$

FOR COMPRESSION, REQUIRED $Q' = \frac{1.75}{7} Q_P + 2Q'_D = 0.25\,Q_P + 2Q'_D$

SEE TABLES II-A, II-B, II-C, II-D for result of Z. (Pile Penetration, cont'd)

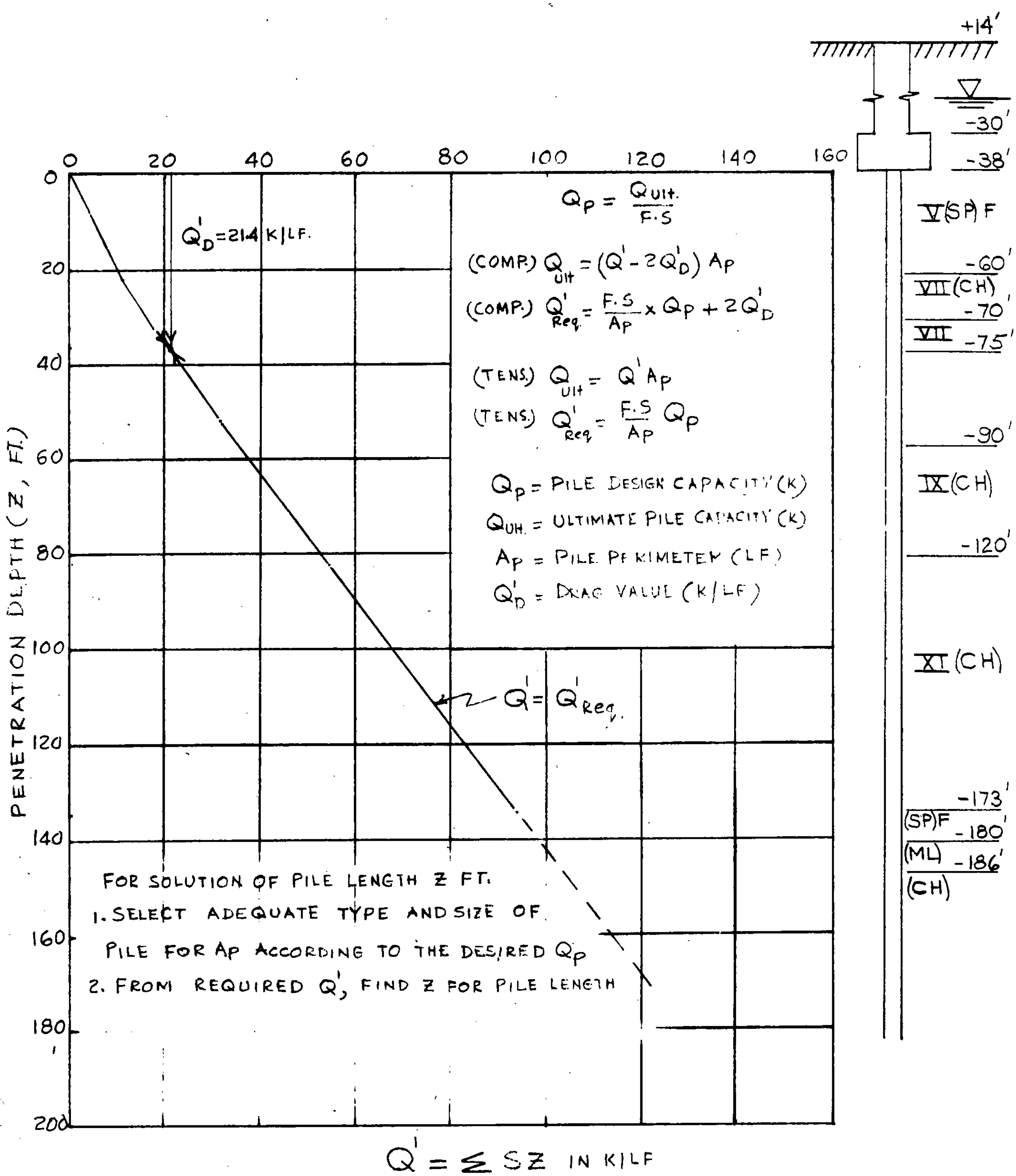
Fig. III - 35



$$Q' = \Sigma S Z \text{ IN K/LF}$$

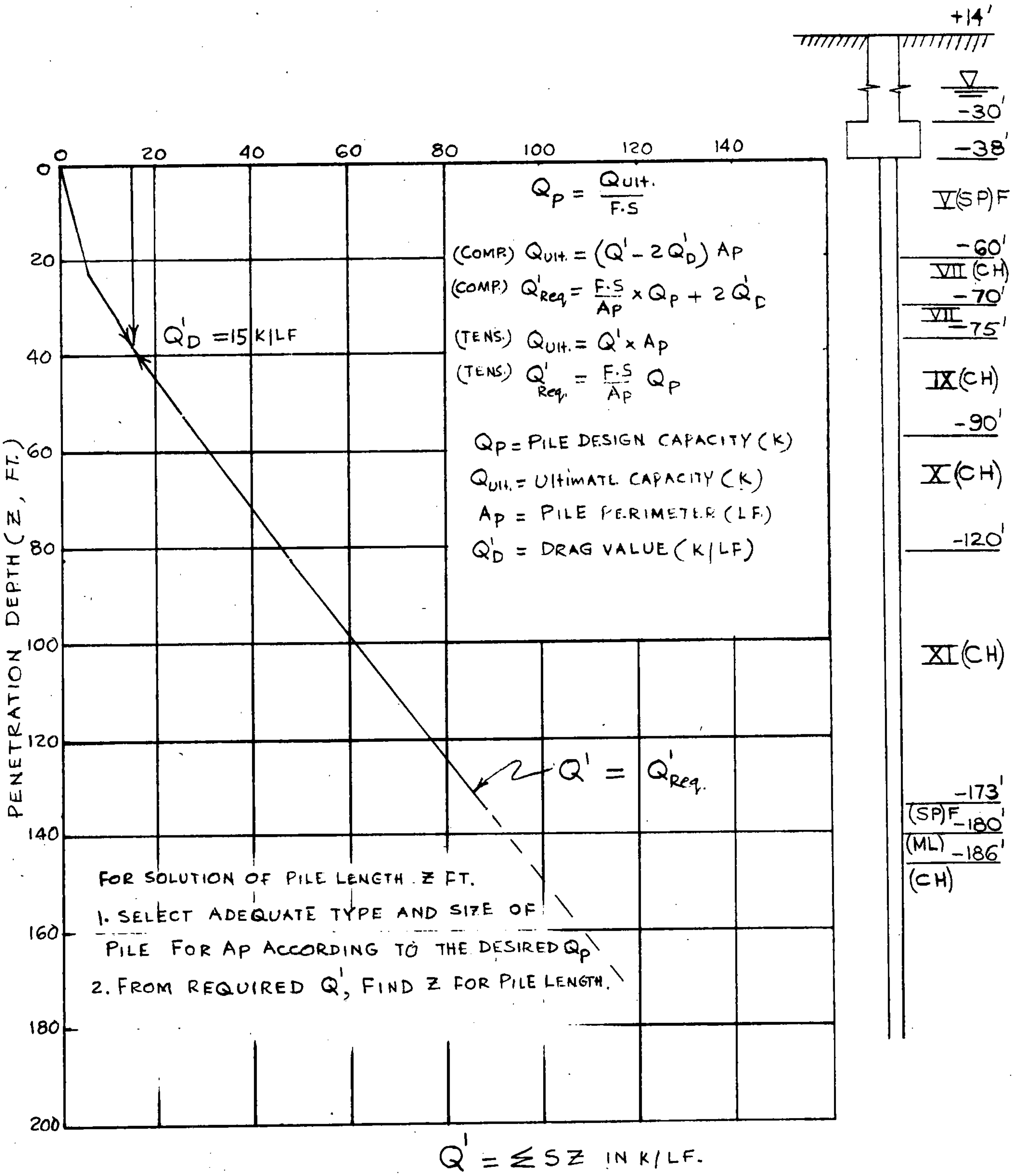
$$S = K_R (P_o)_{AVG.} \times \tan\delta + C_A \text{ IN KSF}$$

FIG. I – PILE DESIGN CHART (Q' VS Z) FOR CONTROL STRUCTURE BAY. 1-2 AND 22-23



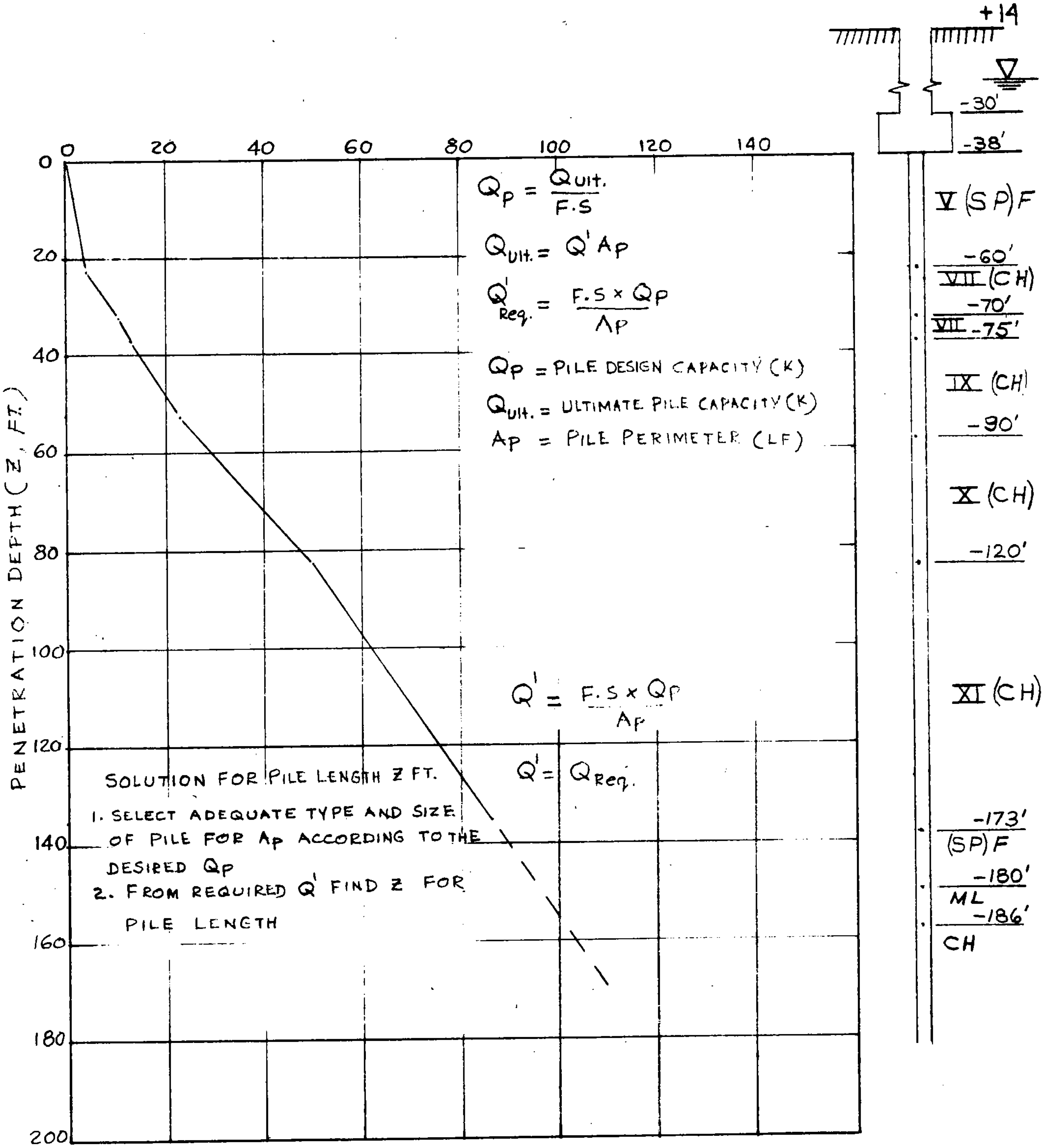
$S = K_R (P_O)_{AVG} \times \tan\delta + C_A$ IN KSF

FIG. 2 - PILE DESIGN CHART (Q' VS Z) FOR CONTROL STRUCTURE BAY 2-3 AND 22-23



$S = k_R (P_o)_{AVG} \tan\delta + C_A$ IN KSF.

FIG. 3 PILE DESIGN CHART (Q' VS Z) FOR CONTROL STRUCTURE FOR BAY 3-4 AND 20-21



$$Q' = \Sigma S Z \text{ IN K/LFT}$$

$$S = K_R (P_o)_{AVG.} \times \tan\delta + C_A \text{ IN KSF}$$

FIG. 4 PILE DESIGN CHART (Q' VS Z) - CONTROL STRUCTURE
BAY 4 TO 20.

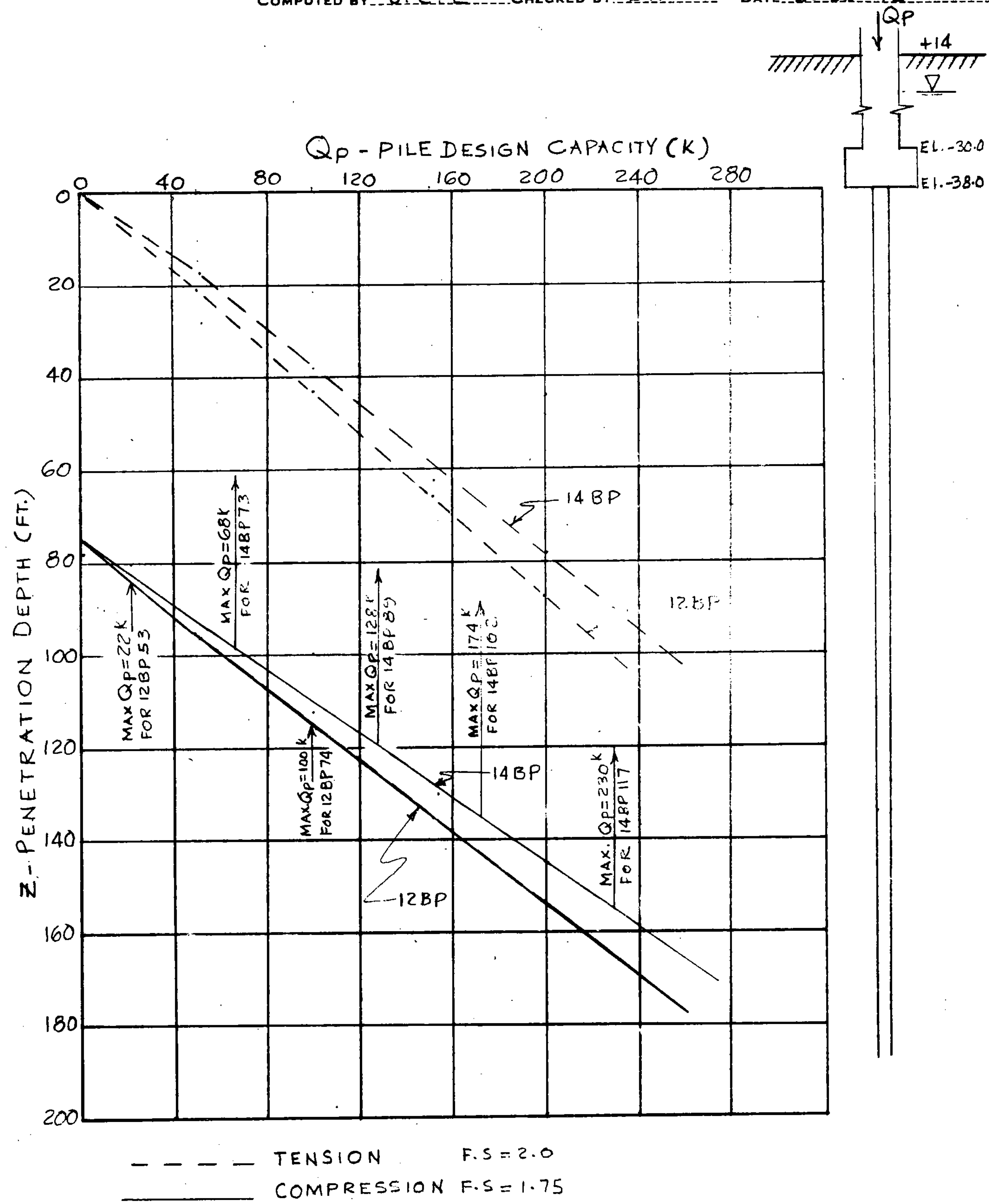
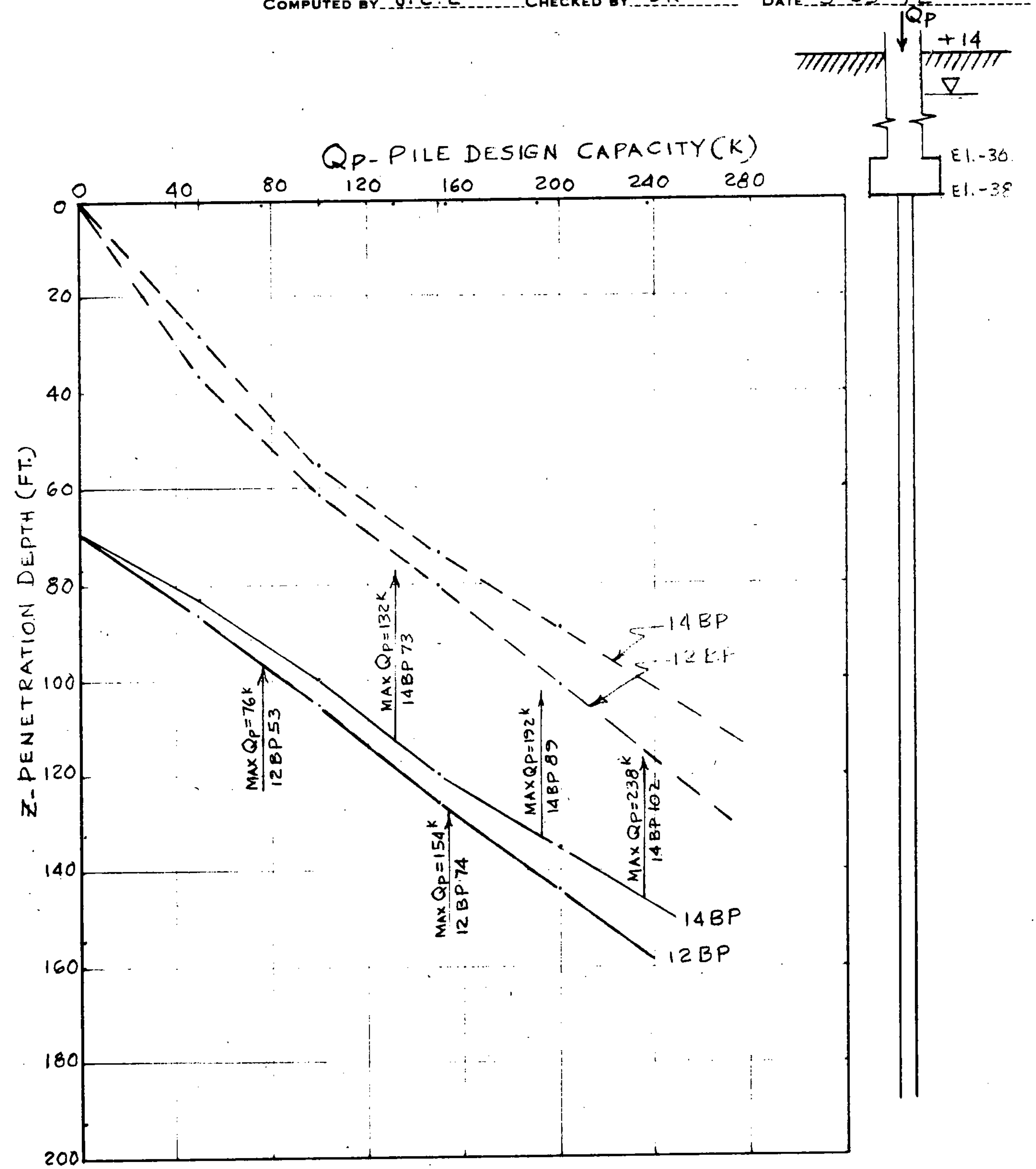


FIG. A. PILE DESIGN CHART FOR CONTROL STRUCTURE - BAY 1-2 AND 22-23



TENSION PILE F.S = 2.0

COMPRESSION PILE F.S = 1.75

FIG. B PILE DESIGN CHART FOR CONTROL STRUCTURE
BAY 2-3 AND 21-22

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SUBJECT RIGOLETS COMPLEX
PILE DESIGN FOR CONTROL STRUCTURE
COMPUTED BY J.C.L CHECKED BY SA

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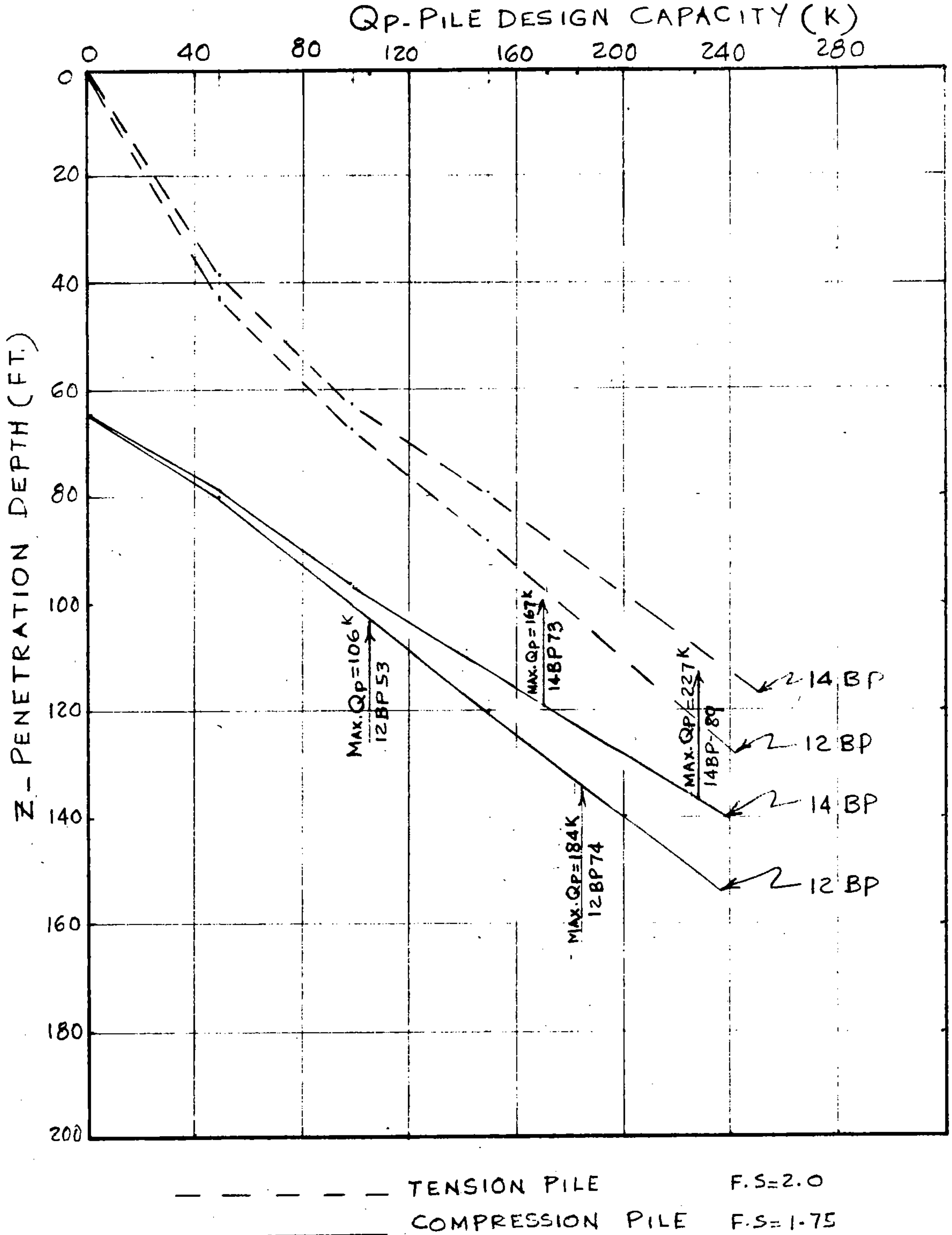
JOB NO. 18-11-02
DATE 5-25-72



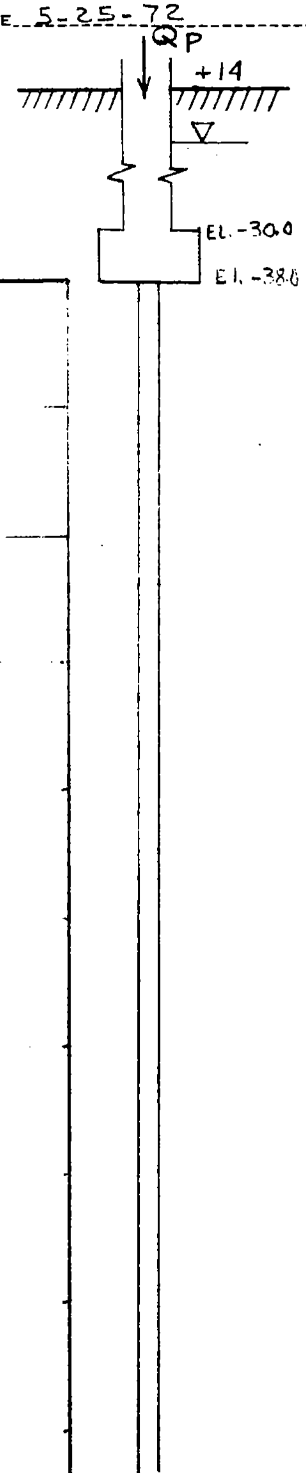
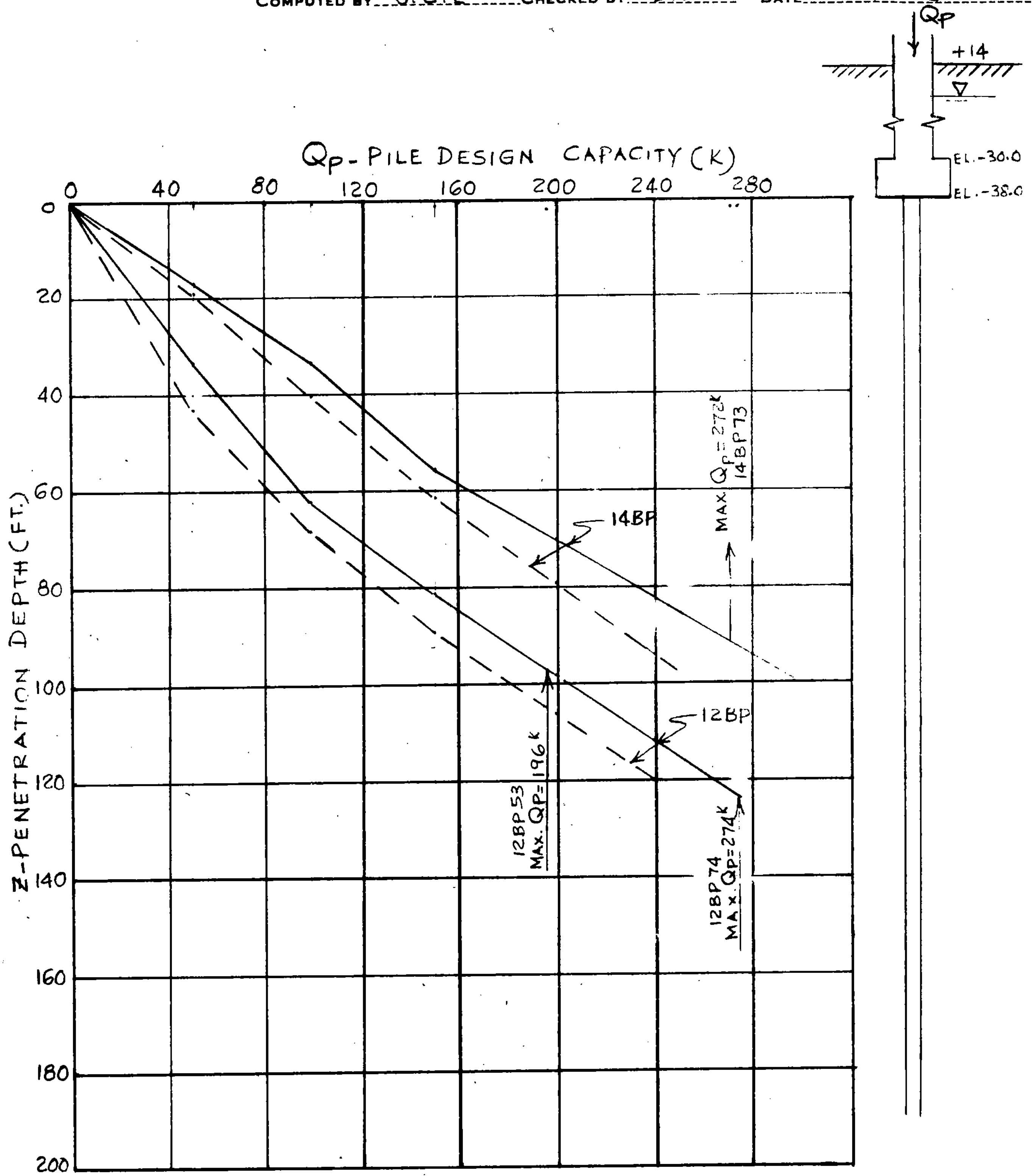


FIG.C- PILE DESIGN CHART FOR CONTROL STRUCTURE BAY 3-4 AND 20-21



— — — TENSION PILE, F.S = 2.0

———— COMPRESSION PILE, F.S = 1.75

FIG. D. PILE DESIGN CHART FOR CONTROL STRUCTURE, BAY 4 TO 20

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SETTLEMENT ANALYSIS

ASSUMPTIONS

(1) SOIL STRATIFICATION AND EMBANKMENT CROSS SECTION AS SHOWN IN FIG. ARE ASSUMED FOR SETTLEMENT COMPUTATIONS.

(2) WATER LEVEL IS TAKEN AS 0.0

(3) SETTLEMENT COMPUTATIONS FOR LAYER VI-CH APPLY TO THE CONDITION WHEN NO DREDGING IS PROPOSED

(4) SETTLEMENT COMPUTATIONS OF SUBSEQUENT CLAY LAYERS IS BASED WHEN REMOVAL OF LAYER VI-CH AND BACKFILL WITH HYDRAULIC SAND IS PROPOSED

(5) WHEN PRE-CONSOLIDATION PRESSURE 'P_c' > $P_o + \Delta P$ (WHERE P_o = OVERBURDEN PRESSURE AND ΔP = PRESSURE INCREMENT), THE RE-COMPRESSION INDEX 'C_r' IS USED FOR SETTLEMENT COMPUTATIONS

(6) THE SETTLEMENTS ARE COMPUTED UPTO AN EL: -312.0 WHERE THE EFFECT OF 'ΔP' ($\frac{\Delta P}{P_o} \simeq 0.1$) IS NEGLIGIBLE.

(7) AS THERE ARE NO BORING INFORMATION BELOW EL. -200.0, IT IS ASSUMED THAT LAYER XIV. CH EXTENDS UPTO EL. -312.0

(8) DUE TO LOW MAGNITUDE OF COMPRESSIONS, SECONDARY SETTLEMENTS ARE NEGLECTED

$*C_c = 1.18$

CENTER POINT & ELEVATION	DEPTH BELOW GROUND SURF. (FT.)	LAYER	LAYER THICKNESS H (FT.)	INITIAL VOID RATIO e_o	UNIT WEIGHT $\gamma_{sub.}$ (LB./CU.FT)	RE-COMPRESSION INDEX C_r	OVERBURDEN PRESSURE P_o (LB./SQ.FT.)	PRESSURE INCREMENT (DISPERSED) ΔP (LB./SQ.FT.)	PRIMARY SETTLEMENT $= \frac{H \times C_r \log \frac{P_o+\Delta P}{P_o}}{1+e_o}$ (INCHES)
A -44.0	20	VI-CH	16	1.82	30	*1.18	1,020	3,100	49.1
B -64.0	40	VII-CH	12	1.15	50	0.05	1,920	3,350	1.5
C -84.0	60	IX-CH	14	1.45	50	0.07	2,930	3,220	1.5
D -104.0	80	X-CH	30	1.11	55	0.05	4030	3,000	2.1
E -147.0	123	XI-CH	56	1.26	50	0.06	6,320	2,500	2.5
F -250.0	227	XIV-CH	124	1.26	55	0.06	11,965	1,760	2.4

Fig. III - 45

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SUBJECT RIGOLETS COMPLEX — SETTLEMENT ANALYSIS
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JOB NO. [illegible]
COMPUTED BY SA — CHECKED BY MR
DATE [illegible]

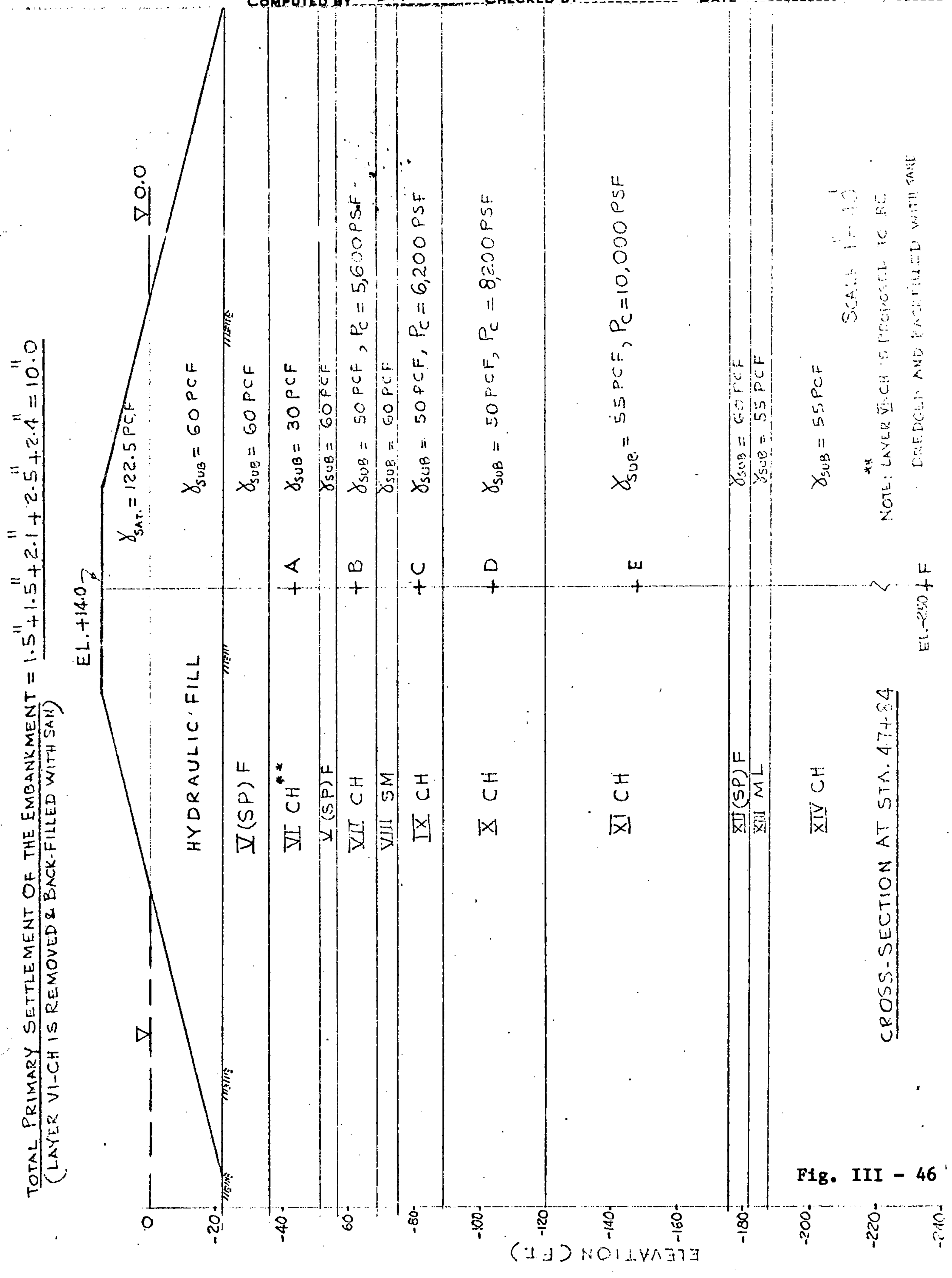


Fig. III - 46

TIME-RATE OF SETTLEMENT

ASSUMPTIONS

(1) THE CO-EFFICIENT OF VERTICAL CONSOLIDATION FOR DIFFERENT CLAY LAYERS IS EVALUATED FROM THE LABORATORY CONSOLIDATION CURVES. IT WAS FREQUENTLY NOTICED THAT THERE HAS BEEN A BIG DIFFERENCE IN 'C_v' VALUES FOR THE SAME LAYER AND SAME RANGE OF PRESSURE, SO THE 'C_v' VALUES USED FOR TIME-RATE OF SETTLEMENT COMPUTATIONS ARE PICKED UP ON THE BASIS OF STATISTICAL AVERAGE.

(2) THE 'C_v' VALUES FOR LAYERS XI CH AND XIV CH ARE ASSUMED AS 0.30 FT^2/DAY, DUE TO LACK OF SUFFICIENT DATA.

(3) FOR LAYER VII-CH, $C_v = 0.07\ FT^2/DAY$, DRAINAGE PATH 'H' = 6'
FOR " IX-CH, $C_v = 0.04\ FT^2/DAY$, DRAINAGE PATH 'H' = 14'
FOR LAYERS X-CH AND XI-CH (COMBINED) $C_v = 0.3\ FT^2/DAY$, DRAINAGE PATH 'H' = 43'
FOR LAYER XIV-CH, $C_v = 0.3\ FT^2/DAY$, DRAINAGE PATH 'H' = 62'

COMPUTATIONS

TIME FACTOR 'T' $= \dfrac{C_v \times t}{H^2}$, WHERE t = TIME

IF, t = 1 YEAR = 365 DAYS

FOR LAYER VII-CH, $T = \dfrac{.07 \times 365}{6 \times 6} = 0.71$, AND U = 87%, WHERE U = PERCENT CONSOLIDATION

" " IX-CH, T = 0.075, " " = 30%

" " X & XI-CH, T = 0.059 " " = 30%

" " XIV-CH, T = 0.03 " " = 20%

∴ IN ONE YEAR PRIMARY SETTLEMENT = 0.87×1.5" + 0.3×1.5" + .28(2.1"+2.5") + 0.2×2.4"
= 3.6"

COMPUTATIONS (CONTD)

IF, t = 2 YEARS

FOR LAYER VII - CH, T = 1.41, U = 90%
" " IX - CH, T = 0.15, U = 45%
" " X & XI - CH, T = 0.118, U = 37%
" " XIV - CH, T = 0.057, U = 30%

∴ IN 2 YEARS PRIMARY SETTLEMENT
= 0.9 × 1.5" + 0.45 × 1.5" + 0.37 × 4.6" + 0.3 × 2.4" = 4.5"

IF t = 5 YEARS

FOR LAYER VII - CH, T = 3.35, U = 95%
" " IX - CH, T = 0.38, U = 68%
" " X & XI - CH, T = 0.30, U = 60%
" " XIV - CH, T = 0.14, U = 43%

∴ IN 5 YEARS PRIMARY SETTLEMENT
= 0.95 × 1.5" + 0.68 × 1.5" + 0.60 × 4.6" + 0.43 × 2.4" = 6.2"

IF t = 6 MONTHS

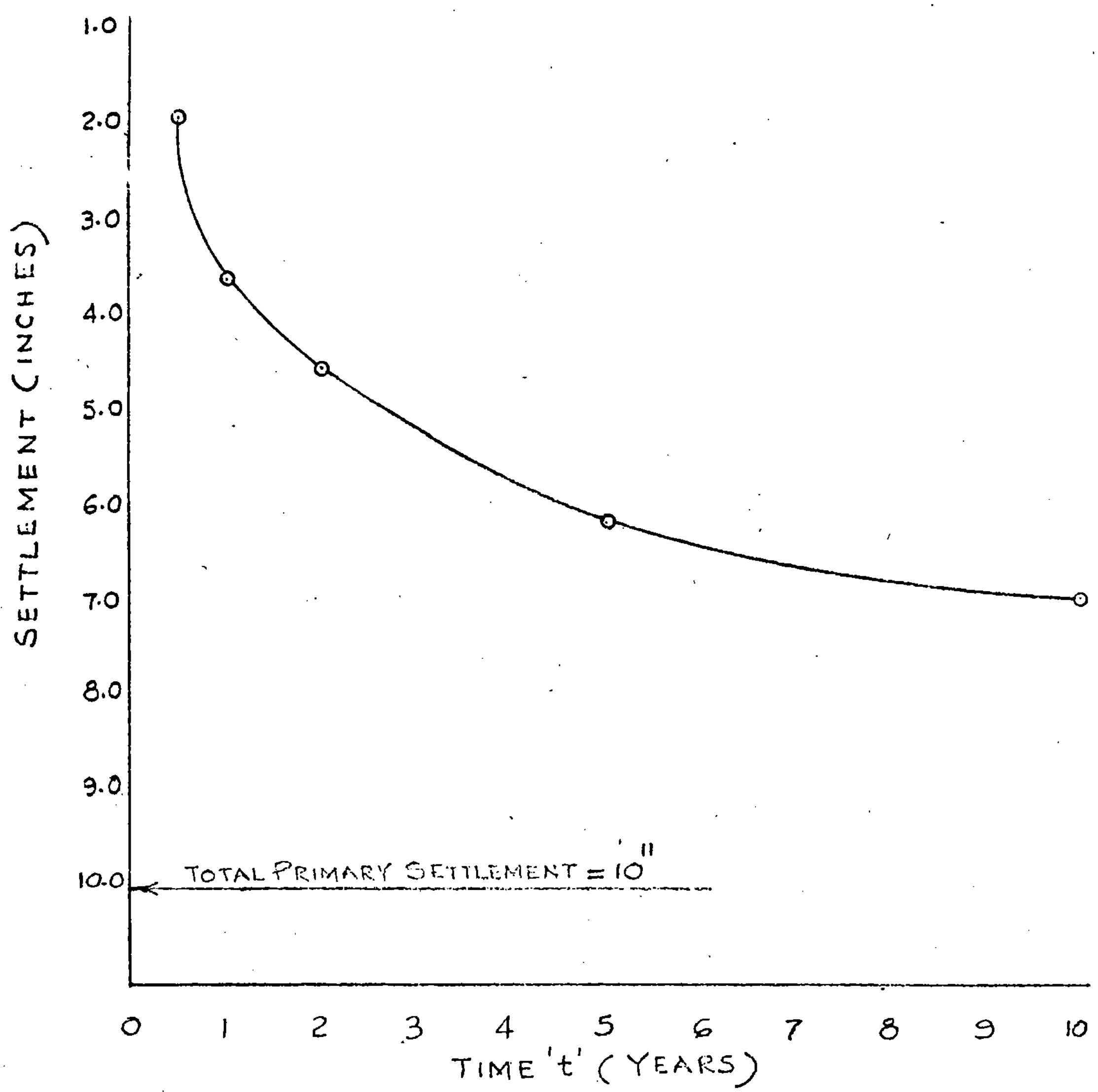
FOR LAYER VII - CH, T = 0.36, U = 65%
" " IX - CH, T = 0.04, U = 24%
" " X & XI - CH, T = 0.03, U = 20%
" " XIV - CH, T = 0.014, U = 10%

∴ IN 6 MONTHS PRIMARY SETTLEMENT
= 0.65 × 1.5" + 0.24 × 1.5" + 0.20 × 4.6" + 0.10 × 2.4" = 1.9"

IF t = 10 YEARS

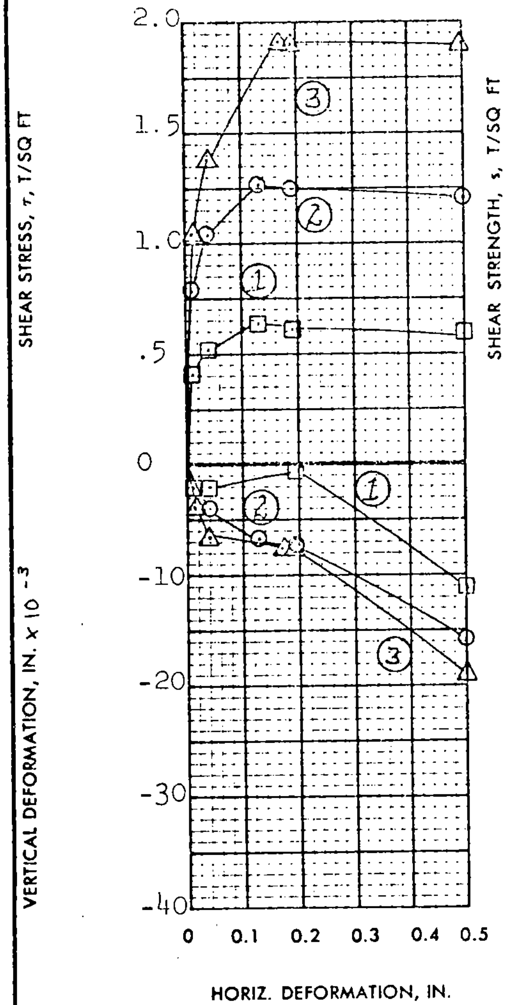
FOR LAYER VII-CH, T = 7.1, U = 97%
" " IX-CH, T = 0.76, U = 87%
" " X & XI-CH, T = 0.59, U = 81%
" " XIV-CH, T = 0.29, U = 61%

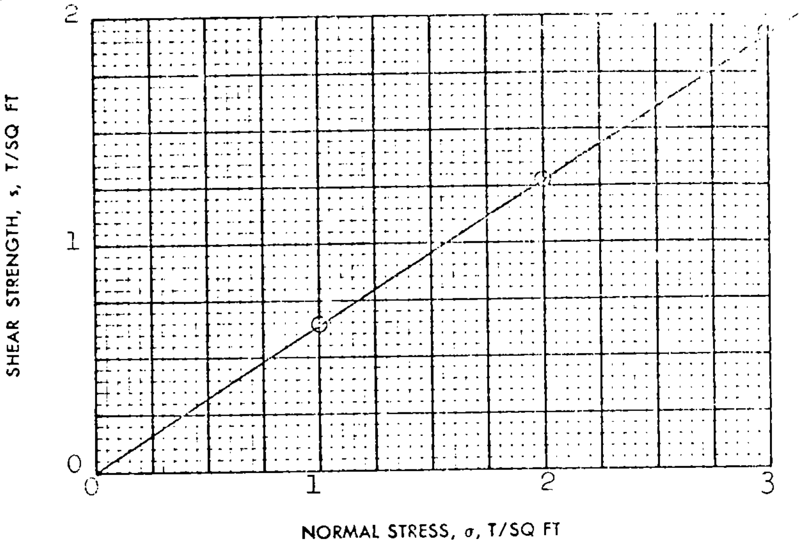
IN 10 YEARS PRIMARY SETTLEMENT = 0.97×1.5" + 0.87×1.5" + 0.59×4.6" + .61×2.4"
= 6.9"



TIME-RATE SETTLEMENT CURVE (AFTER REMOVAL OF LAYER VI-CH AND BACKFILL WITH HYDRAULIC SAND)

Fig. III - 49





SHEAR STRENGTH PARAMETERS

ϕ' = 32°

TAN ϕ' = 0.635

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	25.1 %	25.2 %	26.1 %	%
	VOID RATIO	e_o	0.682	0.691	0.716	
	SATURATION	S_o	97.9 %	97.0 %	97.0 %	%
	DRY DENSITY, LB/CU FT	γ_d	98.7	98.2	96.8	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	< 1	
FINAL	WATER CONTENT	w_f	23.3 %	23.1 %	23.2 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.64	1.27	1.93	
ACTUAL TIME TO FAILURE, MIN		t_f	960	900	1200	
RATE OF STRAIN, IN./MIN			.00016	.00016	.00016	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.539 IN. THICK

CLASSIFICATION SILTY SAND(SM), gray

LL – | PL – | PI – | G_s 2.66

REMARKS

PROJECT LK. PONT., LA.&VIC.-HURR. PROT. (1971)

RIGOLETS CONTROL STR.&CLOSURE DAM; DDM NO. 6

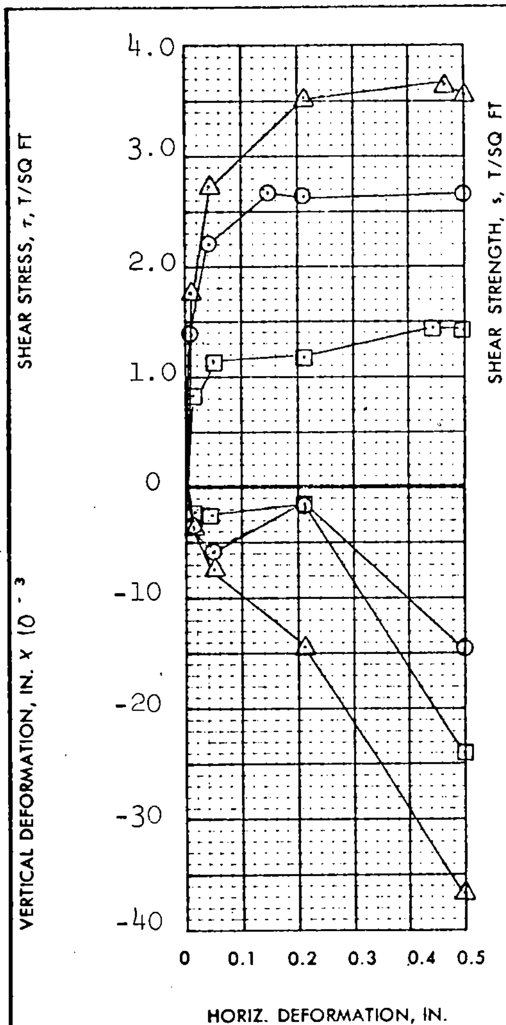
AREA

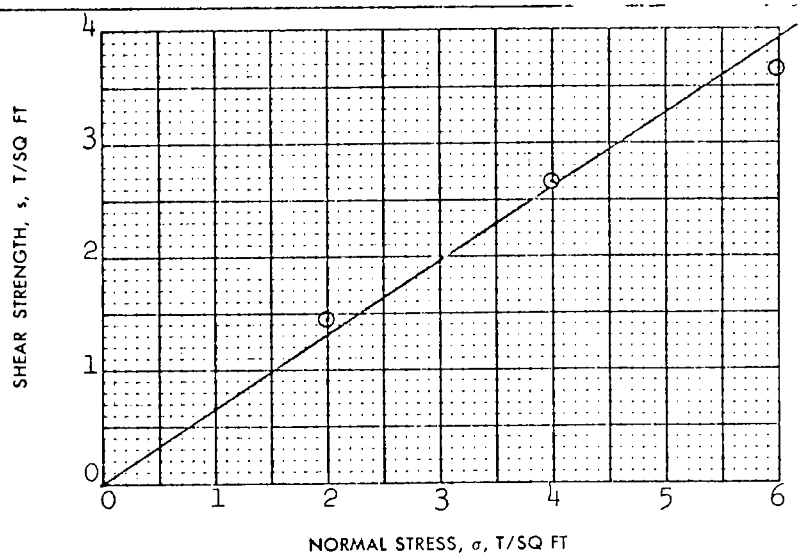
BORING NO. X-1-U — SAMPLE NO. 4-C

DEPTH El. -35.1 MSL — DATE 13 Dec. 1971

GDA

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 33°

TAN ϕ' = 0.655

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	22.5 %	22.1 %	21.8 %	%
	VOID RATIO	e_o	0.641	0.621	0.636	
	SATURATION	S_o	93.7 %	95.0 %	91.5 %	%
	DRY DENSITY, LB/CU FT	γ_d	101.6	102.8	101.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	24.3 %	24.2 %	23.8 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.44	2.68	3.65	
ACTUAL TIME TO FAILURE, MIN		t_f	2580	900	2880	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN	UNDISTURBED	3.00	IN. SQUARE	0.546	IN. THICK

CLASSIFICATION SAND(SP)F, gray, contains a trace of silt and clay

LL -	PL -	PI -		G_s 2.67

REMARKS

PROJECT LK. PONT., LA.&VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO. 6

AREA

BORING NO.	X-10-U	SAMPLE NO.	6-D
~~DEPTH~~ EL	-51.9 MSL	DATE	5 Jan. 1972

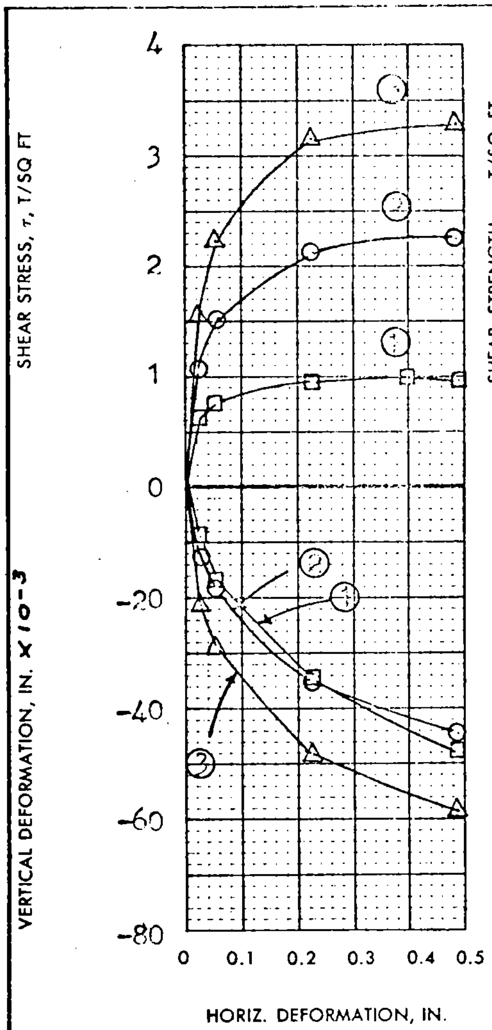
BWG

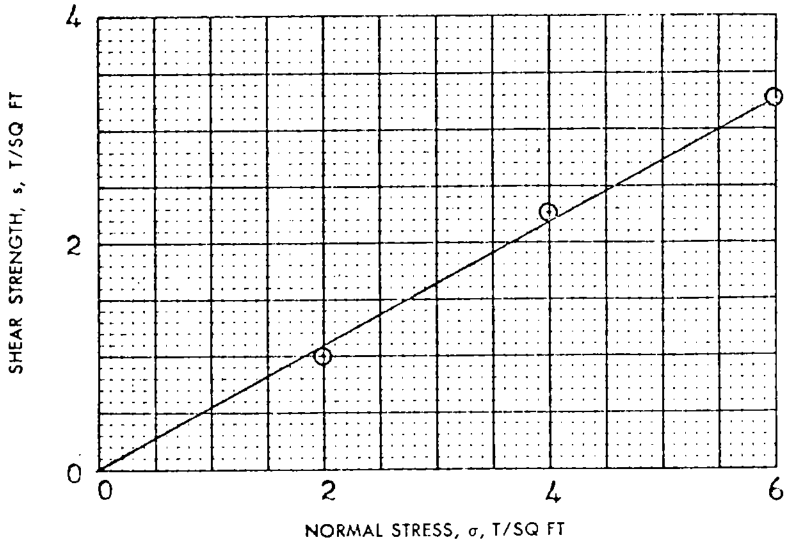
DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

GPO : 1966 OF

Fig. A - 2





SHEAR STRENGTH PARAMETERS

ϕ' = 29°

TAN ϕ' = 0.548

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	33.4%	31.6%	33.3%	%
	VOID RATIO	e_o	0.955	0.913	0.951	
	SATURATION	S_o	94.4%	93.4%	94.5%	%
	DRY DENSITY, LB/CU FT	γ_d	86.2	88.1	86.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	< 1	
FINAL	WATER CONTENT	w_f	25.3%	23.9%	28.3%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.00	2.24	3.29	
ACTUAL TIME TO FAILURE, MIN		t_f	2280	2760	2760	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION LEAN CLAY(CL), gray, contains pockets of sand

LL 41 | PL 15 | PI 26 | G_s 2.70

REMARKS

PROJECT LK. PONT., LA.& VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR.& CLOSURE DAM; DDM NO.6

AREA

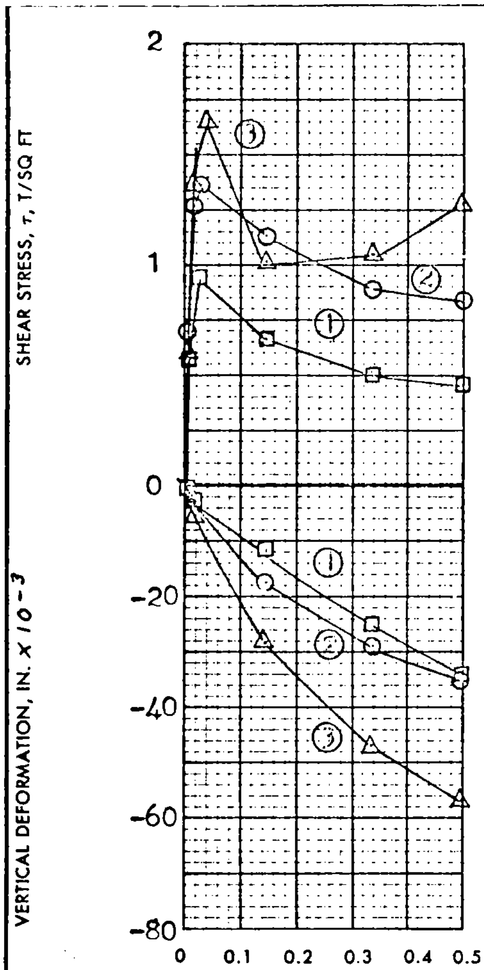
BORING NO. X-10U | SAMPLE NO. 8-B

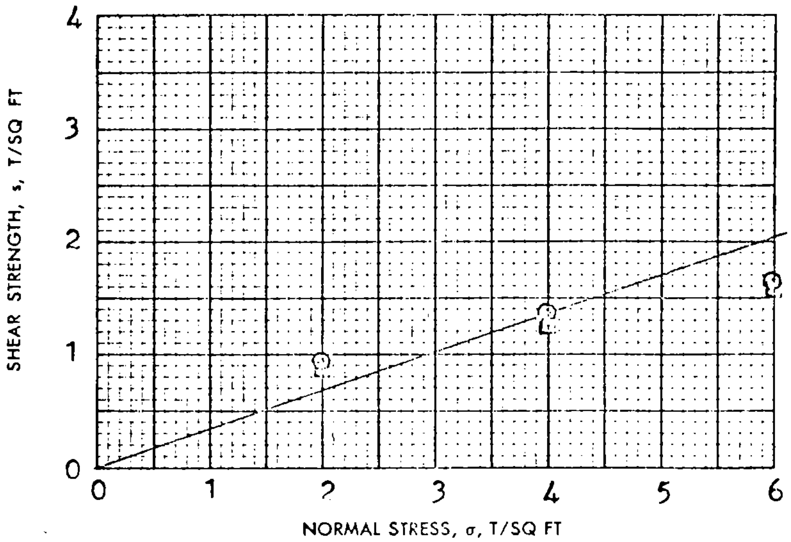
~~DEPTH~~ EL -58.2 MSL | DATE 6 January, 1972

GDA **DIRECT SHEAR TEST REPORT**

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—

Fig. A - 3





SHEAR STRENGTH PARAMETERS

ϕ' = 19°

TAN ϕ' = 0.343

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	44.6%	43.9%	44.4%	%
	VOID RATIO	e_o	1.25	1.22	1.23	
	SATURATION	S_o	98.5%	99.3%	99.6%	%
	DRY DENSITY, LB/CU FT	γ_d	76.7	77.5	77.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1	3	
FINAL	WATER CONTENT	w_f	46.5%	42.3%	40.8%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.94	1.37	1.66	
ACTUAL TIME TO FAILURE, MIN		t_f	390	450	540	
RATE OF STRAIN, IN./MIN			.00013	.00013	.00013	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH), gray, slightly slickensided

LL 91 — PL 28 — PI 63 — G_s 2.76

REMARKS Rerun test
⊡ Strength values of original test.

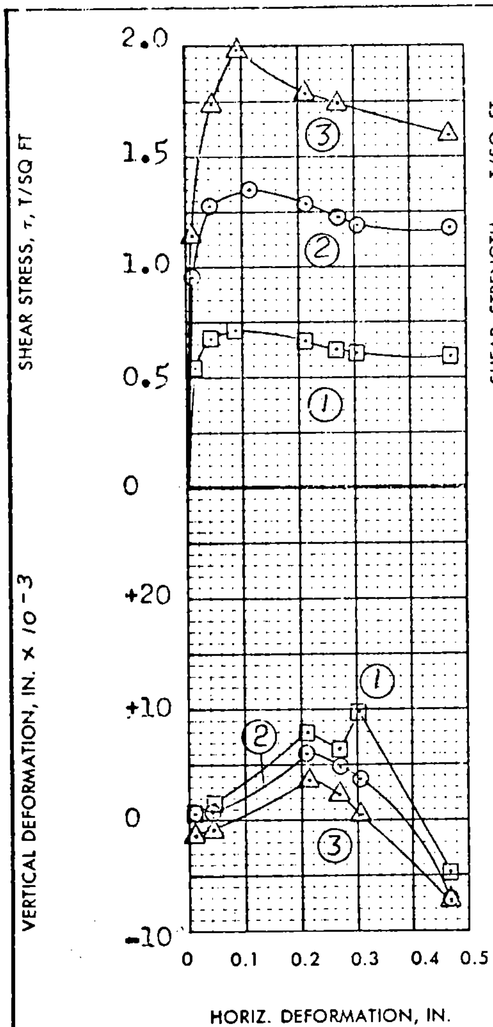
PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR. & CLOSURE DAM, DDM NO.6

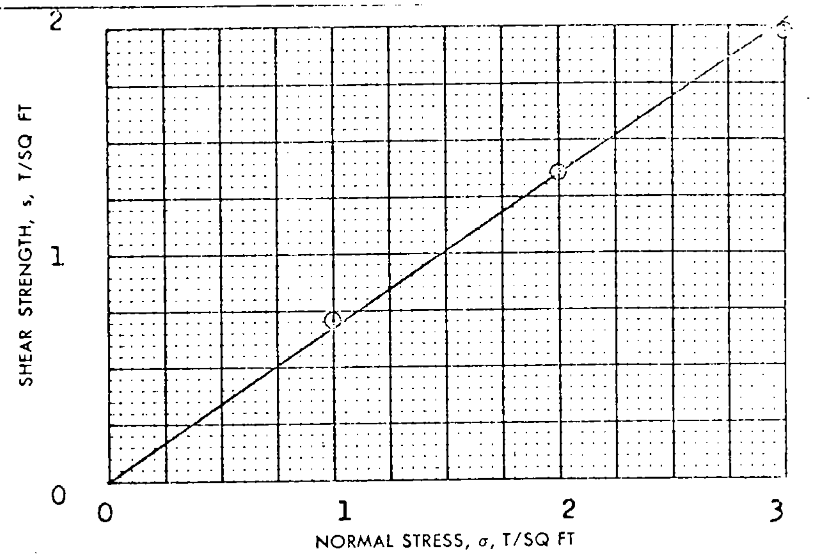
AREA

BORING NO. X-10U — SAMPLE NO. 13-D

~~DEPTH~~ EL -80 MSL — DATE 8 February,1972

WJH **DIRECT SHEAR TEST REPORT**





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	22.4%	21.9 %	21.1%	%
INITIAL	VOID RATIO	e_o	0.647	0.620	0.620	
INITIAL	SATURATION	S_o	92.1%	94.0 %	90.5 %	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	100.8	102.5	102.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	23.2 %	22.4%	22.9%	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.71	1.35	1.98	
ACTUAL TIME TO FAILURE, MIN		t_f	660	780	660	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 34°

TAN ϕ' = 0.675

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 0.550 IN. THICK

CLASSIFICATION SILTY SAND(SM),gray

LL - PL - PI - G_s 2.66

REMARKS

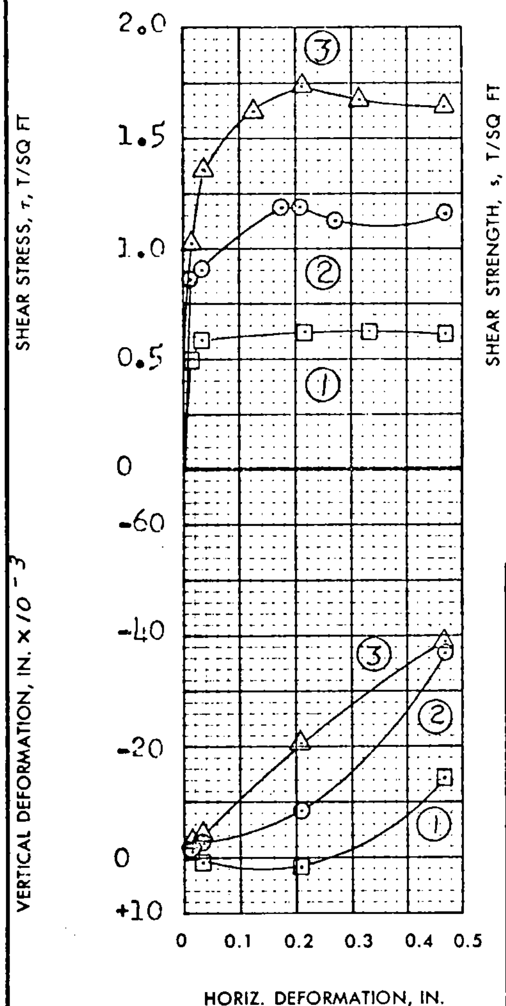
PROJECT LK.PONT.,LA.,& VIC.- HURR. PROT. RIGOLETS CONTROL STRUCT. & CLOSURE DAM DDM # 6(1971)

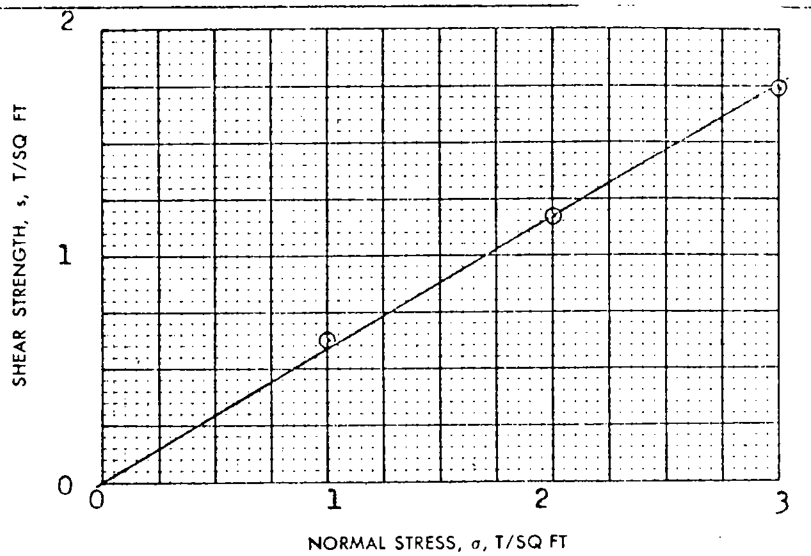
AREA

BORING NO. X3-U SAMPLE NO. 6-D

~~DEPTH~~ EL - 40.8 MSL DATE 16 June, 1971

JHMc **DIRECT SHEAR TEST REPORT**





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	24.5 %	24.1 %	24.5 %	%
	VOID RATIO	e_o	0.719	0.710	0.757	
	SATURATION	S_o	91.7 %	91.3 %	87.1 %	%
	DRY DENSITY, LB/CU FT	γ_d	97.7	98.2	96.7	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	24.5%	25.1%	25.8 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.63	1.18	1.74	
ACTUAL TIME TO FAILURE, MIN		t_f	2010	1140	1320	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 31°

TAN ϕ' = 0.59

c' = 0 T/SQ FT

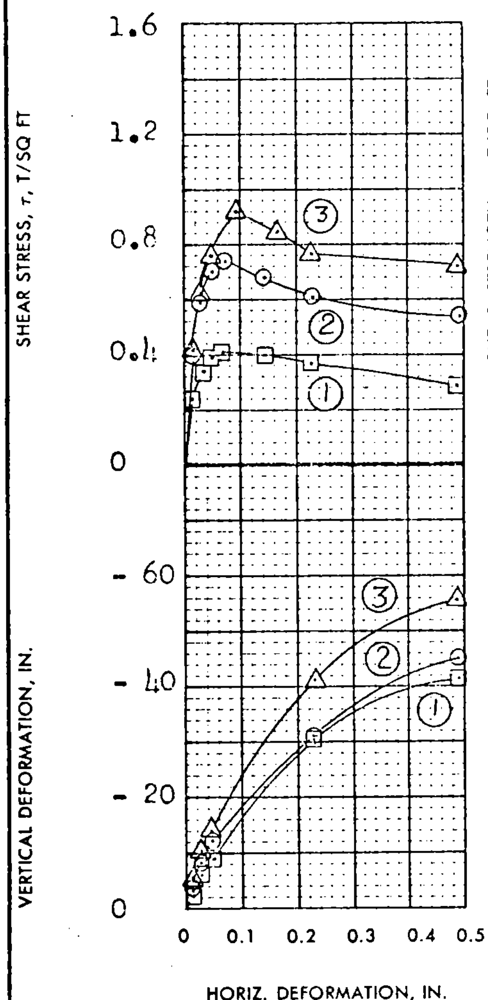
☐ CONTROLLED STRESS

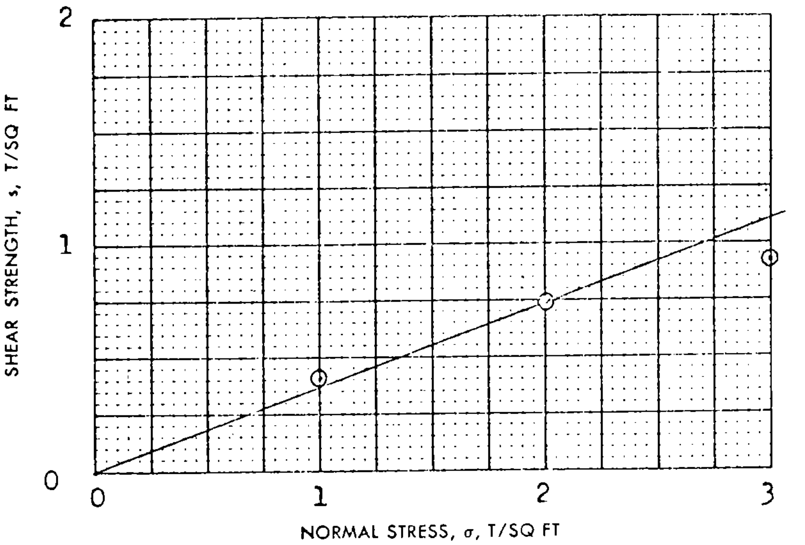
☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.550 IN. THICK
CLASSIFICATION SILTY SAND(SM),gray, contains finely divided organic matter			
LL -	PL -	PI -	G_s 2.69

REMARKS

PROJECT	LK.PONT.LA.,& VIC.- HURR. PROT.- RIGOLETS CONTROL STRUCT. AND CLOSURE DAM
AREA	D.D.M. # 6(1971)
BORING NO. X 3-U	SAMPLE NO. 15 - C
DEPTH / EL - 76.0 MSL	DATE 21 June 1971
JHMc	**DIRECT SHEAR TEST REPORT**





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	62.3 %	57.3 %	64.2 %	%
	VOID RATIO	e_o	1.77	1.65	1.80	
	SATURATION	S_o	96.1 %	94.8 %	97.4 %	%
	DRY DENSITY, LB/CU FT	γ_d	61.6	64.2	60.8	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	7	9	10	
FINAL	WATER CONTENT	w_f	60.6 %	54.4 %	50.3 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.41	0.74	0.92	
ACTUAL TIME TO FAILURE, MIN		t_f	480	510	660	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 20°

TAN ϕ' = 0.37

c' = 0 T/SQ FT

[ ] CONTROLLED STRESS

[X] CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED		3.00 IN. SQUARE	0.550 IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH), gray			
LL 105	PL 26	PI 79		G_s 2.73

REMARKS

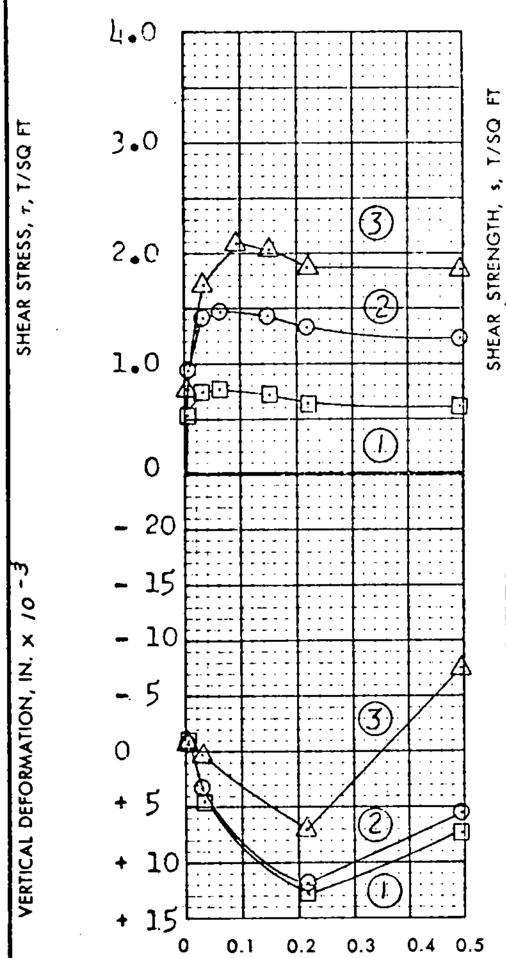
PROJECT LK.PONT.,LA.,&VIC.-HURR. PROT.-RIGOLETS CONTROL STR. & CLOSURE DAM; D D M # 6(1971)

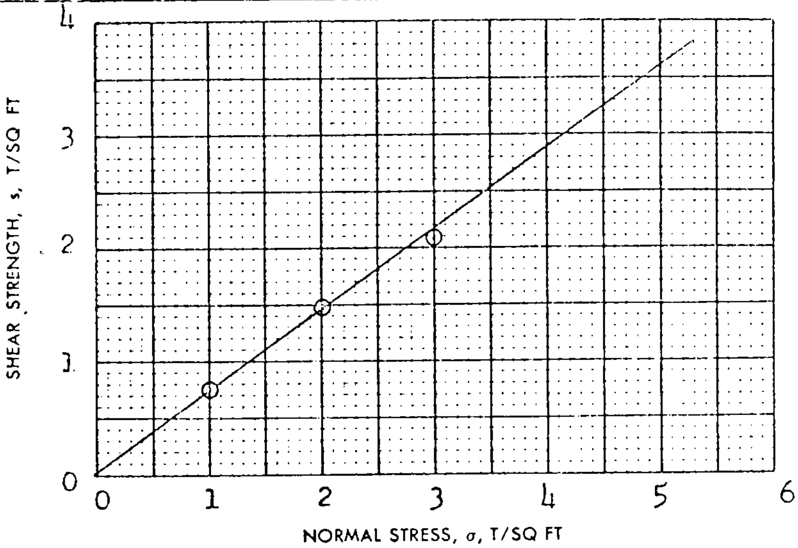
AREA

BORING NO. X 3-U — SAMPLE NO. 19-B

DEPTH EL - 90.9 MSL — DATE 23 June 1971

JHMc **DIRECT SHEAR TEST REPORT**





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	22.2 %	21.7 %	22.1 %	%
	VOID RATIO	e_o	0.582	0.561	0.558	
	SATURATION	S_o	100+ %	100+ %	100+ %	%
	DRY DENSITY, LB/CU FT	γ_d	98.9	100.2	100.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	18.9 %	19.2 %	20.2 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.76	1.48	2.09	
ACTUAL TIME TO FAILURE, MIN		t_f	450	450	630	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 36°

TAN ϕ' = 0.74

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.550 IN. THICK

CLASSIFICATION SAND(SP)F, off-white, contains pockets of clay and finely divided*

LL – | PL – | PI – | G_s 2.65

REMARKS *organic matter.

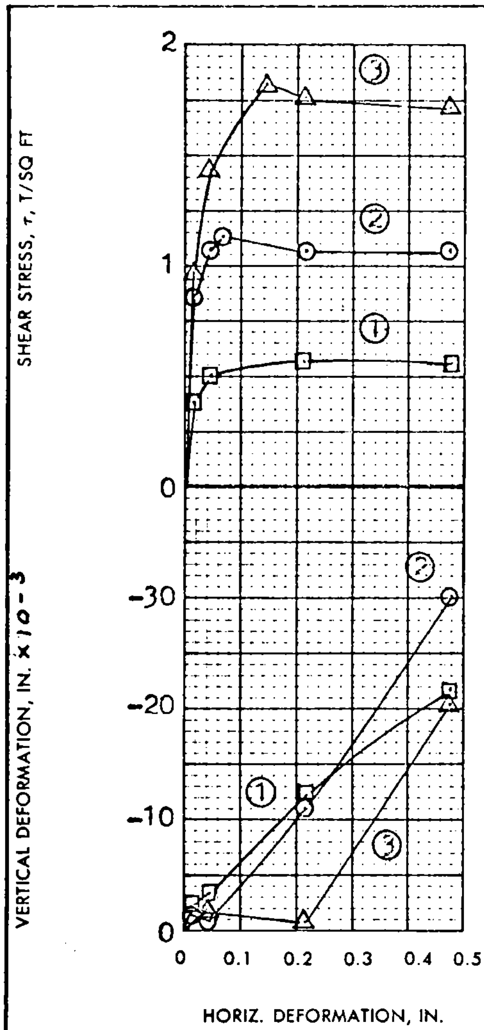
PROJECT LK.PONT.LA.,&VIC.- HURR.PROT.-RIGOLETS CONTROL STR. & CLOSURE DAM, D D M # 6

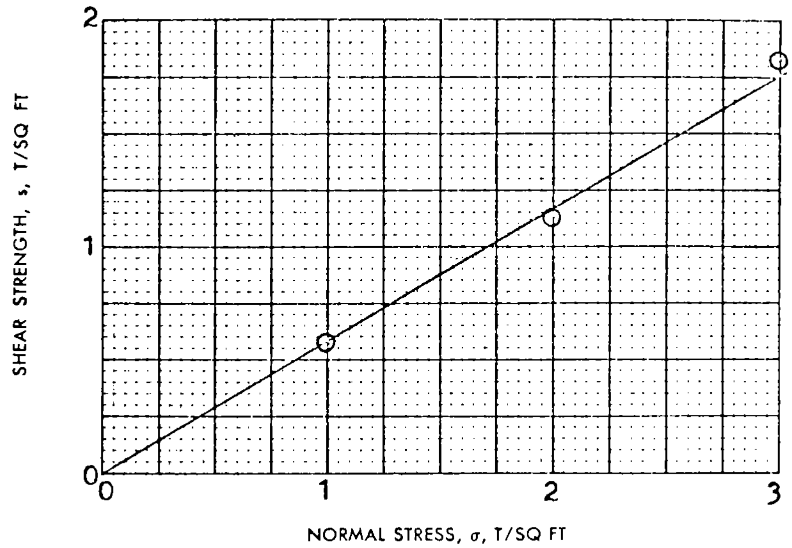
AREA (1971)

BORING NO. X 5-U — SAMPLE NO. 1-B

~~DEPTH~~ EL - 21.9 MSL — DATE 28 June 1971

WJH — **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

ϕ' = 30°

TAN ϕ' = 0.57

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	25.5 %	26.0 %	26.0 %	%
INITIAL	VOID RATIO	e_o	0.786	0.789	0.752	
INITIAL	SATURATION	S_o	88.6 %	88.3 %	92.6 %	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	93.7	93.5	95.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1	<1	
FINAL	WATER CONTENT	w_f	27.1 %	26.8 %	20.8 %	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.57	1.13	1.83	
ACTUAL TIME TO FAILURE, MIN		t_f	1320	510	930	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.550 IN. THICK

CLASSIFICATION SILTY SAND(SM),gray, contains thin layers of plastic clay

LL - | PL - | PI - | G_s 2.68

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.

RIGOLETS CONTROL STR.& CLOSURE,DDM#6 (1971)

AREA

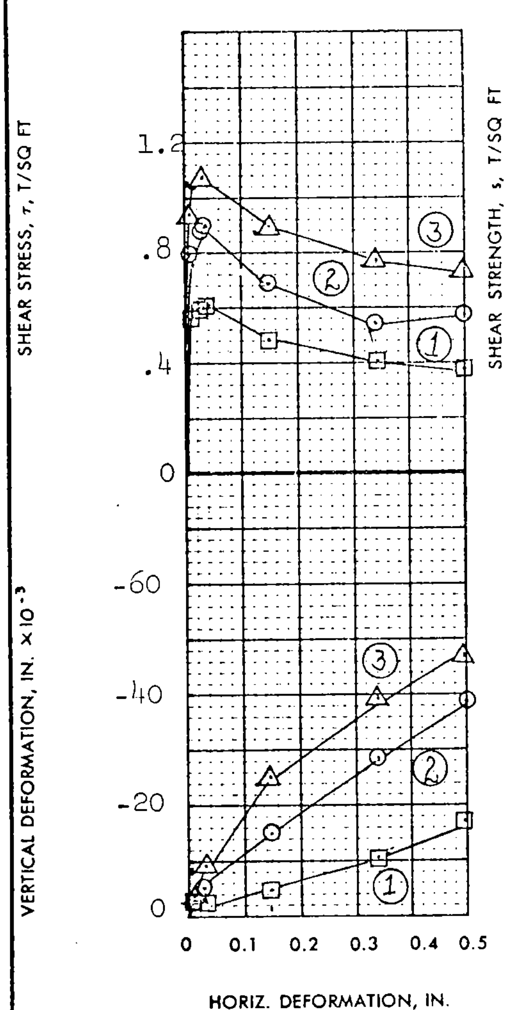
BORING NO. X5-U — SAMPLE NO. 11-B

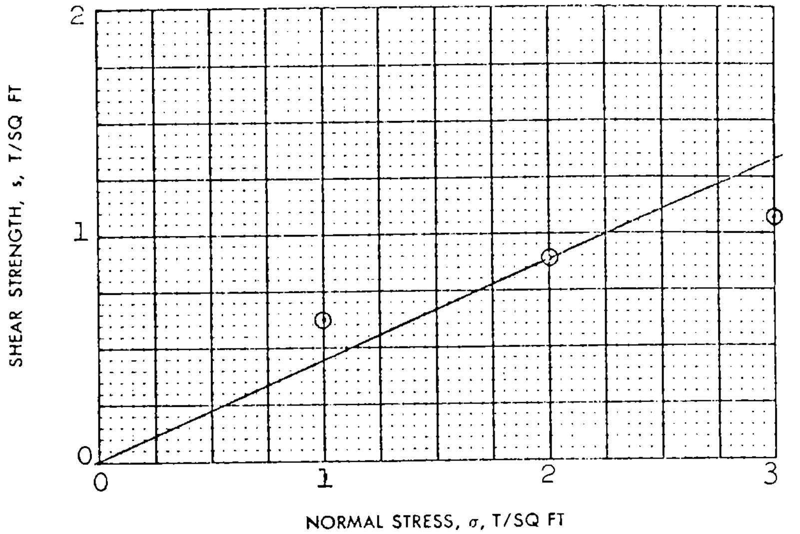
~~DEPTH~~ EL -61.9 MSL — DATE 1 July,1971

WJH — DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) USAEWES SOIL

Fig. A - 9





SHEAR STRENGTH PARAMETERS

ϕ' = 24°

TAN ϕ' = 0.445

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	47.8%	50.5%	52.2%	%
	VOID RATIO	e_o	1.37	1.40	1.45	
	SATURATION	S_o	95.9%	99.2%	99.0%	%
	DRY DENSITY, LB/CU FT	γ_d	72.4	71.6	70.0	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	3	3	
FINAL	WATER CONTENT	w_f	49.4%	48.1%	48.6%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.61	0.89	1.07	
ACTUAL TIME TO FAILURE, MIN		t_f	450	330	360	
RATE OF STRAIN, IN./MIN			.00012	.00012	.00012	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.550 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, fissured

LL 104 | PL 27 | PI 77 | G_s 2.75

REMARKS

PROJECT LK. PONT. LA., & VIC.-HURR. PROTECTION-RIGOLETS CONTROL STRUCT. & CLOSURE DAM; DDM

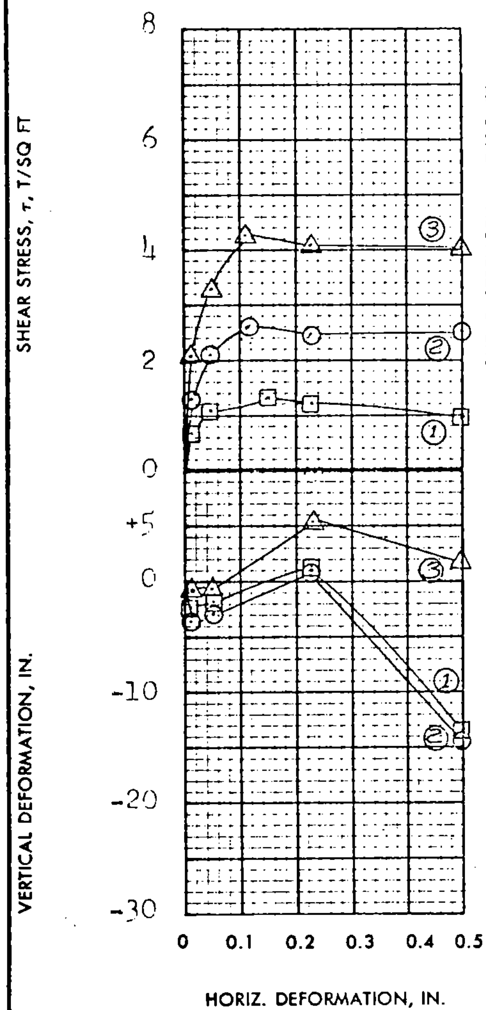
AREA NO. 6 (1971)

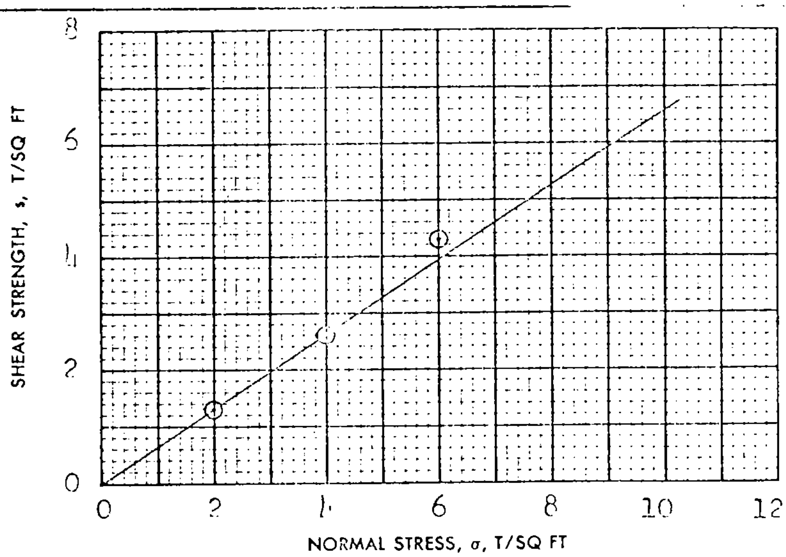
BORING NO. X5-U — SAMPLE NO. 16-C

~~DEPTH~~ EL -83.2 MSL — DATE 31 August 1971

BWG

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

$\phi' = 33^\circ$

TAN $\phi' = 0.65$

$c' = 0$ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	21.2 %	21.6 %	22.9 %	%
	VOID RATIO	e_o	0.667	0.674	0.684	
	SATURATION	S_o	84.9 %	85.6 %	89.4 %	%
	DRY DENSITY, LB/CU FT	γ_d	100.0	99.6	99.0	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1	1	
FINAL	WATER CONTENT	w_f	23.6 %	22.9 %	23.2 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.34	2.60	4.22	
ACTUAL TIME TO FAILURE, MIN		t_f	930	750	720	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.544 IN. THICK

CLASSIFICATION SAND (SP)F, gray, with a trace of silt; contains plastic clay lenses.

LL – PL – PI – G_s 2.67

REMARKS CORRECTED REPORT
8 February 1972

PROJECT LK. PONT., LA.&VIC: HURRICANE PROTECTION
RIGOLETS CONTROL STR. & CLOSURE DAM:DDM No.6

AREA (1971)

BORING NO. X-7-U SAMPLE NO. 3-R

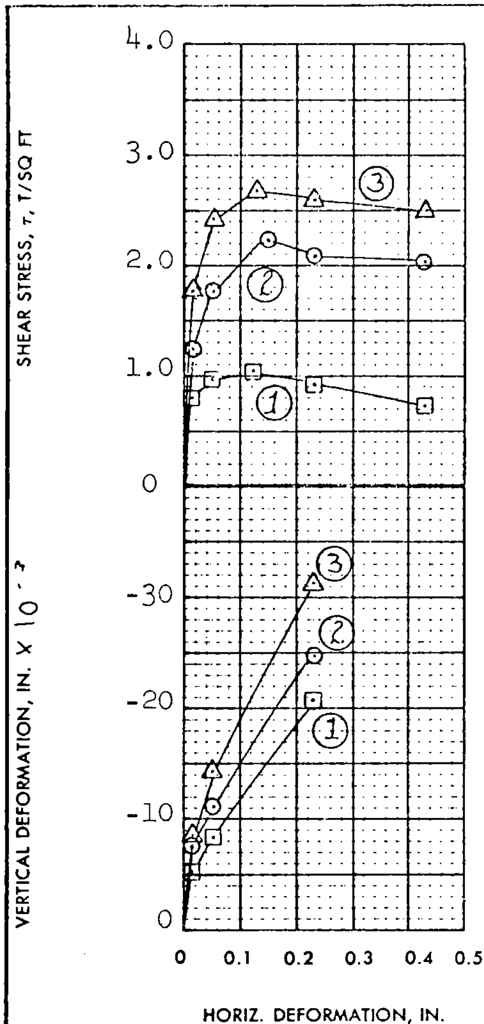
DEPTH EL -42.7 MSL DATE 20 October 1971

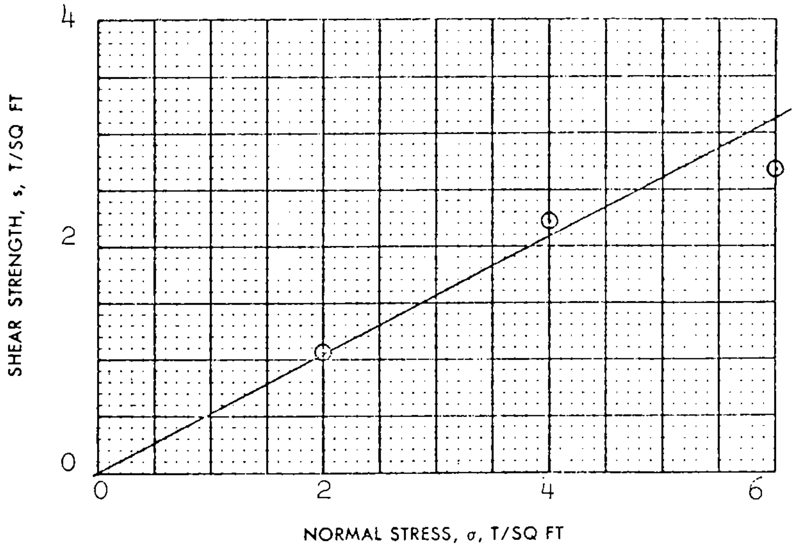
BWG

DIRECT SHEAR TEST REPORT

Fig. A - 11

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—2





SHEAR STRENGTH PARAMETERS

ϕ' = 28°

TAN ϕ' = 0.525

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	34.3 %	34.9 %	35.8 %	%
	VOID RATIO	e_o	0.921	0.924	0.977	
	SATURATION	S_o	100+ %	100+ %	100+ %	%
	DRY DENSITY, LB/CU FT	γ_d	88.7	88.6	86.2	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	< 1	
FINAL	WATER CONTENT	w_f	31.7 %	30.7 %	31.3 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.05	2.21	2.69	
ACTUAL TIME TO FAILURE, MIN		t_f	750	900	750	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION Alternate seams of PLASTIC CLAY (CH) and CLAYEY SILT (ML), gray.			
LL 48	PL 19	PI 29	G_s 2.73

REMARKS Atterberg limits on mixture of materials.

PROJECT LK. PONT., LA.& VIC.; HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM; DDM NO.6

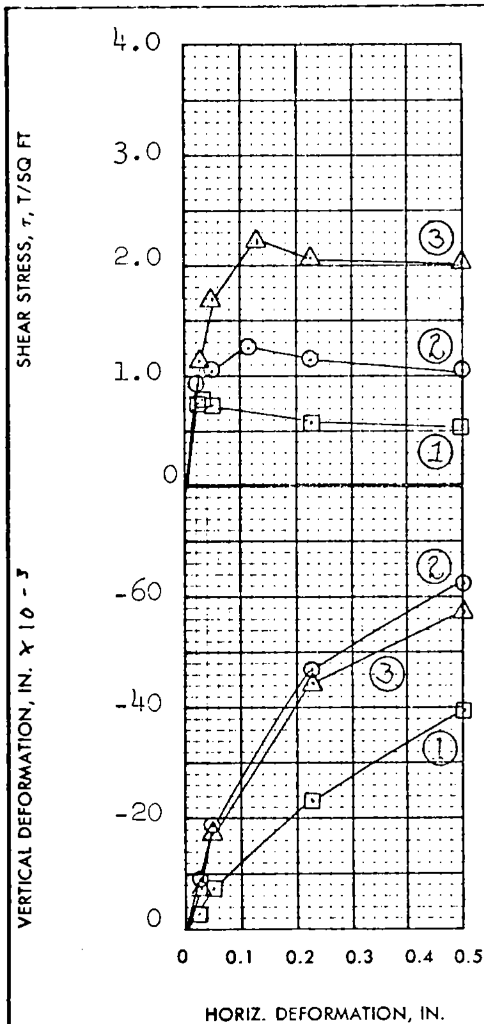
AREA (1971)

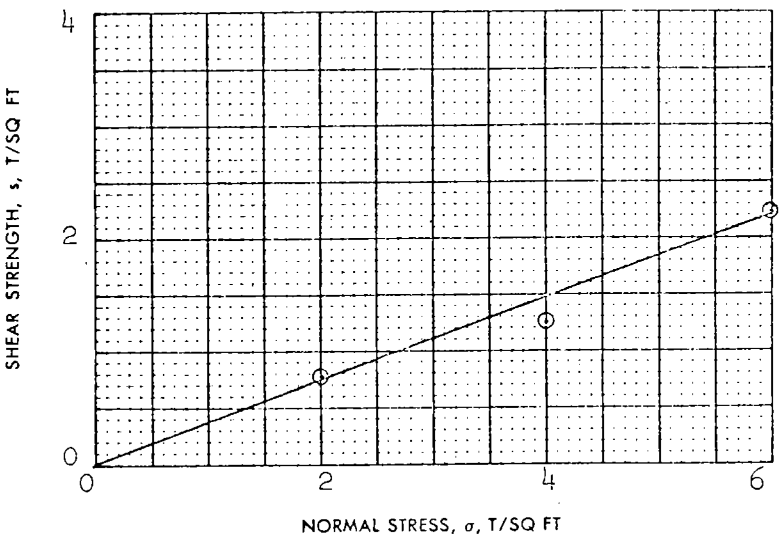
BORING NO. X-7-U — SAMPLE NO. 12-A

DEPTH / EL -78.1 MSL — DATE 21 Oct. 1971

BWG

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 20°

TAN ϕ' = 0.368

c' = 0 T/SQ FT

[ ] CONTROLLED STRESS

[X] CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	45.0%	42.6%	42.8 %	%
	VOID RATIO	e_o	1.23	1.20	1.18	
	SATURATION	S_o	100+ %	95.9 %	99.4 %	%
	DRY DENSITY, LB/CU FT	γ_d	76.7	77.9	78.3	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1	1	
FINAL	WATER CONTENT	w_f	46.9%	32.7%	38.8%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.78	1.25	2.21	
ACTUAL TIME TO FAILURE, MIN		t_f	210	720	810	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

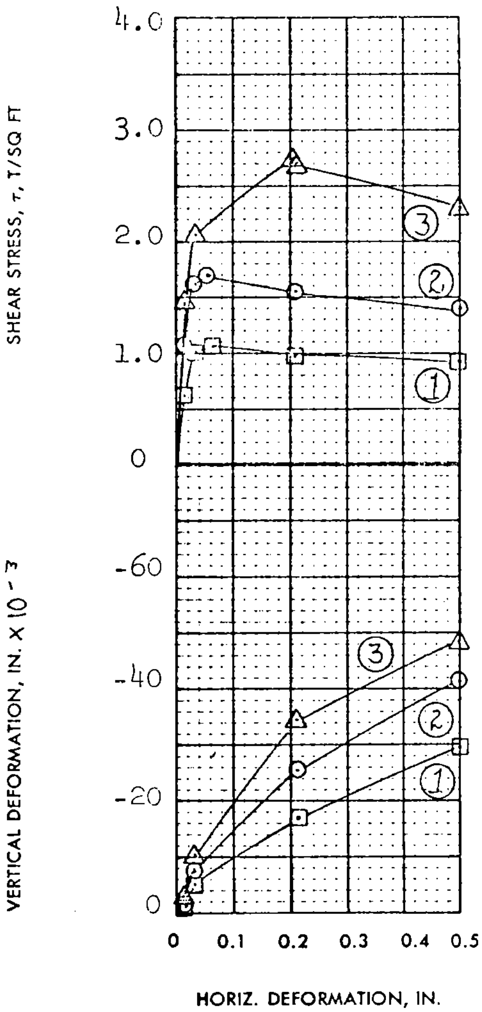
TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION	PLASTIC CLAY (CH), gray, fissured.		
LL 75	PL 24	PI 51	G_s 2.74

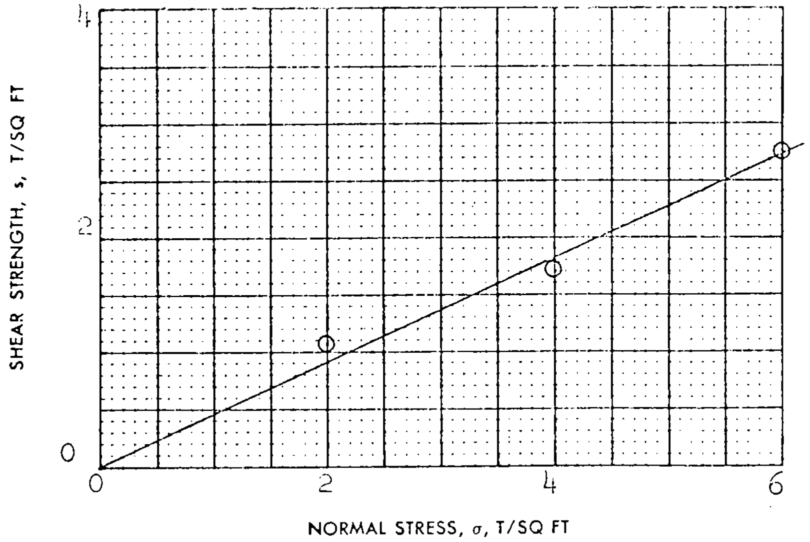
REMARKS

PROJECT	LK. PONT., LA.&VIC.;HURR. PROT.
	RIGOLETS CONT. STR. AND CLOSURE DAM, DDM NO.6
AREA	(1971)
BORING NO. X-7-U	SAMPLE NO. 20-D
EL -112.2 MSL	DATE 26 Oct. 1971

BWG

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 24°

TAN ϕ' = 0.455

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	39.0 %	35.5 %	39.6 %	%
	VOID RATIO	e_o	1.09	1.05	1.07	
	SATURATION	S_o	97.3 %	92.0 %	100+ %	%
	DRY DENSITY, LB/CU FT	γ_d	81.1	82.7	82.1	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	2	<1	< 1	
FINAL	WATER CONTENT	w_f	37.8 %	37.3%	31.6%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.06	1.70	2.73	
ACTUAL TIME TO FAILURE, MIN		t_f	510	480	1230	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	0.544 IN. THICK	
CLASSIFICATION	PLASTIC CLAY (CH), gray, containing silt strata.			
LL 59	PL 22	PI 37		G_s 2.72

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.-
RIGOLETS CONT. STRUCTURE & CLOSURE DAM,DDM NO.6

AREA (1971)

BORING NO. X-7-U — SAMPLE NO. 41-C

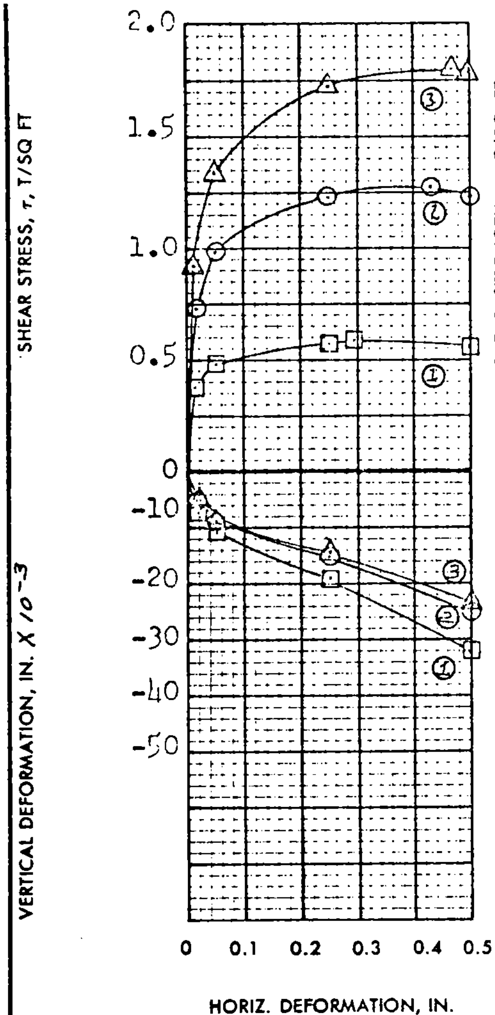
~~DEPTH~~ EL -195.4 MSL — DATE 27 Oct. 1971

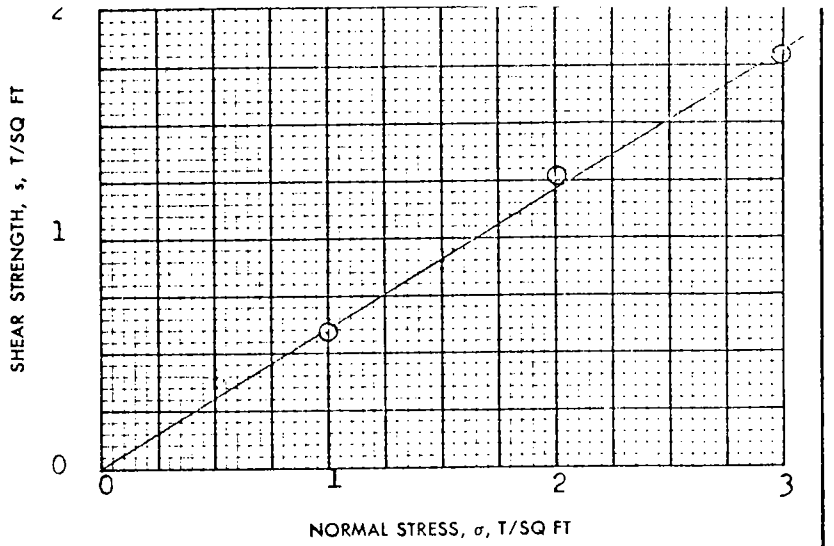
BWG

DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF —

Fig. A - 14





SHEAR STRENGTH PARAMETERS

ϕ' = 31°

TAN ϕ' = 0.6125

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	29.0 %	27.9 %	28.8 %	%
	VOID RATIO	e_o	0.793	0.737	0.730	
	SATURATION	S_o	97.3 %	100+ %	100+ %	%
	DRY DENSITY, LB/CU FT	γ_d	92.6	95.6	96.0	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1.0	< 1.0	< 1.0	
FINAL	WATER CONTENT	w_f	21.1 %	19.9 %	19.7 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.58	1.26	1.80	
ACTUAL TIME TO FAILURE, MIN		t_f	1560	2220	2160	
RATE OF STRAIN, IN./MIN			.00019	.00019		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN: UNDISTURBED		3.00 IN. SQUARE	0.539 IN. THICK
CLASSIFICATION: SILTY SAND(SM),dark gray, contains pockets of plastic clay			
LL –	PL –	PI –	G_s 2.66

REMARKS

PROJECT LK. PONT., LA.&VIC. - HURR. PROT.(1971)

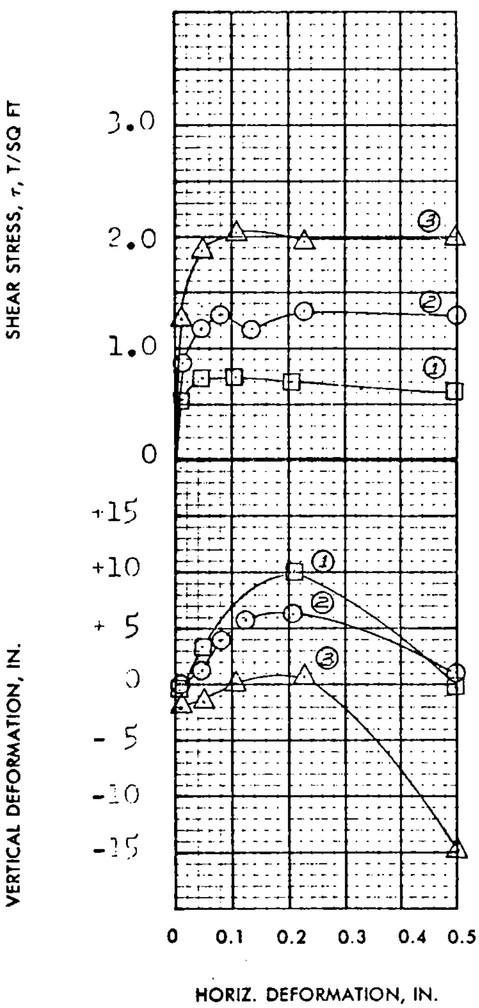
RIGOLETS CONTROL STR. & CLOSURE DAM;DDM No.6

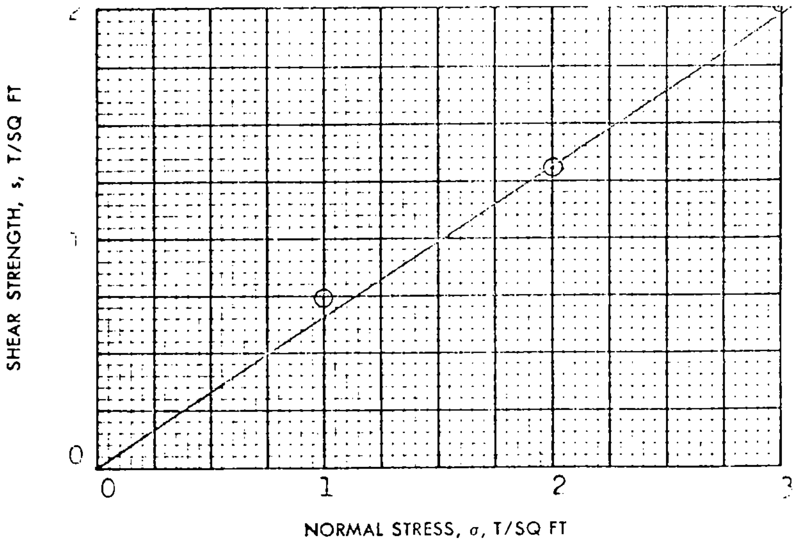
AREA

BORING NO. X-8U SAMPLE NO. 1-D

DEPTH/EL -24.3 MSL DATE 14 December 1971

CDA **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

φ' = 33°

TAN φ' = 0.651

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	26.9%	25.8%	25.5%	%
	VOID RATIO	e_o	0.746	0.741	0.741	
	SATURATION	S_o	95.9%	92.6%	91.5%	%
	DRY DENSITY, LB/CU FT	γ_d	95.1	95.4	95.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1.0	< 1.0	< 1.0	
FINAL	WATER CONTENT	w_f	25.1 %	25.4%	26.7%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.74	1.30	2.01	
ACTUAL TIME TO FAILURE, MIN		t_f	660	540	660	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 0.537 IN. THICK

CLASSIFICATION SILTY SAND(SM),gray

LL - PL - PI - G_s 2.66

REMARKS

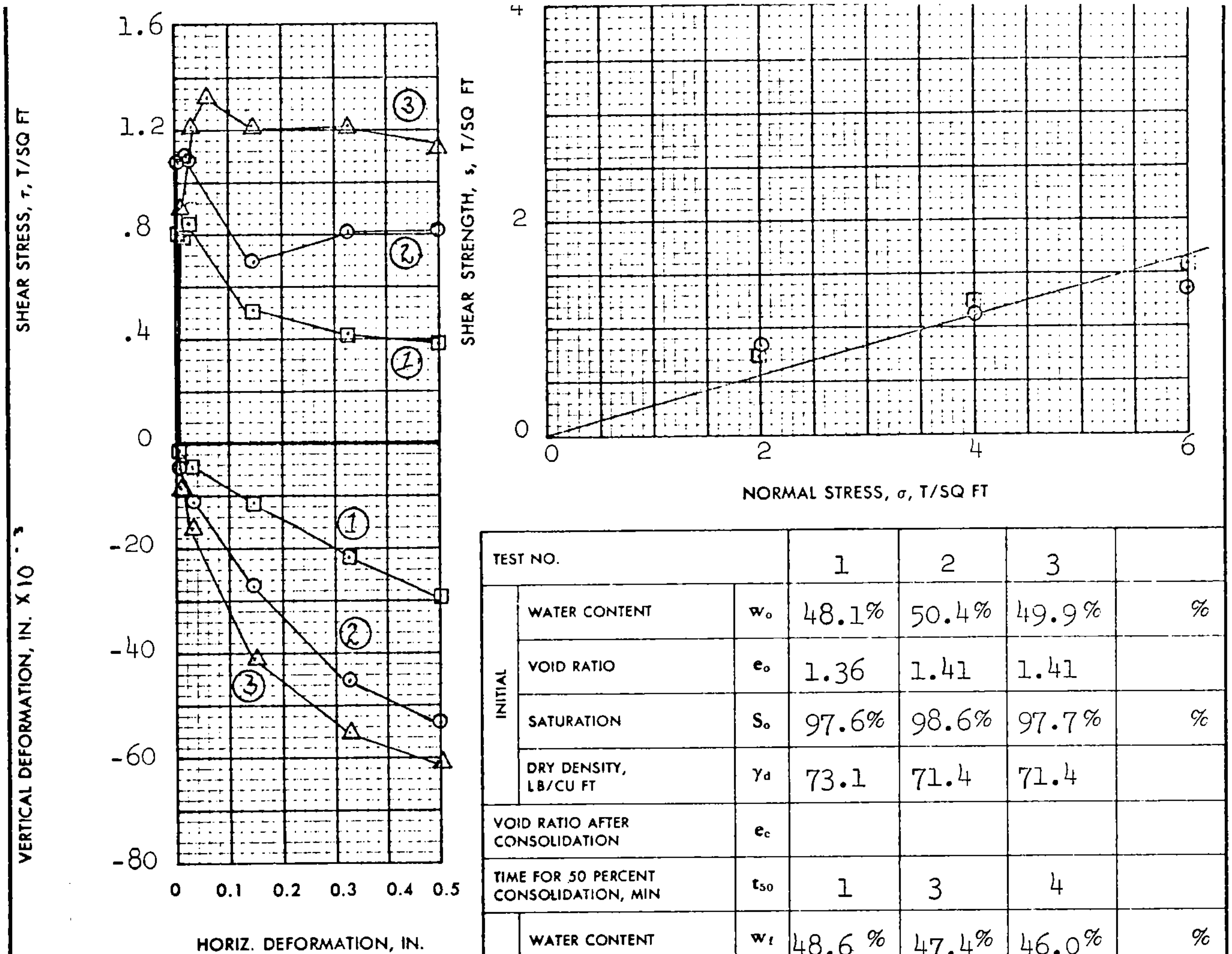
PROJECT LK. PONT.,LA. & VICINITY-HURR. PROT.(1971)
RIGOLETS CONTROL STR.& CLOSURE DAM;DDM.No.6

AREA

BORING NO. X-8U SAMPLE NO. 5-D

DEPTH/EL -40.3 MSL DATE 15 December 1971

GDA **DIRECT SHEAR TEST REPORT**



SHEAR STRENGTH PARAMETERS

ϕ' = 15°

TAN ϕ' = 0.275

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	48.1%	50.4%	49.9%	%
INITIAL	VOID RATIO	e_o	1.36	1.41	1.41	
INITIAL	SATURATION	S_o	97.6%	98.6%	97.7%	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	73.1	71.4	71.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	3	4	
FINAL	WATER CONTENT	w_f	48.6 %	47.4%	46.0%	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.84	1.10	1.33	
ACTUAL TIME TO FAILURE, MIN		t_f	240	270	660	
RATE OF STRAIN, IN./MIN			.00012	.00012	.00012	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, with traces of fine sand

LL 100 | PL 29 | PI 71 | G_s 2.76

REMARKS Rerun test.
☐ Strength values of original test

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO.6

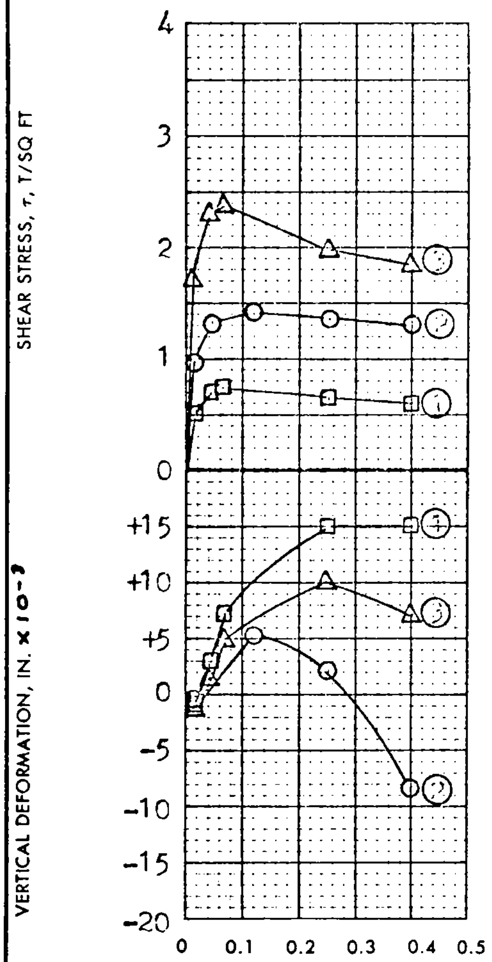
AREA

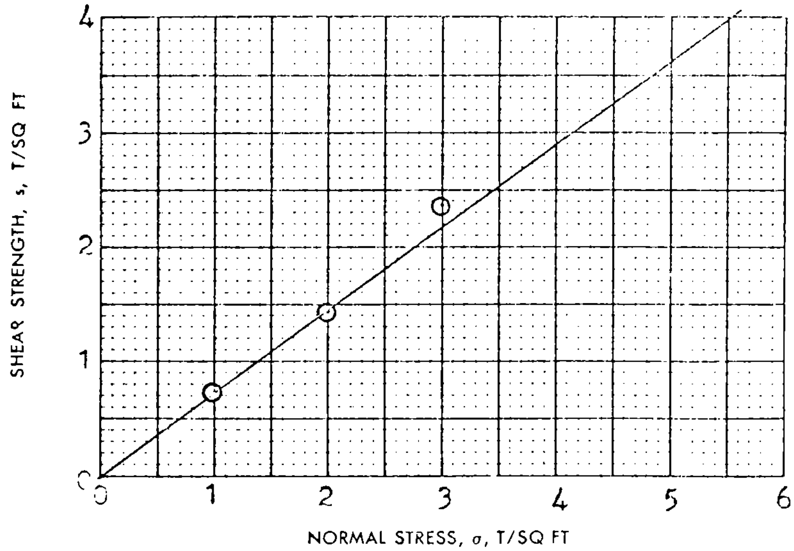
BORING NO. X-8-U — SAMPLE NO. 15-B

~~DEPTH~~ EL -73.7 MSL — DATE 8 Feb. 1972

WJH

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 36°

TAN ϕ' = 0.72

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	20.4 %	20.0 %	20.0 %	%
	VOID RATIO	e_o	0.621	0.626	0.604	
	SATURATION	S_o	87.7 %	85.3 %	88.4 %	%
	DRY DENSITY, LB/CU FT	γ_d	102.8	102.5	103.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	18.0 %	16.1 %	16.4 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.74	1.44	2.38	
ACTUAL TIME TO FAILURE, MIN		t_f	480	720	480	
RATE OF STRAIN, IN./MIN			.00015	.00015	.00015	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED	3.00 in.Square	0.538	IN. THICK
CLASSIFICATION SAND (SP)F, tan			
LL –	PL –	PI –	G_s 2.67

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR. CONTROL DAM; DDM # 6

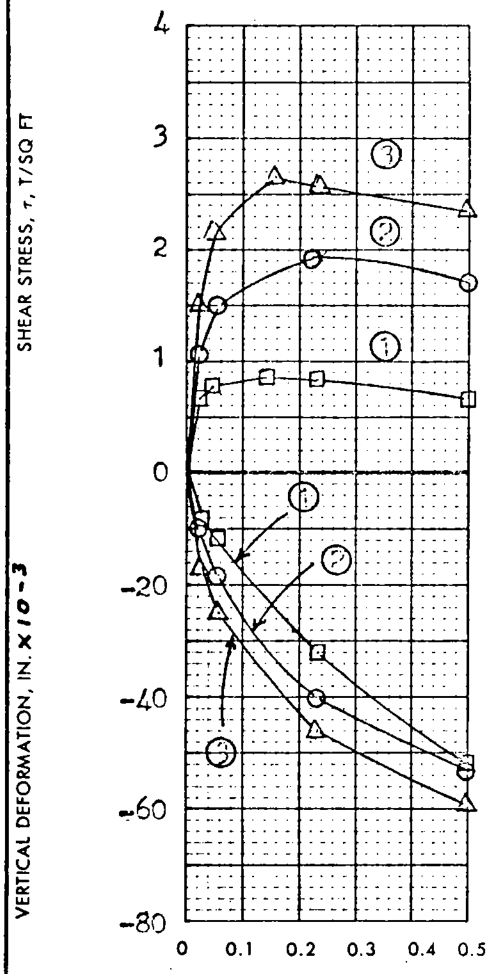
AREA

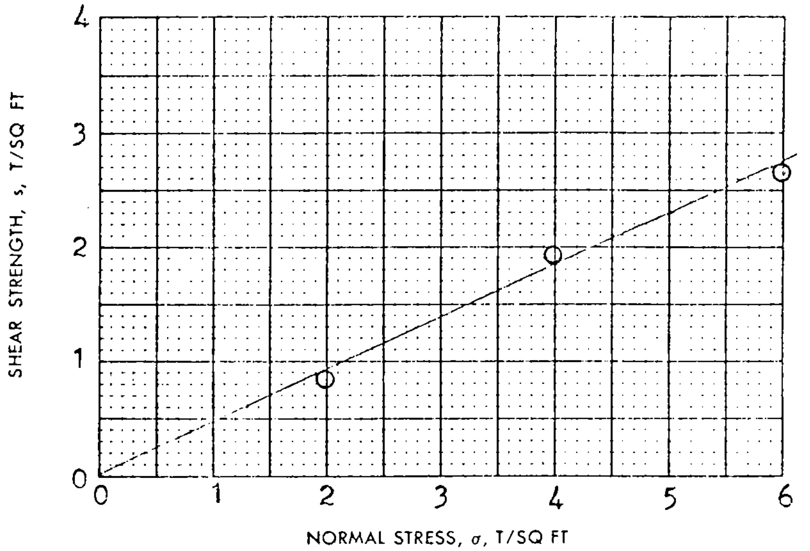
BORING NO. X-9U SAMPLE NO. 4-D

~~DEPTH~~ EL -44.4 MSL DATE 21 December,1971

GDA DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—





SHEAR STRENGTH PARAMETERS

ϕ' = 25°

TAN ϕ' = 0.467

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	38.5 %	41.9%	43.2%	%
	VOID RATIO	e_o	1.13	1.13	1.13	
	SATURATION	S_o	92.7%	100+ %	100+ %	%
	DRY DENSITY, LB/CU FT	γ_d	79.7	79.6	79.7	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1	2	
FINAL	WATER CONTENT	w_f	43.6 %	35.7%	32.2%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.85	1.93	2.65	
ACTUAL TIME TO FAILURE, MIN		t_f	840	1260	900	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 1=0.546; 2&3=0.625 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH), gray, contains SAND(SM) & SILT(ML) strata

LL 56 | PL 18 | PI 38 | G_s 2.72

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT. (1971)

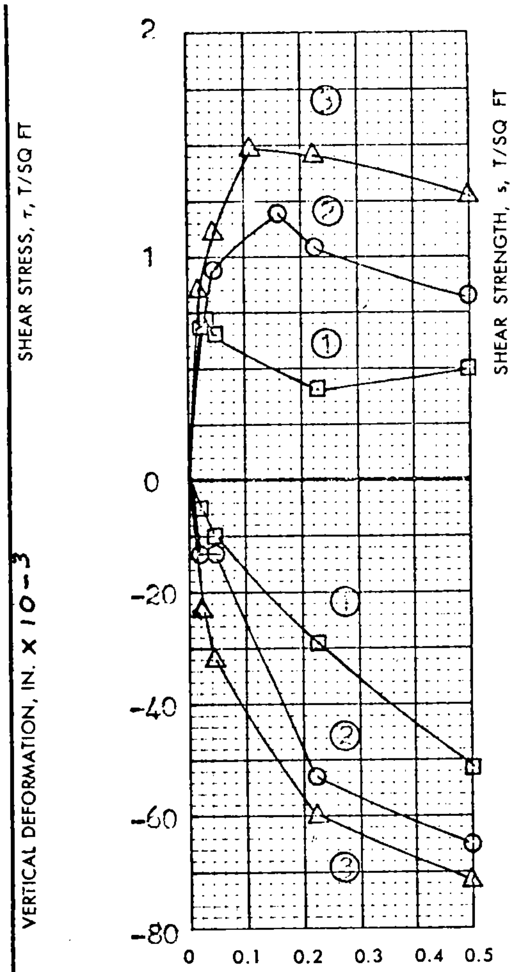
RIGOLETS CONTROL STR.& CLOSURE DAM; DDM # 6

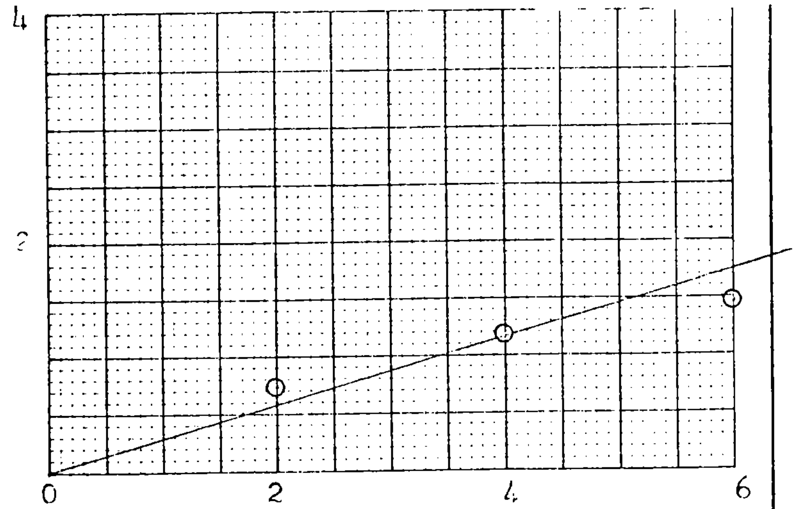
AREA

BORING NO. X-9U | SAMPLE NO. 8-D

~~DEPTH~~ EL -60.5 MSL | DATE 16 December, 1971

BWG DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 16°

TAN ϕ' = 0.294

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	60.9 %	61.0%	61.2%	%
	VOID RATIO	e_o	1.72	1.71	1.71	
	SATURATION	S_o	97.4 %	98.1%	98.4%	%
	DRY DENSITY, LB/CU FT	γ_d	63.1	63.3	63.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	7	3	8	
FINAL	WATER CONTENT	w_f	58.1 %	48.5%	41.9%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.72	1.19	1.48	
ACTUAL TIME TO FAILURE, MIN		t_f	240	960	720	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH), gray

LL 101 | PL 27 | PI 74 | G_s 2.75

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.(1971)

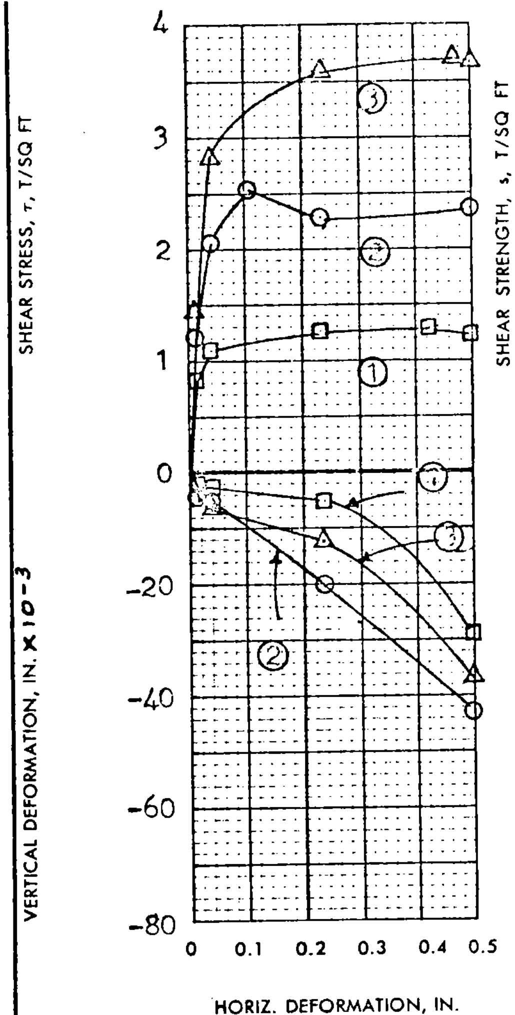
RIGOLETS CONTROL STR. & CLOSURE DAM; DDM #6

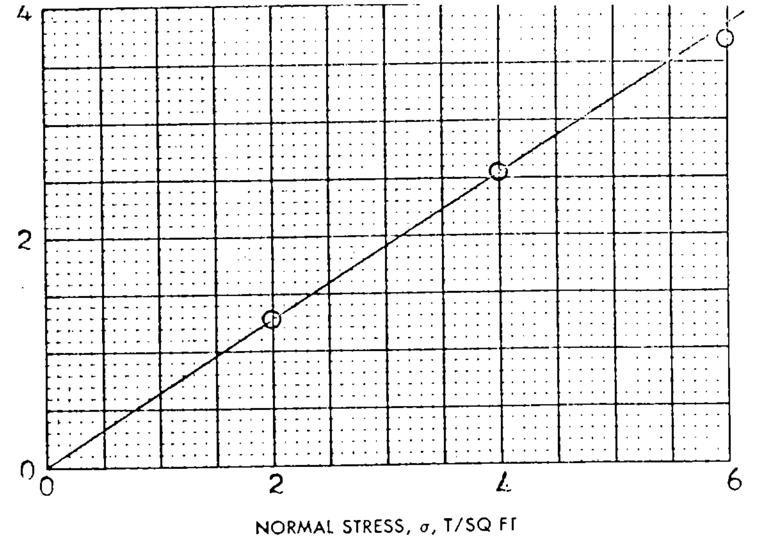
AREA

BORING NO. X-9U — SAMPLE NO. 15-A

~~DEPTH~~ EL -85.7 MSL — DATE 28 December, 1971

GDA — **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

ϕ' = 33°

TAN ϕ' = 0.645

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

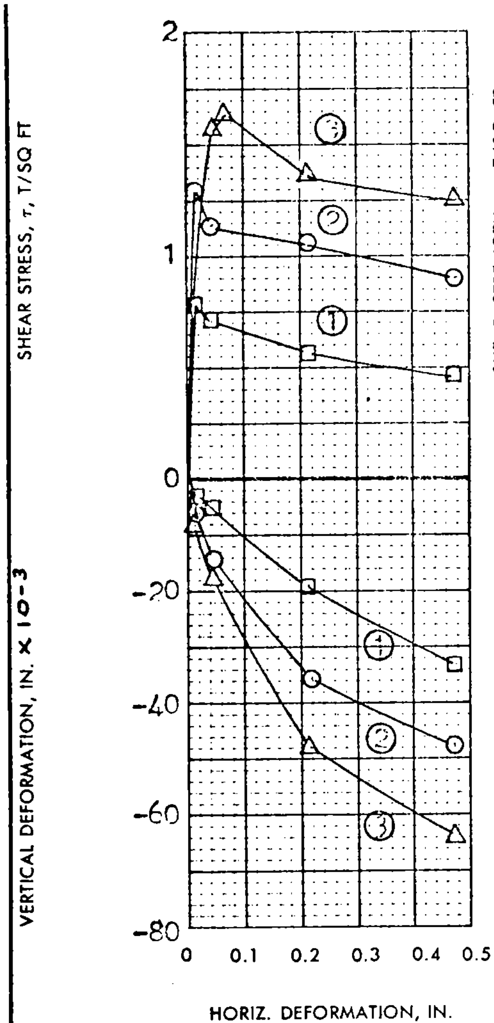
☒ CONTROLLED STRAIN

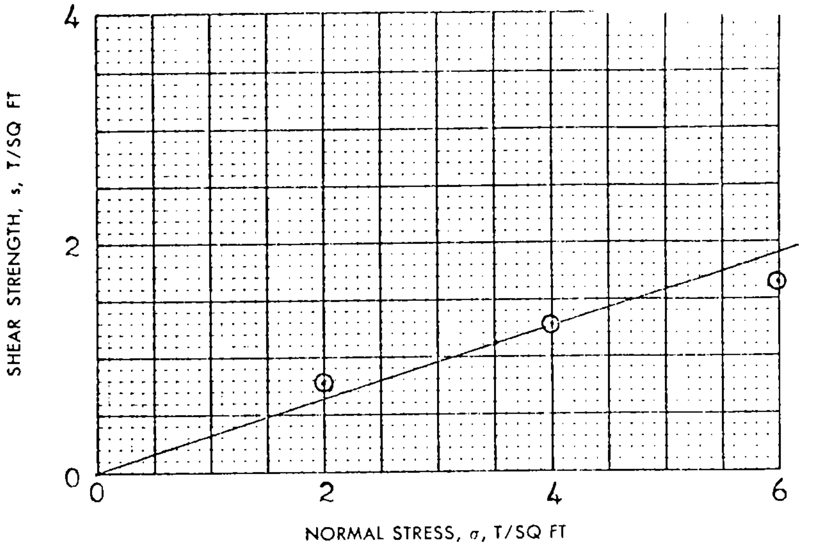
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.4 %	23.5 %	23.9 %	%
	VOID RATIO	e_o	0.672	0.670	0.658	
	SATURATION	S_o	91.6 %	92.2 %	95.5 %	%
	DRY DENSITY, LB/CU FT	γ_d	98.2	98.3	99.0	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	25.4 %	24.7 %	24.3 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.29	2.53	3.70	
ACTUAL TIME TO FAILURE, MIN		t_f	2310	690	2550	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	0.546 IN. THICK
CLASSIFICATION	SAND (SP)F, light gray		
LL -	PL -	PI -	G_s 2.63

REMARKS

PROJECT	LK.PONT.,LA.& VIC.-HURR. PROT.(1971)	
	RIGOLETS CONTROL STR.& CLOSURE DAM; DDM #6	
AREA		
BORING NO. X-11U		SAMPLE NO. 5-D
~~DEPTH~~ EL -50.8 MSL		DATE 6 January,1972
BWG	DIRECT SHEAR TEST REPORT	





TEST NO.		1	2	3	
INITIAL: WATER CONTENT	w_o	52.2%	50.4%	50.8%	%
INITIAL: VOID RATIO	e_o	1.46	1.41	1.41	
INITIAL: SATURATION	S_o	99.0%	99.0%	99.8%	%
INITIAL: DRY DENSITY, LB/CU FT	γ_d	70.2	71.8	71.7	
VOID RATIO AFTER CONSOLIDATION	e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN	t_{50}	1	2	3	
FINAL: WATER CONTENT	w_f	52.0%	47.0%	43.7%	%
FINAL: VOID RATIO	e_f				
FINAL: SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT	σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT	τ_{max}	0.78	1.29	1.64	
ACTUAL TIME TO FAILURE, MIN	t_f	180	180	480	
RATE OF STRAIN, IN./MIN		.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT	τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 17°

TAN ϕ' = 0.314

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, slickensided

LL 98 | PL 32 | PI 66 | G_s 2.77

REMARKS

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT. (1971)

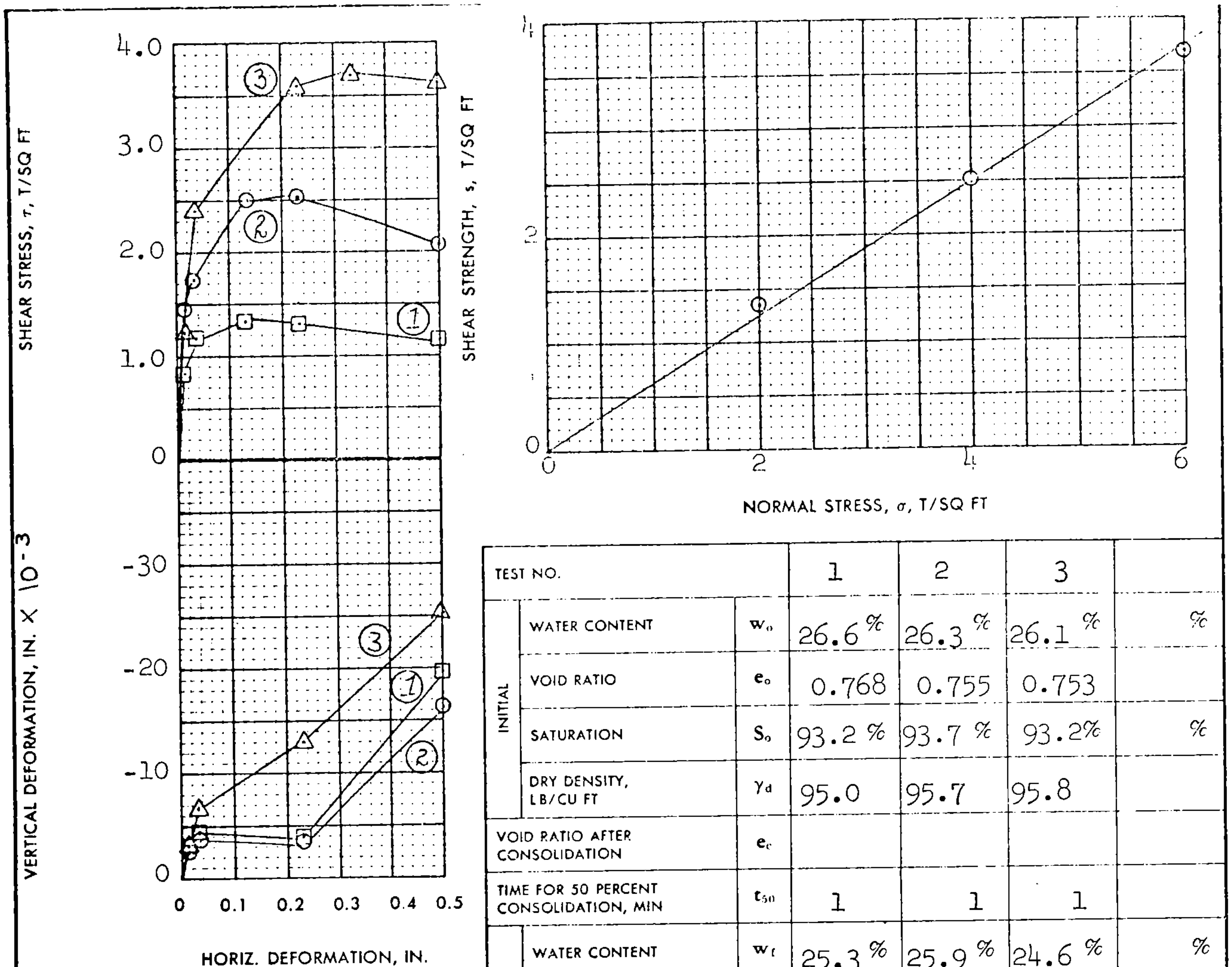
RIGOLETS CONTROL STR.& CLOSURE DAM; DDM #6

AREA

BORING NO. X-11U — SAMPLE NO. 12-D

~~DEPTH~~ EL -78.5 MSL — DATE 11 January,1972

GDA — **DIRECT SHEAR TEST REPORT**



SHEAR STRENGTH PARAMETERS

ϕ' = 32°

TAN ϕ' = 0.625

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	26.6 %	26.3 %	26.1 %	%
INITIAL	VOID RATIO	e_o	0.768	0.755	0.753	
INITIAL	SATURATION	S_o	93.2 %	93.7 %	93.2%	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	95.0	95.7	95.8	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1	1	
FINAL	WATER CONTENT	w_f	25.3 %	25.9 %	24.6 %	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.33	2.50	3.70	
ACTUAL TIME TO FAILURE, MIN		t_f	810	840	1830	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 0.539 IN. THICK

CLASSIFICATION SILTY SAND(SM),gray, contains a few small shells and traces of *

LL - PL - PI - G_s 2.69

REMARKS *organic matter

PROJECT LK. PONT.LA.&VIC.-HURR. PROT.-(1971)

RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM;

AREA DDM NO.6.

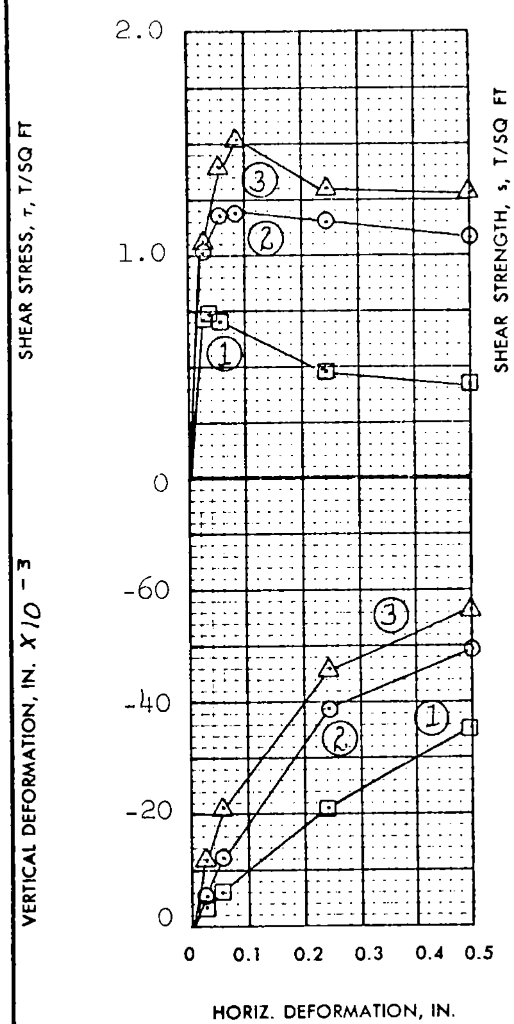
BORING NO. X-12-U SAMPLE NO. 7-A

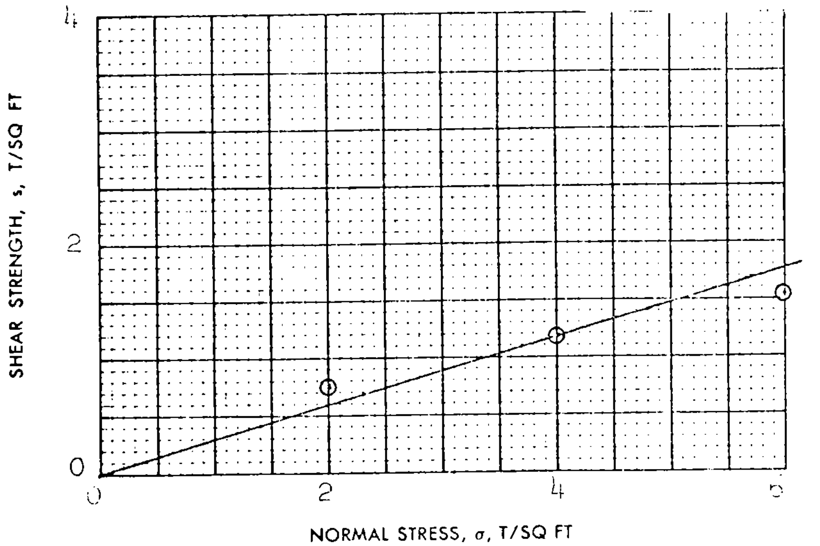
DEPTH EL -57.2 MSL DATE 28 Sept. 1971

WJH

DIRECT SHEAR TEST REPORT SOILS TEST SECTION

Fig. A - 23





SHEAR STRENGTH PARAMETERS

ϕ' = 17°

TAN ϕ' = 0.298

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

			1	2	3	
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	40.8 %	40.2 %	39.4 %	%
INITIAL	VOID RATIO	e_o	1.14	1.12	1.10	
INITIAL	SATURATION	S_o	98.1 %	98.3 %	98.1 %	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	80.1	80.7	81.2	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	2	2	
FINAL	WATER CONTENT	w_f	44.2 %	40.5 %	36.0 %	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.74	1.19	1.53	
ACTUAL TIME TO FAILURE, MIN		t_f	270	480	540	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN	UNDISTURBED	3.00	IN. SQUARE	0.539	IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH),gray				
LL 70	PL 23	PI 47		G_s 2.74	

REMARKS

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.-(1971)
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM;

AREA DDM NO.6.

BORING NO. X-12-U | SAMPLE NO. 34-C

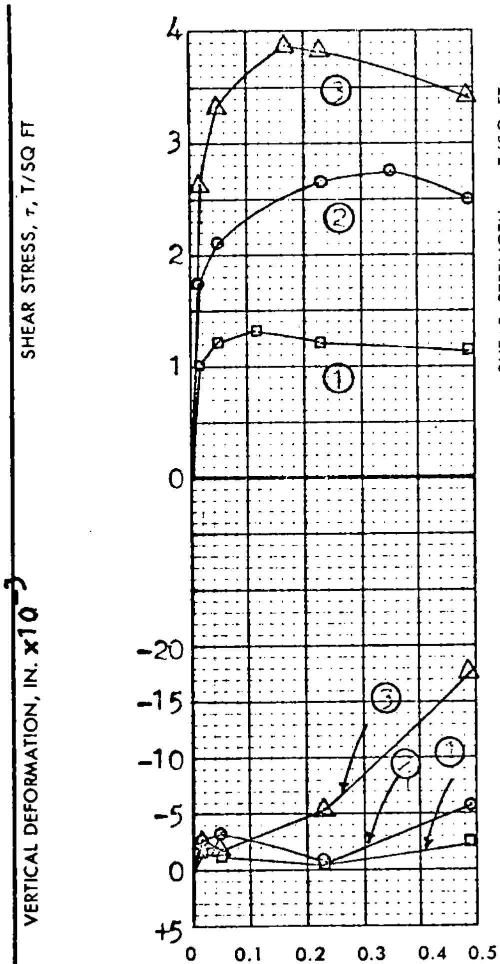
~~DEPTH~~ EL -167.0 MSL | DATE 7 October 1971

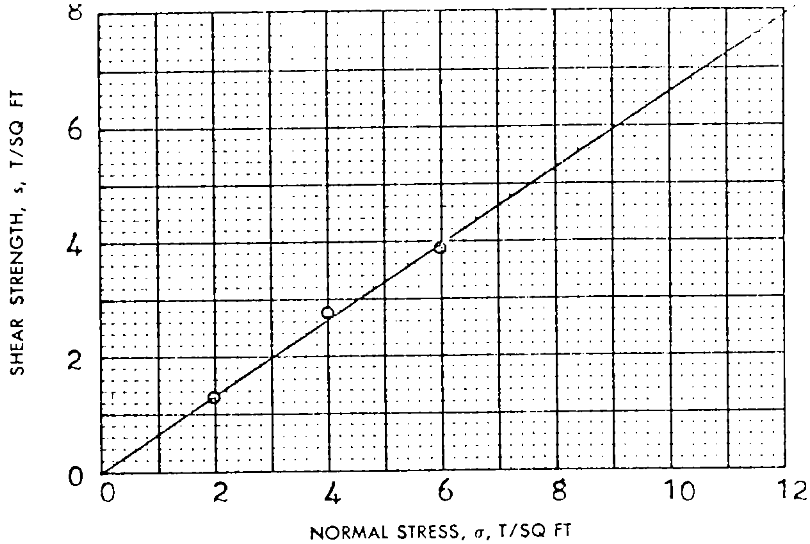
WJH
BWG

DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—

Fig. A - 24





SHEAR STRENGTH PARAMETERS

ϕ' = 33°

TAN ϕ' = 0.655

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

			1	2	3	
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.2 %	21.8 %	22.3 %	%
	VOID RATIO	e_o	0.615	0.572	0.610	
	SATURATION	S_o	100+ %	100+ %	98.0 %	%
	DRY DENSITY, LB/CU FT	γ_d	103.6	106.4	103.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	23.4 %	23.4 %	23.1 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.31	2.76	3.88	
ACTUAL TIME TO FAILURE, MIN		t_f	720	2110	990	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	0.544 IN. THICK	
CLASSIFICATION	SAND(SP)F, gray, contains CLAY(CH) lenses			
LL –	PL –	PI –		G_s 2.68

REMARKS

PROJECT LK.PONT.,LA.& VIC.-HURR. PROT. -(1971)

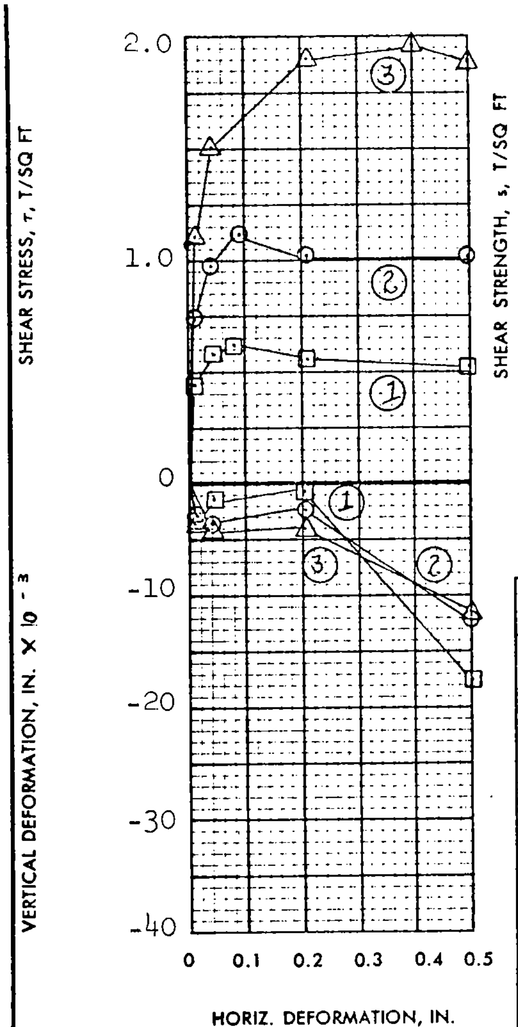
RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM

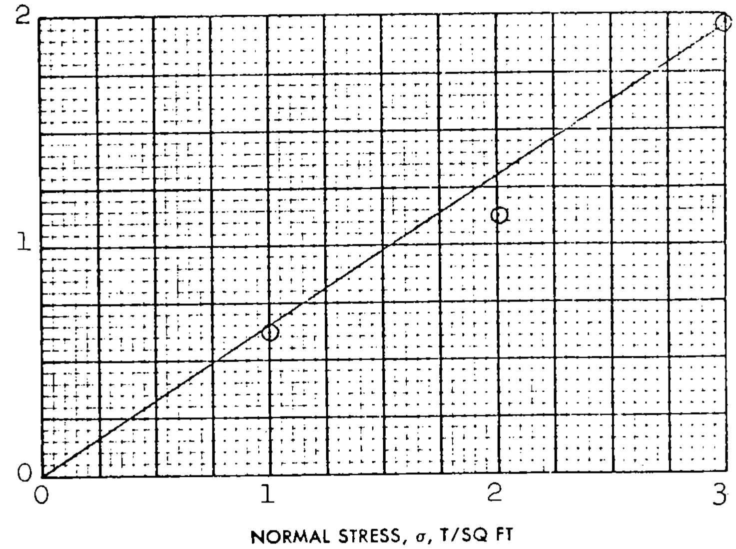
AREA DDM NO. 6

BORING NO. X-12U SAMPLE NO. 36-C

~~DEPTH~~ EL -175.0 MSL DATE 29 September, 1971

EG **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

ϕ' = 33°

TAN ϕ' = 0.653

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.9 %	23.8 %	24.1 %	%
	VOID RATIO	e_o	0.642	0.625	0.652	
	SATURATION	S_o	99.0 %	100+ %	98.3 %	%
	DRY DENSITY, LB/CU FT	γ_d	101.1	102.2	100.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	< 1	
FINAL	WATER CONTENT	w_f	20.9 %	21.7 %	21.4%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.63	1.13	1.96	
ACTUAL TIME TO FAILURE, MIN		t_f	600	660	2400	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN: UNDISTURBED		3.00 IN. SQUARE	0.538 IN. THICK
CLASSIFICATION: SAND(SP),F gray;			
LL -	PL -	PI -	G_s 2.66

REMARKS

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO. 6

AREA

BORING NO. X-13-U — SAMPLE NO. 1-D

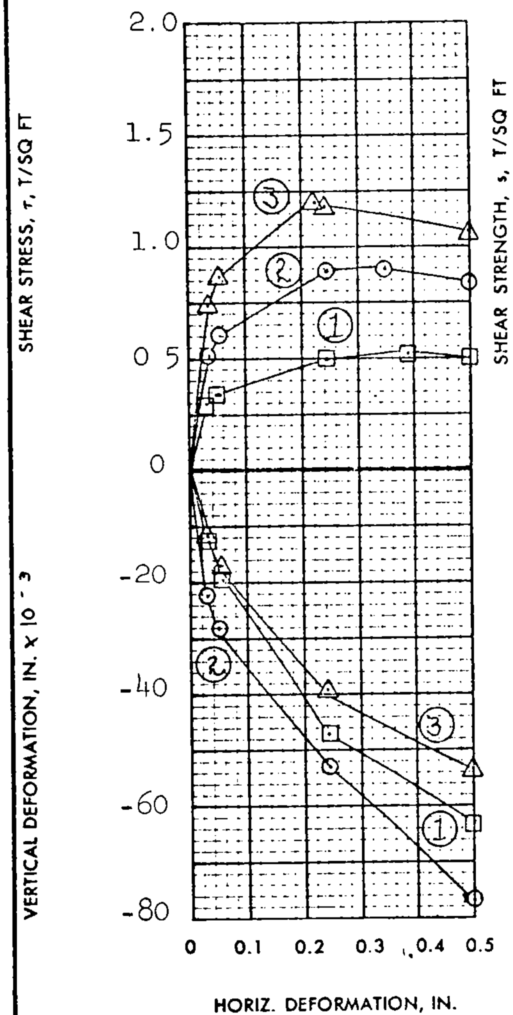
DEPTH/EL -27.4 MSL — DATE 1/17/72

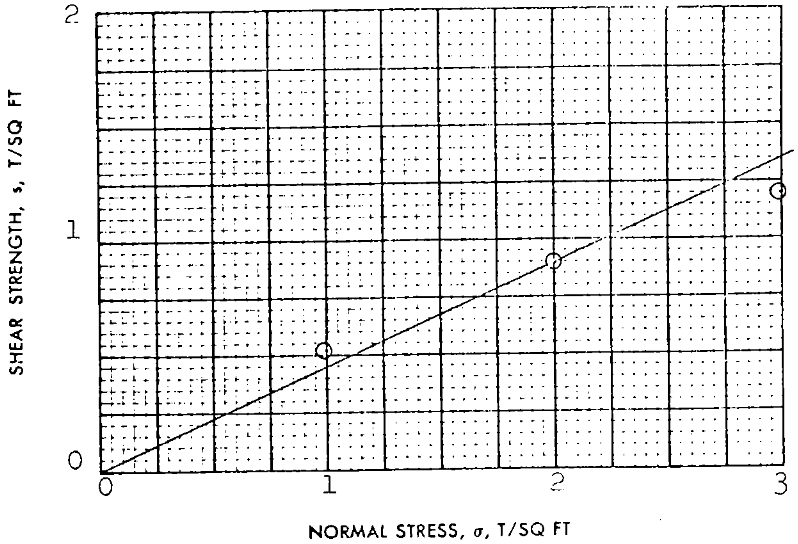
GDA

DIRECT SHEAR TEST REPORT

Fig. A - 26

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF





SHEAR STRENGTH PARAMETERS

ϕ' = 24°

TAN ϕ' = 0.45

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	79.8%	82.2%	80.6%	%
	VOID RATIO	e_o	2.17	2.25	2.33	
	SATURATION	S_o	98.9%	98.3%	93.1%	%
	DRY DENSITY, LB/CU FT	γ_d	53.0	51.7	50.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	4	16	16	
FINAL	WATER CONTENT	w_f	54.1%	46.9%	49.4%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.52	0.90	1.20	
ACTUAL TIME TO FAILURE, MIN		t_f	2130	1920	1260	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED | 3.00 IN. SQUARE | 1= 0.546, 2&3=0.625 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, fissured

LL 112 | PL 27 | PI 85 | G_s 2.69

REMARKS

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO. 6

AREA

BORING NO. X-13-U | SAMPLE NO. 4-D

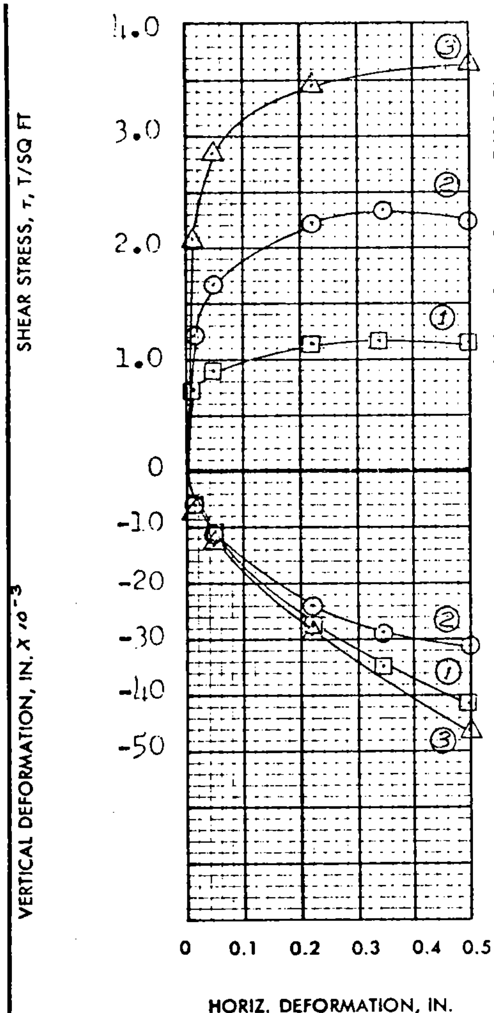
~~DEPTH~~ EL -39.6 MSL | DATE 11 Jan. 1972

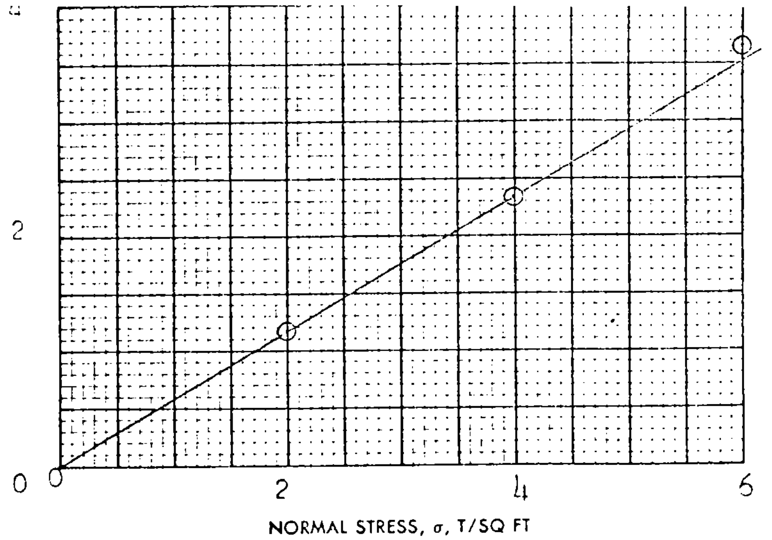
BWG

DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—

Fig. A - 27





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	30.5%	30.9%	31.0%	%
	VOID RATIO	e_o	0.867	0.836	0.842	
	SATURATION	S_o	93.9%	98.7%	98.3%	%
	DRY DENSITY, LB/CU FT	γ_d	89.3	90.8	90.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1	<1	
FINAL	WATER CONTENT	w_f	27.2%	25.4%	24.3%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2	4	6	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.17	2.35	3.64	
ACTUAL TIME TO FAILURE, MIN		t_f	1980	1980	2760	
RATE OF STRAIN, IN./MIN			100018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 30°

TAN ϕ' = 0.585

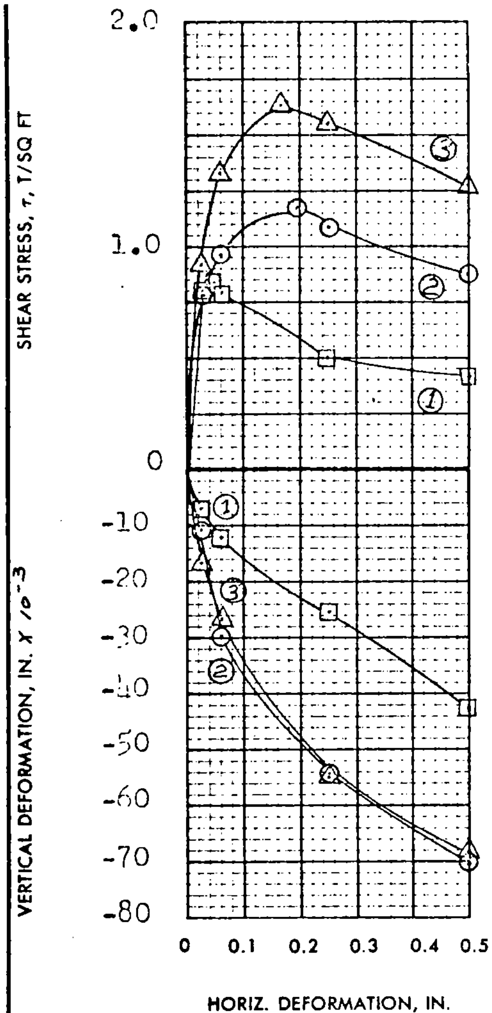
c' = 0 T/SQ FT

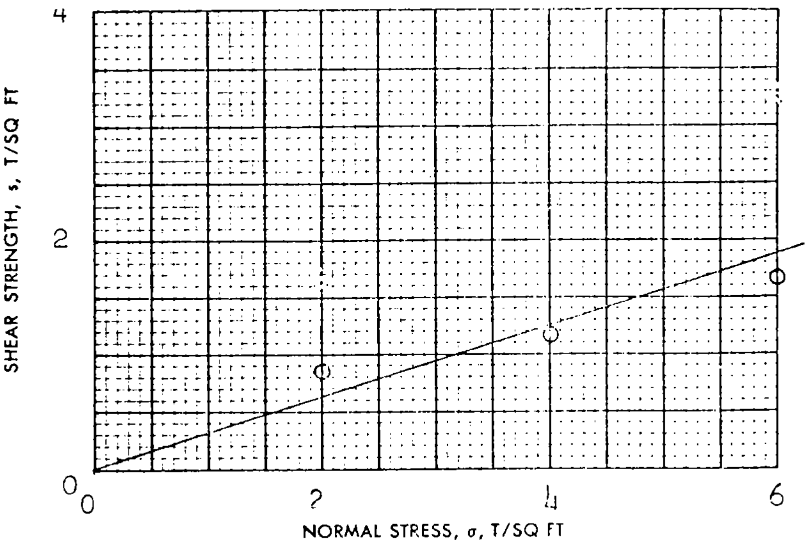
☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	0.538 IN. THICK
CLASSIFICATION	SANDY CLAY(CL),gray, contains shell fragments		
LL 30	PL 18	PI 12	G_s 2.67

REMARKS	PROJECT	LK. PONT., LA.& VIC. - HURR PROT.(1971) RIGOLETS CONTROL STR. & CLOSURE DAM;DDM NO. 6	
	AREA		
	BORING NO.	X-13U	SAMPLE NO. 8-C
	~~DEPTH~~ EL	-54.4 MSL	DATE 12 January 1972
	GDA	**DIRECT SHEAR TEST REPORT**	





SHEAR STRENGTH PARAMETERS

ϕ' = 17°

TAN ϕ' = 0.314

c' = 0 T/SQ FT

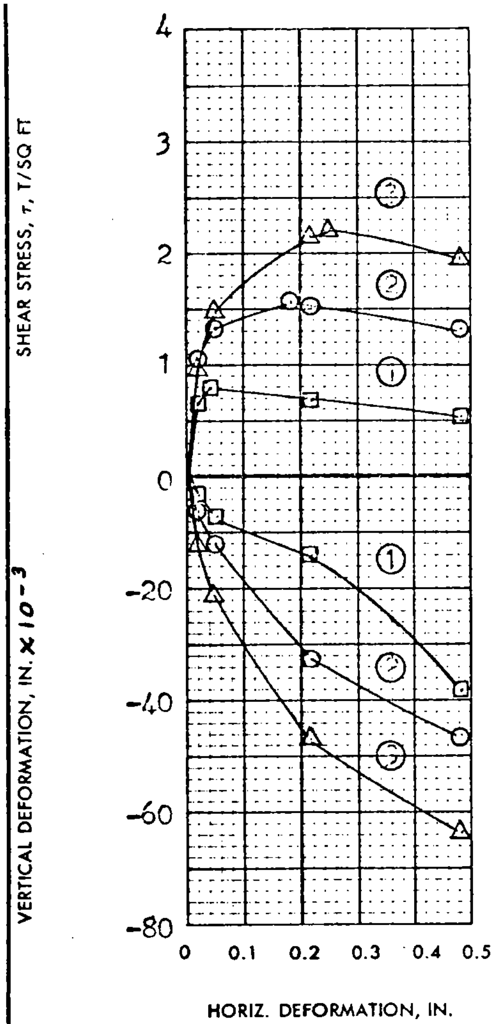
☐ CONTROLLED STRESS

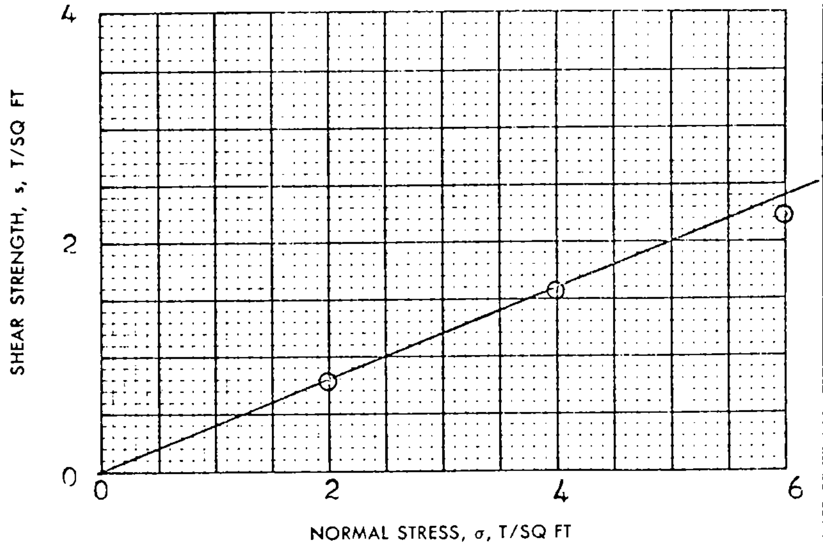
☒ CONTROLLED STRAIN

				1	2	3	
TEST NO.				1	2	3	
INITIAL	WATER CONTENT		w_o	53.4 %	52.6 %	52.7 %	%
INITIAL	VOID RATIO		e_o	1.50	1.49	1.47	
INITIAL	SATURATION		S_o	97.5 %	96.7 %	98.2 %	%
INITIAL	DRY DENSITY, LB/CU FT		γ_d	68.4	68.7	69.2	
VOID RATIO AFTER CONSOLIDATION			e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN			t_{50}	2.8	3.6	4.1	
FINAL	WATER CONTENT		w_f	54.8 %	46.7 %	39.0 %	%
FINAL	VOID RATIO		e_f				
FINAL	SATURATION		S_f	%	%	%	%
NORMAL STRESS, T/SQ FT			σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT			τ_{max}	0.84	1.17	1.63	
ACTUAL TIME TO FAILURE, MIN			t_f	270	1020	8.70	
RATE OF STRAIN, IN./MIN				.00020	.00020	.00020	
ULTIMATE SHEAR STRESS, T/SQ FT			τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED	3.00 IN. SQUARE	1 & 2 0.546 3=0.625	IN. THICK
CLASSIFICATION PLASTIC CLAY(CH),dark gray, contains a trace of sand			
LL 98	PL 26	PI 72	G_s 2.74

REMARKS	PROJECT LK. PONT., LA.& VIC.-HURR. PROT.(1971) RIGOLETS CONTROL STR. & CLOSURE DAM;DDM NO.6
	AREA
	BORING NO. X-13 U — SAMPLE NO. 15-B
	DEPTH EL -81.7 MSL — DATE
	BWG — DIRECT SHEAR TEST REPORT





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	41.4 %	41.5%	41.1%	%
	VOID RATIO	e_o	1.16	1.17	1.15	
	SATURATION	S_o	98.1 %	97.5%	97.4%	%
	DRY DENSITY, LB/CU FT	γ_d	79.4	79.2	79.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	2	2	2	
FINAL	WATER CONTENT	w_f	41.1 %	36.3%	32.2%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.80	1.57	2.21	
ACTUAL TIME TO FAILURE, MIN		t_f	300	1140	1500	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 22°

TAN ϕ' = 0.40

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED	3.00 IN. SQUARE	0.538 IN. THICK	
CLASSIFICATION PLASTIC CLAY(CH),gray			
LL 72	PL 24	PI 48	G_s 2.75

REMARKS

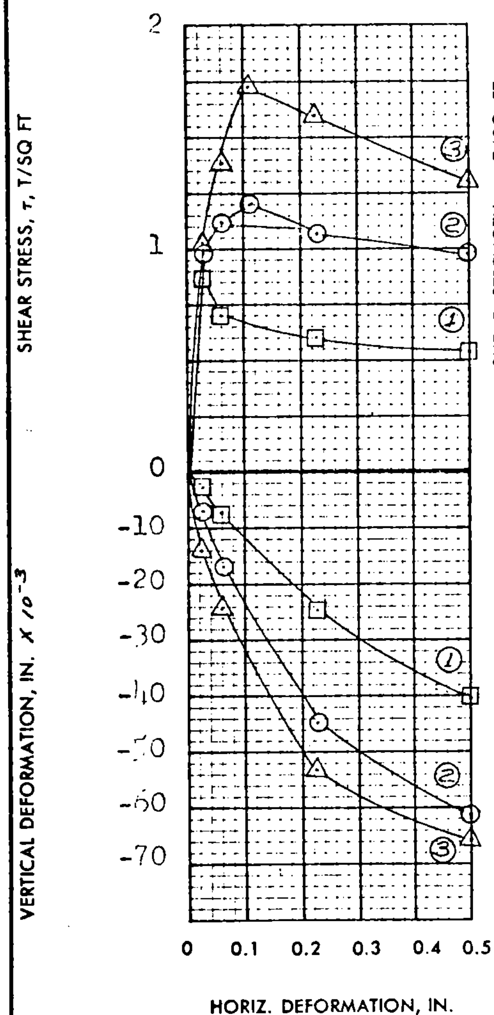
PROJECT LK. PONT.,LA.& VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR. & CLOSURE DAM; DDM # 6

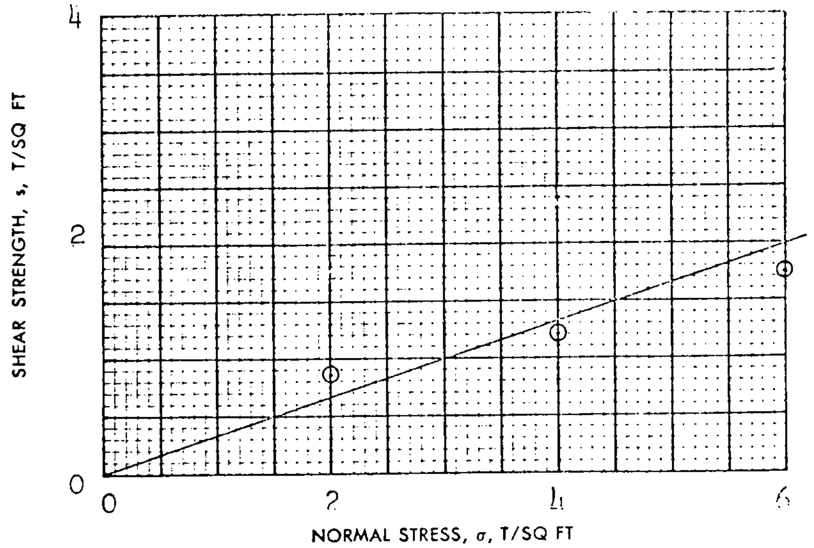
AREA

BORING NO. X-13U	SAMPLE NO. 24-B
~~DEPTH~~ EL -117.9 MSL	DATE 13 January,1972

GDA **DIRECT SHEAR TEST REPORT**

ENG FORM 1 JUN 65 **2092** *(EM 1110-2-1906)* PREVIOUS EDITIONS ARE OBSOLETE *(TRANSLUCENT)* GPO : 1966 OF—2





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	43.3 %	43.9 %	42.2 %	%
	VOID RATIO	e_o	1.22	1.23	1.22	
	SATURATION	S_o	97.6%	98.2%	95.1 %	%
	DRY DENSITY, LB/CU FT	γ_d	77.5	77.1	77.3	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1.4	1.5	1.8	
FINAL	WATER CONTENT	w_f	45.9 %	42.3 %	39.5 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.87	1.20	1.73	
ACTUAL TIME TO FAILURE, MIN		t_f		660	750	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 18°

TAN ϕ' = 0.333

c' = 0 T/SQ FT

[ ] CONTROLLED STRESS

[X] CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	1.2 0.516 3 -0.625 IN. THICK	
CLASSIFICATION	PLASTIC CLAY(CH),gray			
LL 70	PL 24	PI 46		G_s 2.75

REMARKS

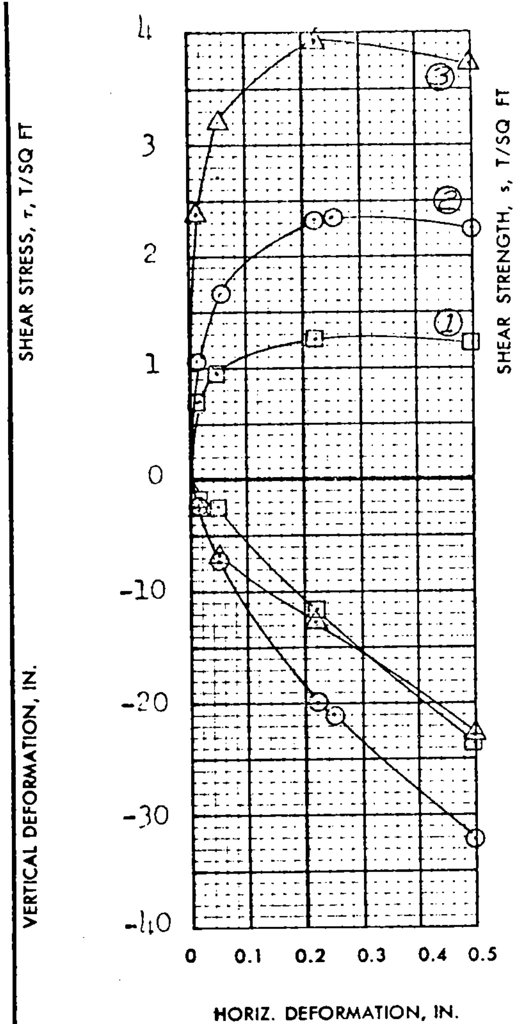
PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS
CONT. STR. & CLOSURE DAM; DDM NO.6

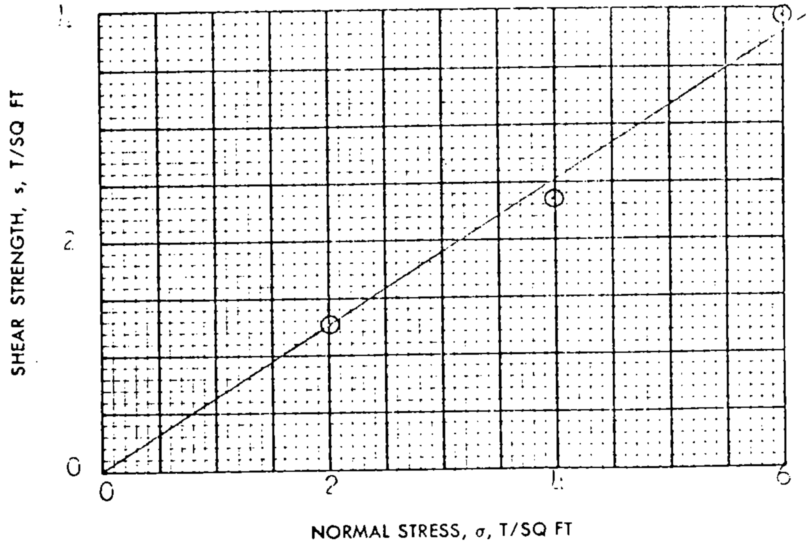
AREA

BORING NO. X-13-U SAMPLE NO. 30-B

DEPTH / EL -141.9 MSL DATE 13 January 1972

BWG **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

ϕ' = 32°

TAN ϕ' = 0.635

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.9 %	24.1 %	24.3 %	%
	VOID RATIO	e_o	0.688	0.697	0.688	
	SATURATION	S_o	93.1 %	92.7 %	94.6 %	%
	DRY DENSITY, LB/CU FT	γ_d	99.1	98.6	99.1	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1	<1	
FINAL	WATER CONTENT	w_f	24.4 %	24.4 %	22.6 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	2.0	4.0	6.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	1.27	2.34	3.95	
ACTUAL TIME TO FAILURE, MIN		t_f	1680	1500	1320	
RATE OF STRAIN, IN./MIN			.00018	.00018		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.538 IN. THICK

CLASSIFICATION CLAYEY SAND(SC),gray, contains pockets of plastic clay

LL – PL – PI — G_s 2.68

REMARKS

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)

RIGOLETS CONTROL STR. & CLOSURE DAM;DDM NO.6

AREA

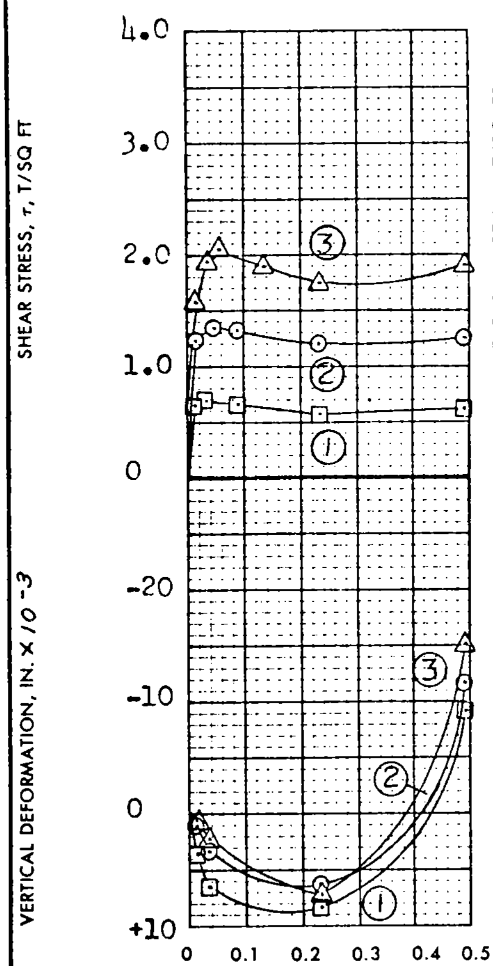
BORING NO. X-13-U — SAMPLE NO. 39-D

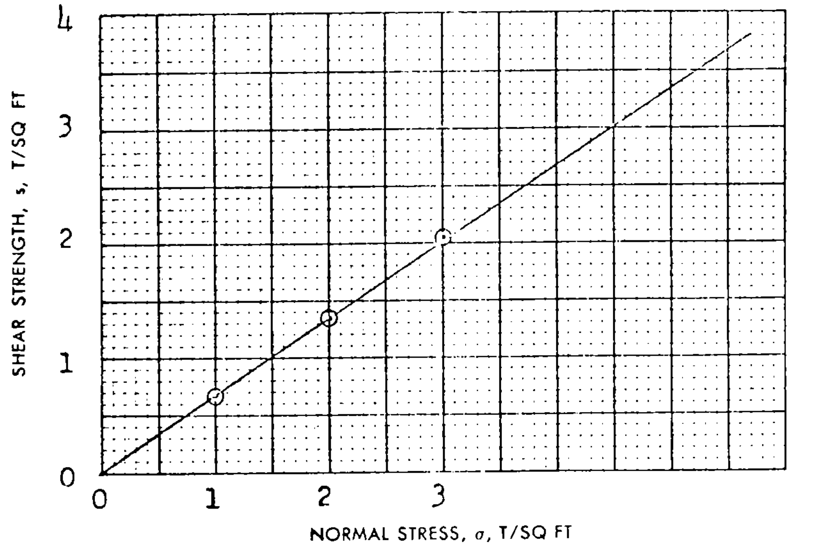
~~DEPTH~~ EL -179.7 MSL — DATE 18 January 1972

GDA

DIRECT SHEAR TEST REPORT

ENG FORM 2092 1 JUN 65 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—





SHEAR STRENGTH PARAMETERS

ϕ' = 34°

TAN ϕ' = 0.675

c' = 0 T/SQ FT

[ ] CONTROLLED STRESS

[X] CONTROLLED STRAIN

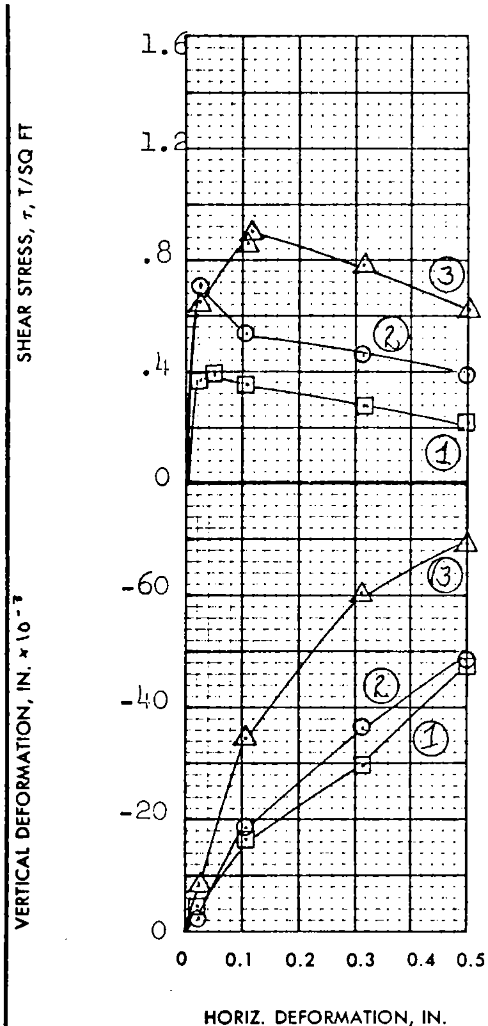
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	21.3 %	21.6 %	22.2%	%
	VOID RATIO	e_o	0.655	0.672	0.679	
	SATURATION	S_o	86.8%	85.8%	87.3 %	%
	DRY DENSITY, LB/CU FT	γ_d	100.7	99.7	99.3	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	23.2 %	24.0 %	24.1%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.69	1.35	2.05	
ACTUAL TIME TO FAILURE, MIN		t_f	360	420	480	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

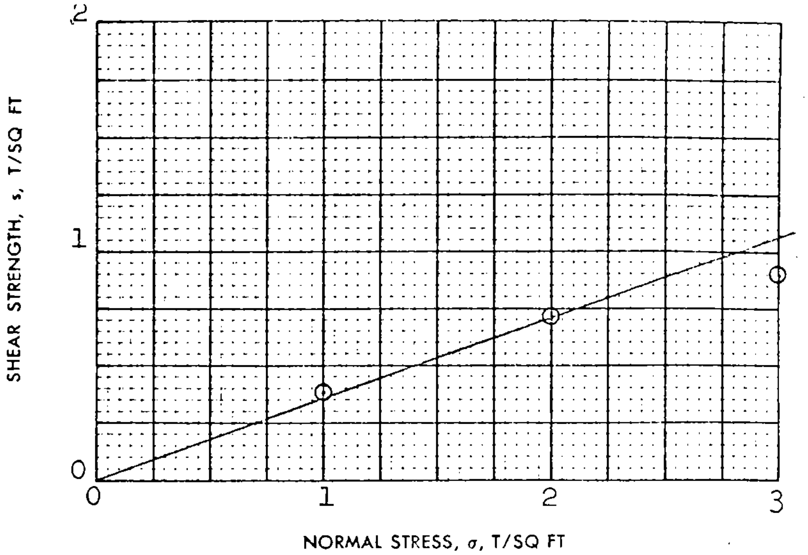
TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.550 IN. THICK
CLASSIFICATION SAND(SP)F, grayish white, contains pockets of plastic clay			
LL –	PL –	PI –	G_s 2.67

REMARKS	PROJECT LK.PONT.LA.,& VIC.-HURR.PROT.- RIGOLETS CONTROL STRUCT. AND CLOSURE DAM
	AREA
	BORING NO. X 14-U — SAMPLE NO. 6-B
	~~DEPTH~~ EL - 22.0 MSL — DATE 14 July 1971
	JHMc — DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

Fig. A - 33





SHEAR STRENGTH PARAMETERS

φ' = 20°

TAN φ' = 0.355

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	52.1 %	51.6 %	54.3 %	%
	VOID RATIO	e_o	1.50	1.50	1.52	
	SATURATION	S_o	96.2 %	95.3 %	99.0 %	%
	DRY DENSITY, LB/CU FT	γ_d	69.1	69.3	68.6	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	2	1	2	
FINAL	WATER CONTENT	w_f	55.8 %	50.1 %	48.6 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.39	0.71	0.90	
ACTUAL TIME TO FAILURE, MIN		t_f	540	300	1560	
RATE OF STRAIN, IN./MIN			.00012	.00012	.00012	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 1&2 0.0550 / 3 0.0625 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, contains trace of finely divided organic*

LL 94 | PL 26 | PI 68 | G_s 2.77

REMARKS *matter, fissured

PROJECT LK. PONT.LA., & VIC.-HURR. PROT.- RIGOLETS CONTROL STURCT. & CLOSURE DAM;

AREA DDM NO 6 (1971)

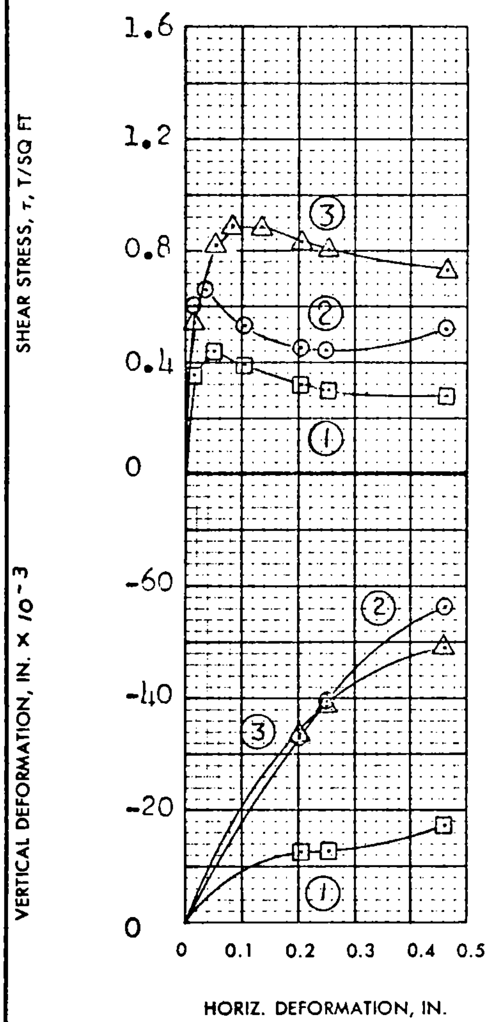
BORING NO. X14-U SAMPLE NO. 16-C

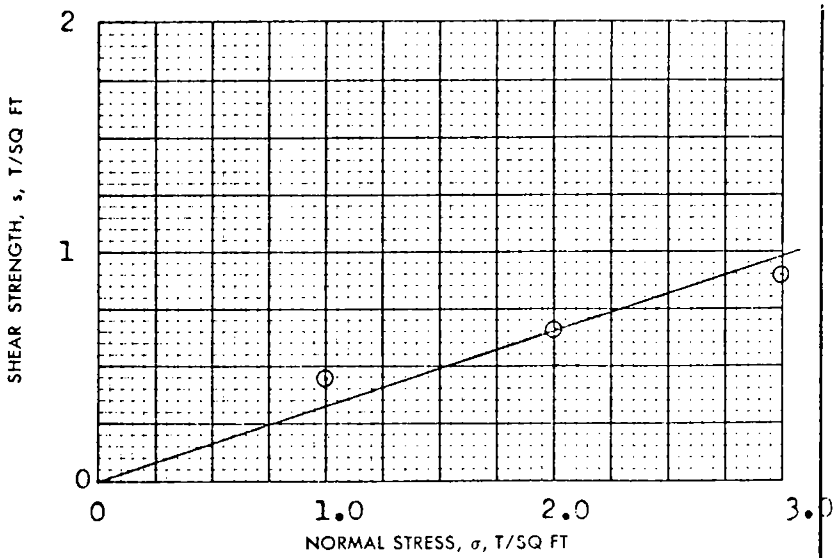
~~DEPTH~~ EL -63.1 MSL DATE 1 September 1971

BWG

DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) Fig. A - 34





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	54.1 %	54.9 %	51.6 %	%
	VOID RATIO	e_o	1.45	1.64	1.57	
	SATURATION	S_o	100+ %	92.7 %	91.0 %	%
	DRY DENSITY, LB/CU FT	γ_d	70.6	65.6	67.3	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1	3	
FINAL	WATER CONTENT	w_f	62.8 %	58.5 %	54.1 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.44	0.66	0.89	
ACTUAL TIME TO FAILURE, MIN		t_f	330	240	540	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 18°

TAN ϕ' = 0.33

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.550 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray, slickensided

LL 106 | PL 28 | PI 78 | G_s 2.77

REMARKS

PROJECT LK.PONT.LA.,& VIC.-HURR. PRCT.-RIGOLETS CONTROL STRUCT. & CLOSURE DAM;

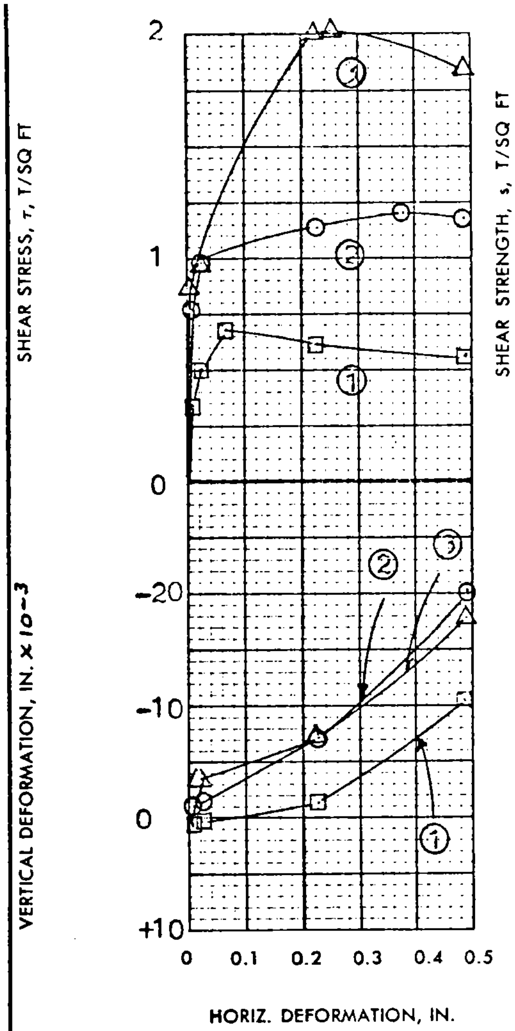
AREA DDM NO. 6 (1971)

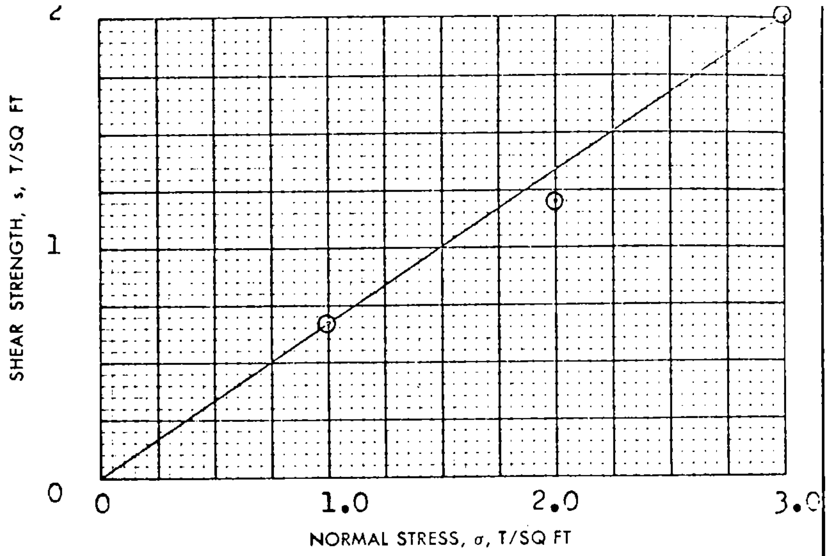
BORING NO. X-14-U — SAMPLE NO. 20 - D

DEPTH EL - 80.1 MSL — DATE 21 July,1971

JHMc

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = 34°

TAN ϕ' = 0.67

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.0%	22.4%	23.0%	%
INITIAL	VOID RATIO	e_o	0.655	0.653	0.658	
INITIAL	SATURATION	S_o	94.1%	91.9%	93.7%	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	101.1	101.2	100.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	15	7	<1	
FINAL	WATER CONTENT	w_f	22.6%	24.2%	22.7%	%
FINAL	VOID RATIO	e_f				
FINAL	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.68	1.20	2.01	
ACTUAL TIME TO FAILURE, MIN		t_f	720	2220	1500	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED — 3.00 IN. SQUARE — 0.550 IN. THICK

CLASSIFICATION SILTY SAND(SM),light gray, trace of organic matter and pockets of*

LL - PL - PI - G_s 2.68

REMARKS * plastic clay

PROJECT LK. PONT.,LA.& VIC.-HURR. PROT. RJGOLETS CONTROL STR. & CLOSURE DAM,DDM #6 (1971)

AREA

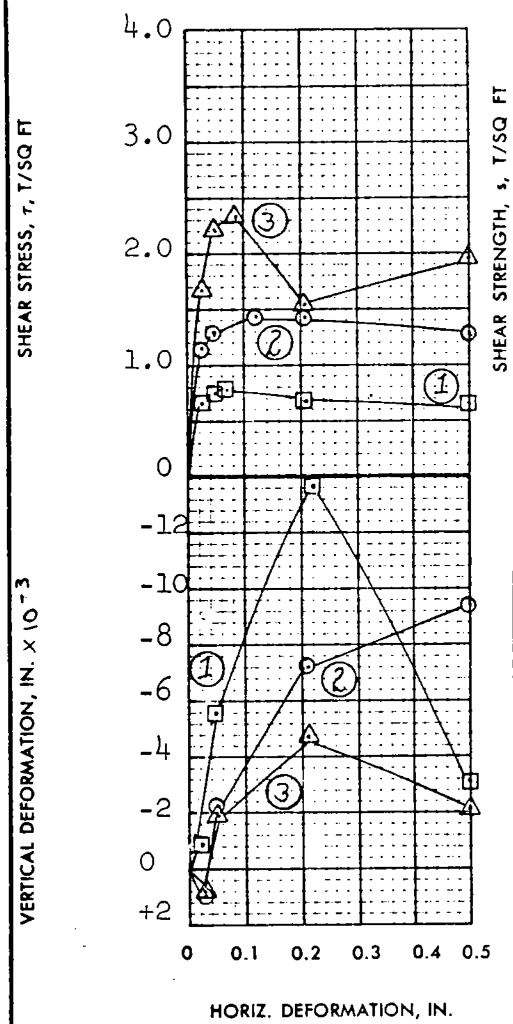
BORING NO. X-14-U SAMPLE NO. 24-C

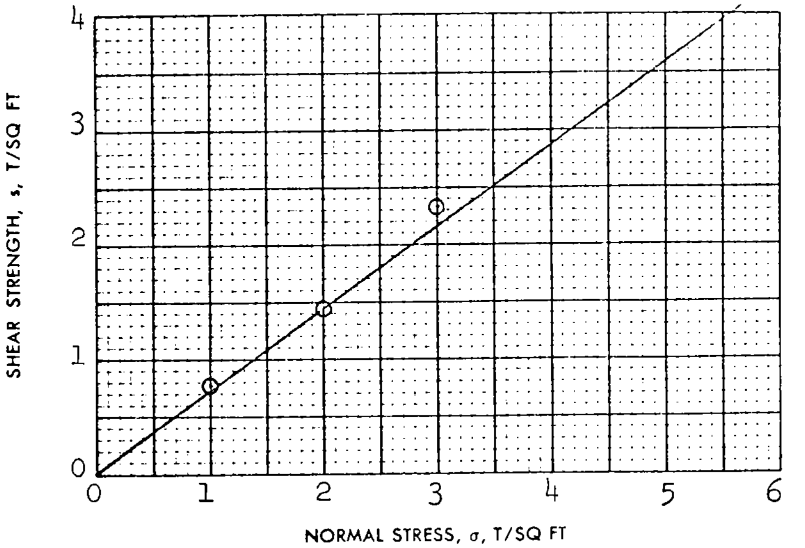
~~DEPTH~~ EL -95.0 MSL DATE 26 July,1971

JHMC **DIRECT SHEAR TEST REPORT**

Fig. A - 36

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)





SHEAR STRENGTH PARAMETERS

ϕ' = 36°

TAN ϕ' = 0.72

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	22.9 %	22.5%	23.1%	%
	VOID RATIO	e_o	0.671	0.633	0.615	
	SATURATION	S_o	90.8 %	94.5 %	99.9%	%
	DRY DENSITY, LB/CU FT	γ_d	99.4	101.7	102.8	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	24.7 %	25.0%	25.1%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.79	1.44	2.32	
ACTUAL TIME TO FAILURE, MIN		t_f	510	780	570	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED 3.00 IN. SQUARE 0.550 IN. THICK

CLASSIFICATION SILTY SAND(SM),gray

LL - PL - PI - G_s 2.66

REMARKS

PROJECT LK. PONT;LA.,&VIC. -HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,

AREA DDM NO. 6

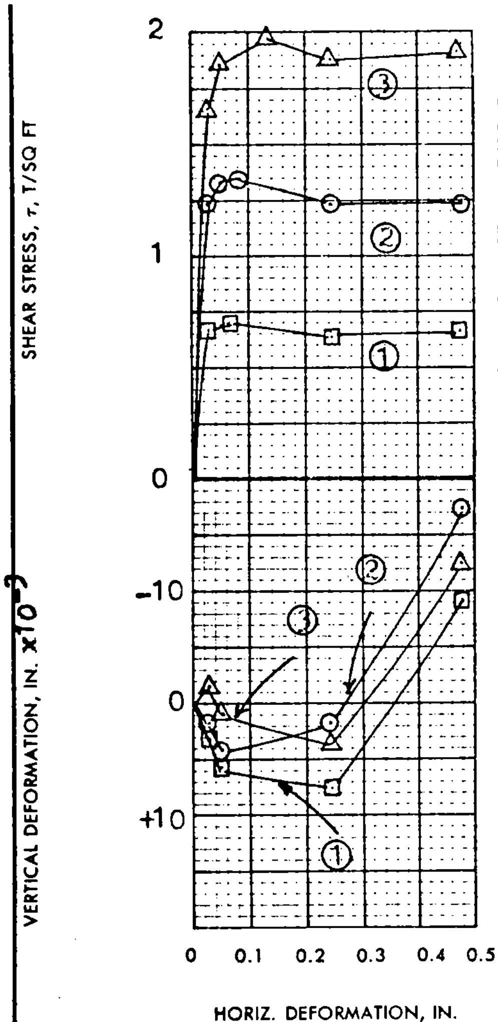
BORING NO. X-15-U SAMPLE NO. 7-B

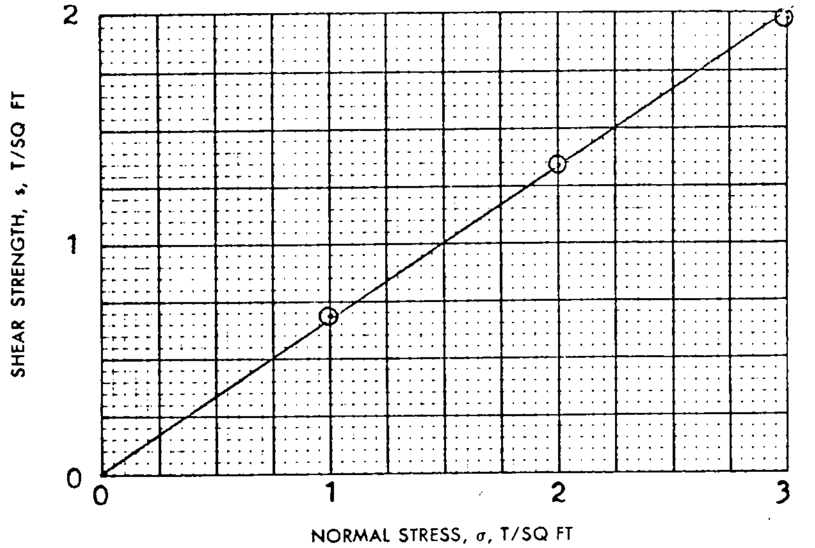
~~DEPTH~~ EL -22.1 MSL DATE 7 September 1971

BWG

DIRECT SHEAR TEST REPORT

ENG FORM 2092 1 JUN 65 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)





SHEAR STRENGTH PARAMETERS

ϕ' = 34°

TAN ϕ' = 0.67

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

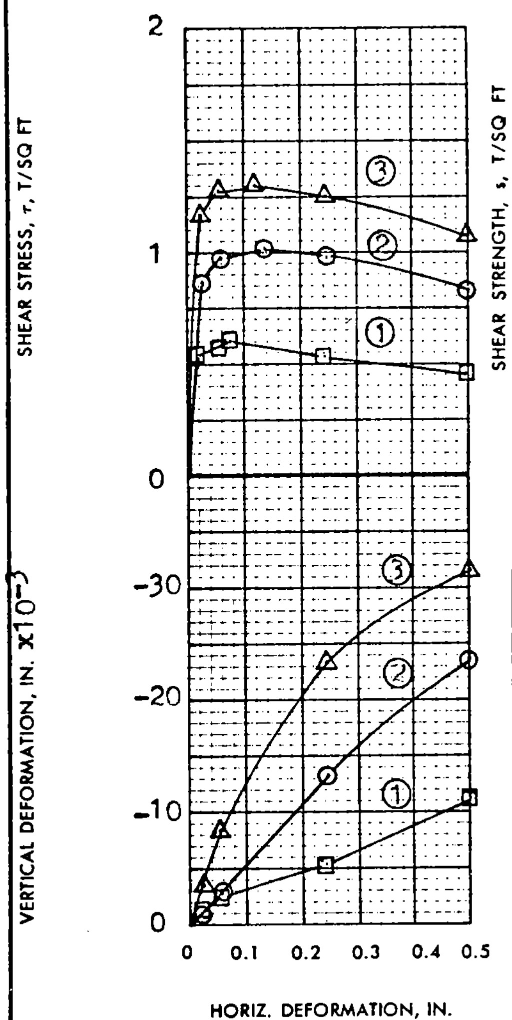
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	23.3 %	23.9 %	22.9 %	%
	VOID RATIO	e_o	0.693	0.702	0.658	
	SATURATION	S_o	90.1 %	91.2 %	93.3 %	%
	DRY DENSITY, LB/CU FT	γ_d	98.8	98.3	100.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	22.5 %	21.9 %	22.6 %	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.69	1.34	1.98	
ACTUAL TIME TO FAILURE, MIN		t_f	420	540	780	
RATE OF STRAIN, IN./MIN			.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

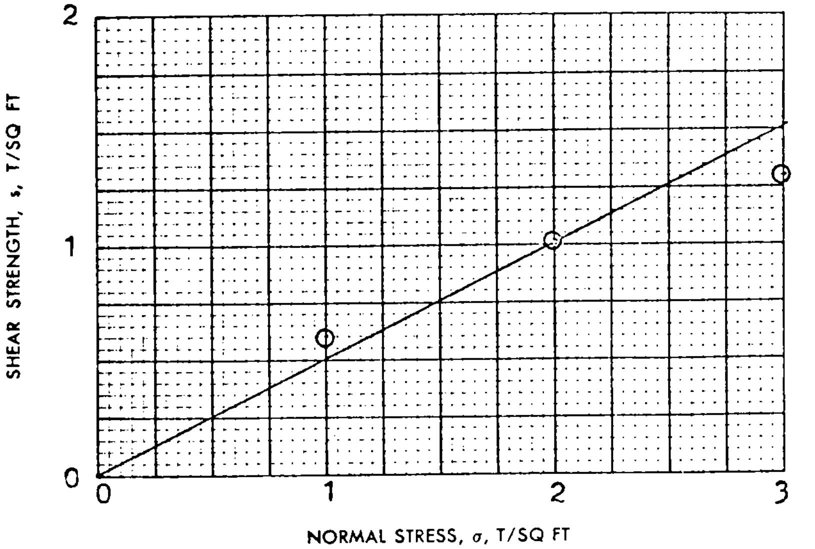
TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION SILTY SAND(SM), light gray			
LL -	PL -	PI -	G_s 2.68

REMARKS	PROJECT LK. PONT. LA. & VIC.-HURR. PROT.-RIGOLETS CONTROL STRUCTURE & CLOSURE DAM; DDM #6
	AREA
	BORING NO. X-15U — SAMPLE NO. 16-D
	~~DEPTH~~ EL -59.8 MSL — DATE 8 September, 1971
	BWG — **DIRECT SHEAR TEST REPORT**

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

Fig. A - 38





SHEAR STRENGTH PARAMETERS

ϕ' = 27°

TAN ϕ' = 0.505

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	27.0%	27.9%	27.9%	%
	VOID RATIO	e_o	0.764	0.797	0.793	
	SATURATION	S_o	97.5%	96.6%	97.1%	%
	DRY DENSITY, LB/CU FT	γ_d	97.7	95.9	96.1	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	< 1	
FINAL	WATER CONTENT	w_f	30.3%	28.6%	27.7%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.60	1.01	1.30	
ACTUAL TIME TO FAILURE, MIN		t_f	450	780	690	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION PLASTIC CLAY(CH),gray, fissured			
LL -	PL -	PI -	G_s 2.76 From Q

REMARKS

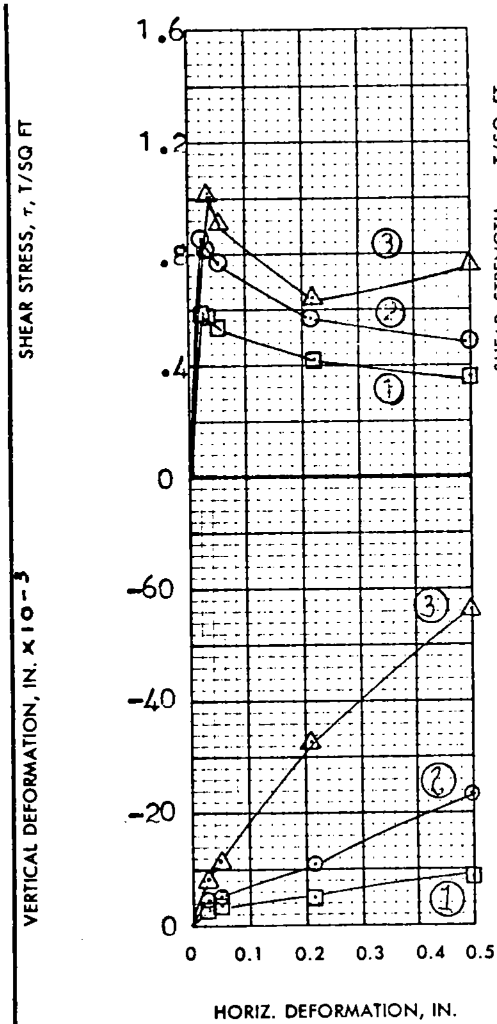
PROJECT LK.PONT.LA,& VIC.-HURR, PROT.-RIGOLETS CONTROL STRUCTURE & CLOSURE DAM; DDM #6

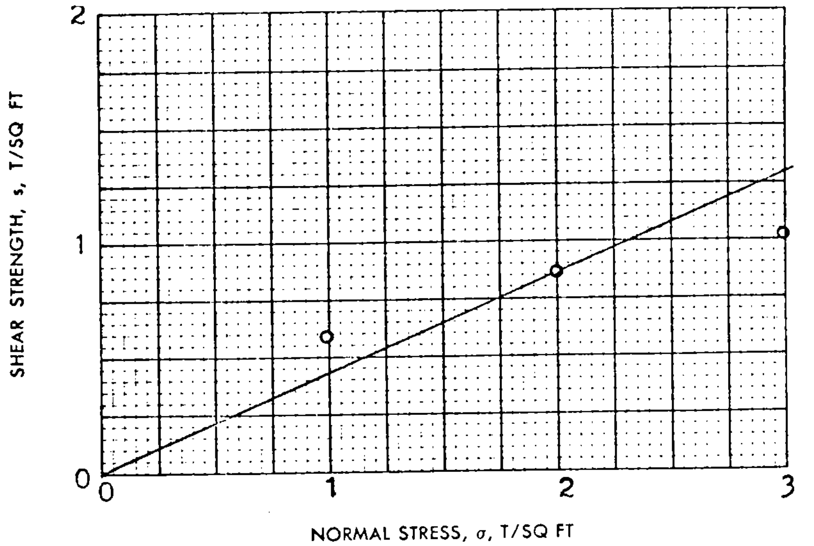
AREA

BORING NO. X-15U — SAMPLE NO. 19-D

~~DEPTH~~ EL -71.9 MSL — DATE 15 September,1971

BWG — **DIRECT SHEAR TEST REPORT**





SHEAR STRENGTH PARAMETERS

ϕ' = 23°

TAN ϕ' = 0.43

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	48.1 %	47.1 %	46.8%	%
	VOID RATIO	e_o	1.32	1.32	1.31	
	SATURATION	S_o	99.8 %	97.8 %	97.9%	%
	DRY DENSITY, LB/CU FT	γ_d	73.8	73.6	74.1	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	2	2	
FINAL	WATER CONTENT	w_f	50.4 %	52.5 %	48.4%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.59	0.86	1.02	
ACTUAL TIME TO FAILURE, MIN		t_f	180	180	270	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

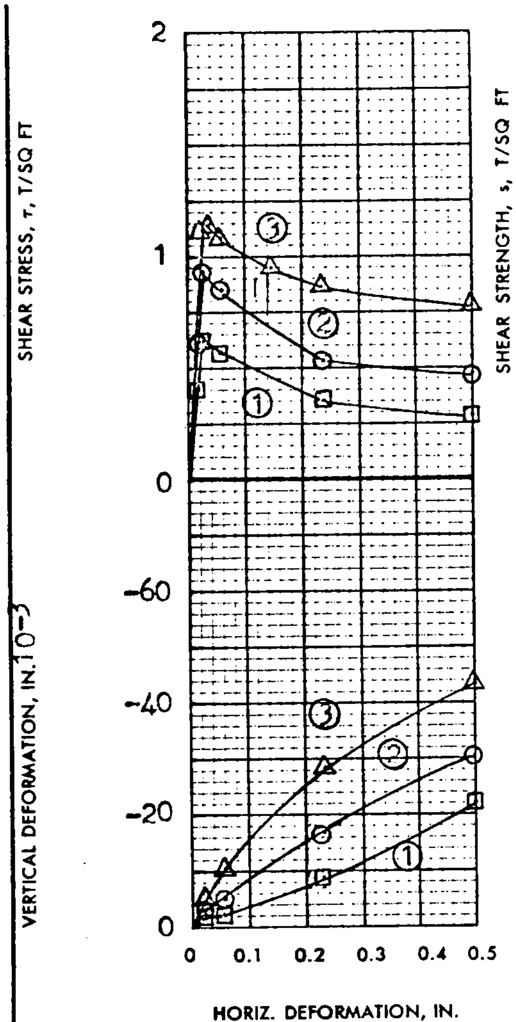
TYPE OF SPECIMEN	UNDISTURBED	3.00 IN. SQUARE	1=0.544 2&3 0.539 IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH),gray, fissured		
LL –	PL –	PI –	G_s 2.74 From Q

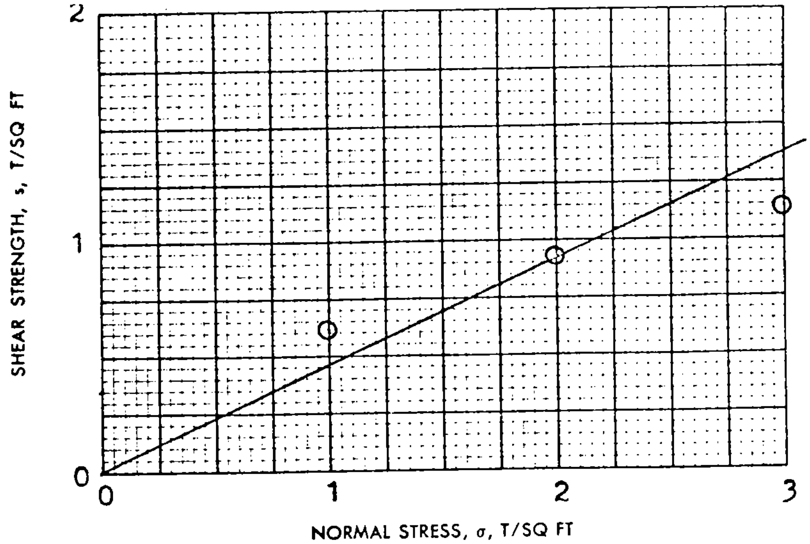
REMARKS

PROJECT	LK.PONT.LA.& VIC.-HURR. PROT.-RIGOLETS CONTROL STRUCTURE & CLOSURE DAM DDM#6		
AREA			
BORING NO.	X-15U	SAMPLE NO.	22-D
~~DEPTH~~ EL	-84.0 MSL	DATE	21 September,1971
BWG	DIRECT SHEAR TEST REPORT		

Fig. A - 40

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	43.6 %	43.2%	42.9%	%
	VOID RATIO	e_o	1.18	1.12	1.19	
	SATURATION	S_o	100+ %	100+%	98.4%	%
	DRY DENSITY, LB/CU FT	γ_d	78.1	80.4	77.9	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1	2	
FINAL	WATER CONTENT	w_f	47.7 %	40.1%	44.1%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.62	0.93	1.14	
ACTUAL TIME TO FAILURE, MIN		t_f	180	180	240	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 25°

TAN ϕ' = 0.465

c' = 0 T/SQ FT

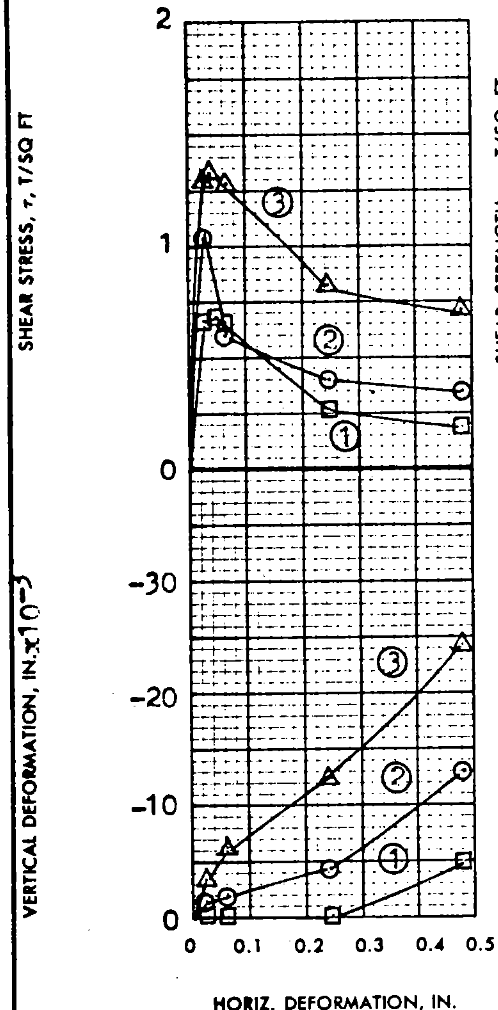
☐ CONTROLLED STRESS

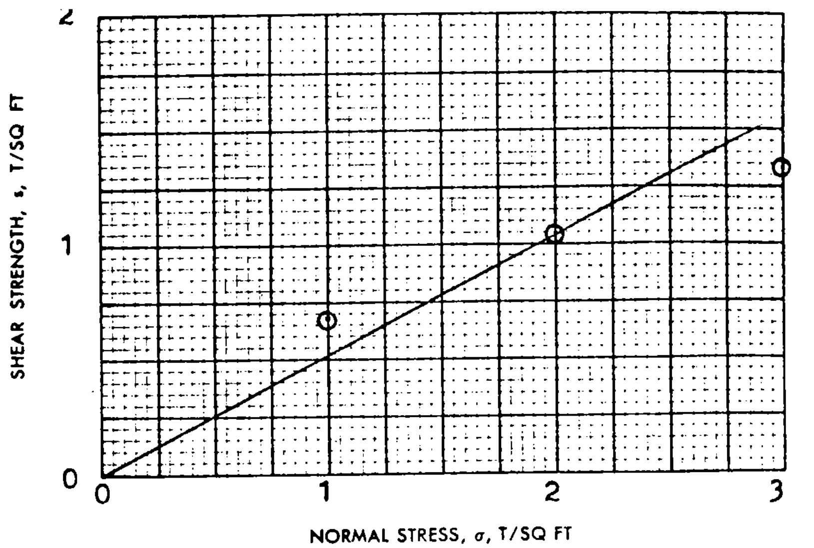
☐ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED		3.00 IN. SQUARE		0.544 IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH),gray, contains silt pockets, fissured				
LL -	PL -	PI -			G_s 2.73 From Q

REMARKS

PROJECT LK.PONT.LA.& VIC.-HURR. PROT.-RIGOLETS CONTROL STRUCTURE & CLOSURE DAM DDM#6		
AREA		
BORING NO. X-15U		SAMPLE NO. 34-B
~~DEPTH~~ EL -130.1 MSL		DATE 22 September,1971
BWG	**DIRECT SHEAR TEST REPORT**	





TEST NO.		1	2	3	
INITIAL — WATER CONTENT	w_o	58.3 %	64.3%	59.3%	%
INITIAL — VOID RATIO	e_o	1.56	1.67	1.60	
INITIAL — SATURATION	S_o	97.9 %	100%	97.1%	%
INITIAL — DRY DENSITY, LB/CU FT	γ_d	63.9	61.3	62.9	
VOID RATIO AFTER CONSOLIDATION	e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN	t_{50}	<1	2	2	
FINAL — WATER CONTENT	w_f	67.0 %	65.4%	62.8%	%
FINAL — VOID RATIO	e_f				
FINAL — SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT	σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT	τ_{max}	0.68	1.04	1.33	
ACTUAL TIME TO FAILURE, MIN	t_f	300	180	240	
RATE OF STRAIN, IN./MIN		.00017	.00017	.00017	
ULTIMATE SHEAR STRESS, T/SQ FT	τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 27°

TAN ϕ' = 0.52

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☐ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED		3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION PLASTIC CLAY(CH), dark gray, contains finely divided organic matter*			
LL –	PL –	PI –	G_s 2.62 From

REMARKS *fissured

PROJECT LK.PONT.LA.& VIC.-HURR. PROT.-RIGOLETS

CONTROL STRUCTURE & CLOSURE DAM; DDM#6

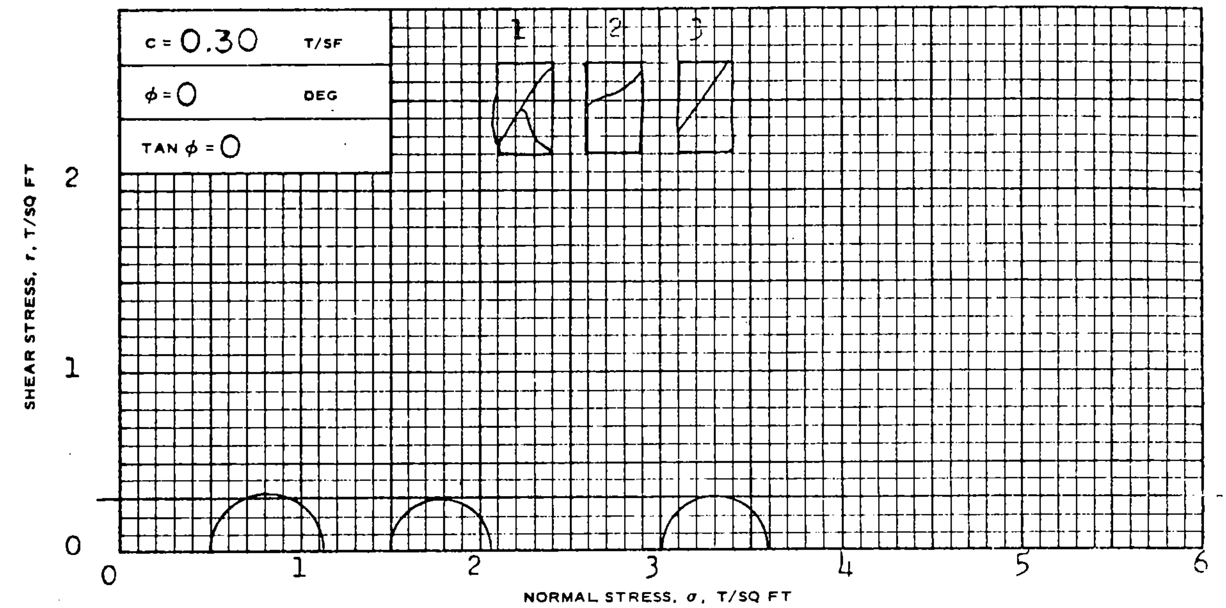
AREA

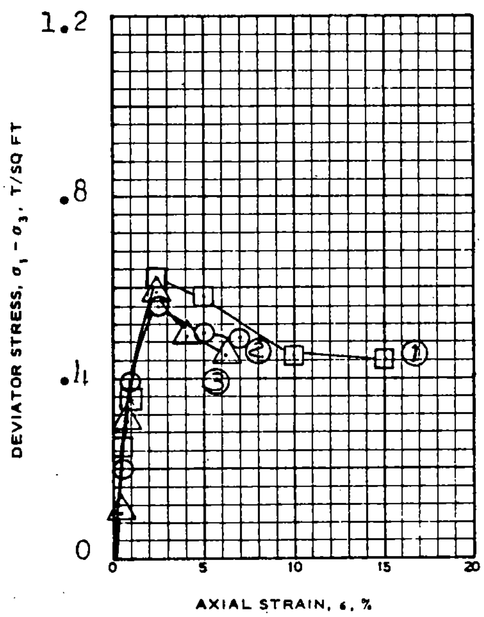
BORING NO. X-15U SAMPLE NO. 45-B

~~DEPTH~~ EL -174.2 MSL DATE 23 September, 1971

BWG **DIRECT SHEAR TEST REPORT**

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)





			1	2	3	
SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	73.3	75.9	78.4	
	DRY DENSITY LB/CU FT	γ_{d_o}	55.2	54.3	53.2	
	SATURATION, %	S_o	97.1	98.1	98.5	
	VOID RATIO	e_o	2.00	2.05	2.11	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.62	0.56	0.59	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	12	19	20	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.41	1.41	1.41	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains sand lenses

LL	112	PL	28	PI	84	Gs	2.65	TYPE OF SPECIMEN	UNDISTURBED	TYPE OF TEST	Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

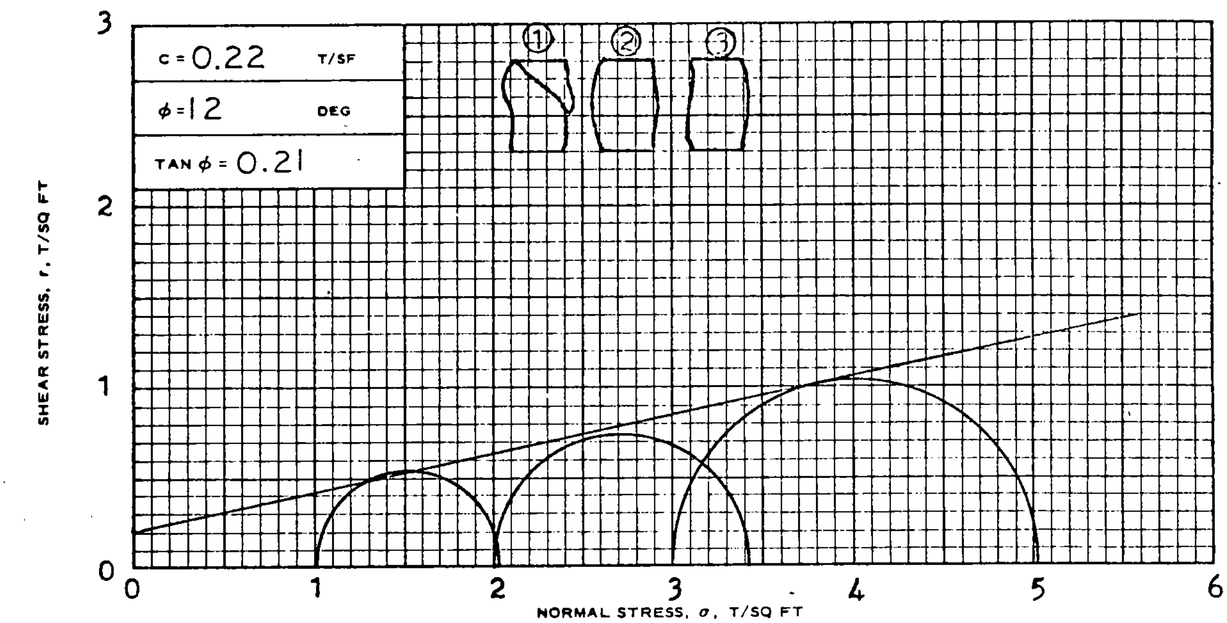
BORING NO. X-1-U SAMPLE NO. 6-B

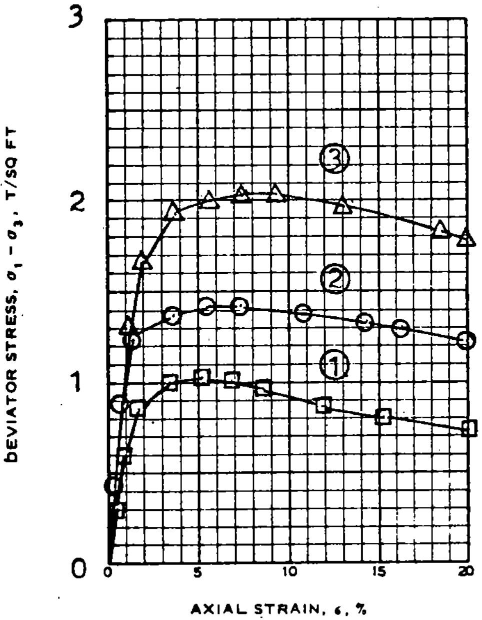
~~DEPTH~~/ELEV -41.7 msl

LABORATORY USAEWES DATE 20 October, 1971

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	84.8	84.3	80.1
	DRY DENSITY LB/CU FT	γd_o	50.8	51.3	52.7
	SATURATION, %	S_o	99.0	99.9	98.7
	VOID RATIO	e_o	2.30	2.26	2.18
BEFORE SHEAR	WATER CONTENT, %	w_c	76.7	63.8	54.0
	DRY DENSITY LB/CU FT	γd_c	54.6	61.3	66.9
	SATURATION, %	S_c	99.6	98.9	96.5*
	VOID RATIO	e_c	2.06	1.73	1.50
	FINAL BACK PRESSURE, T/SQ FT	u_o	3.13	3.13	3.13
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.02	1.42	2.02
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	62	66	90
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), grayish brown, contains a few small silt lenses

LL 128 | PL 29 | PI 99 | Gs 2.68 | TYPE OF SPECIMEN UNDIST. | TYPE OF TEST R

REMARKS: * Pore pressure response indicated 100% saturation

PROJECT LK. PONT., LA.& VICINITY-HURR. PROT. ('71) RIGOLETS CONTROL STR.&CLOSURE DDM #6

BORING NO. X-1U SAMPLE NO. 7-B

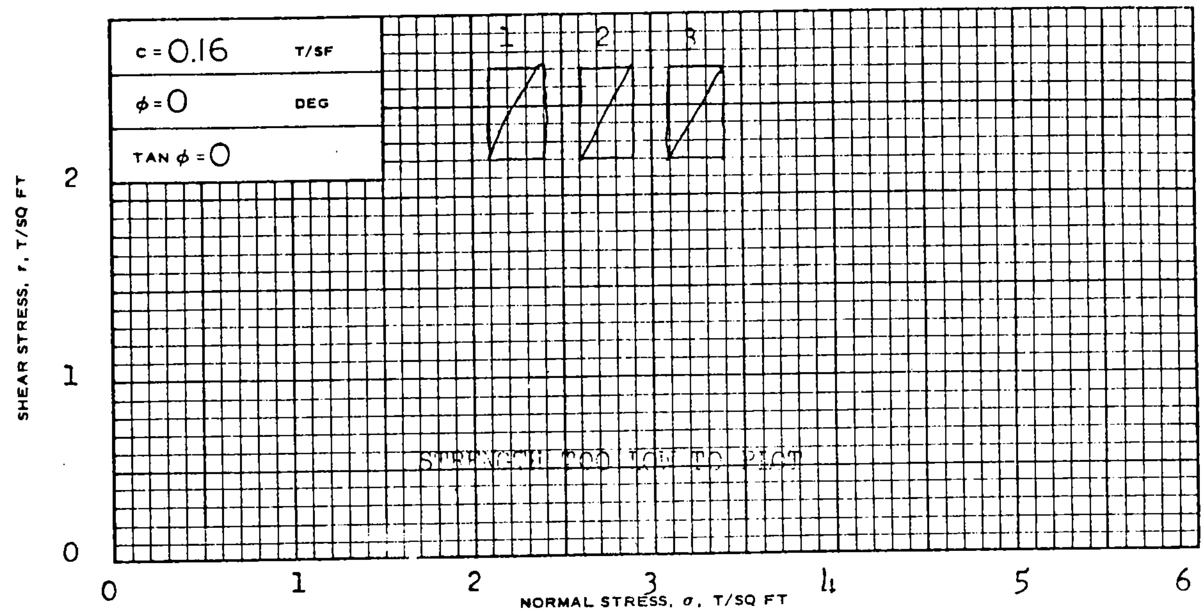
~~DEPTH~~/ELEV -46.3 MSL

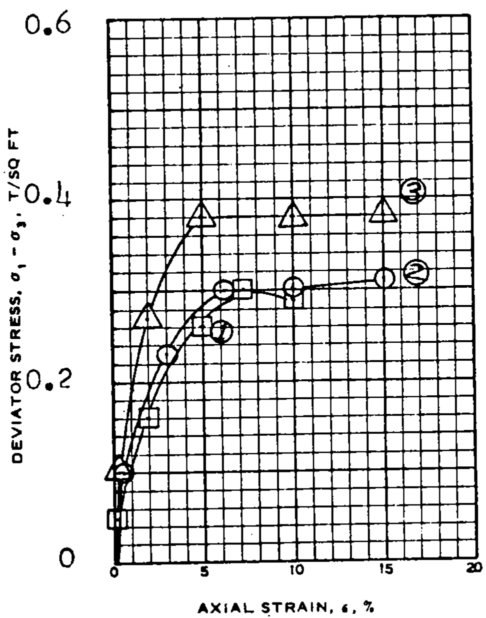
LABORATORY USAEWES DATE 18 October, 1971

JAL TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

Fig. A - 44





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	82.1	84.0	84.1
	DRY DENSITY LB/ CU FT	γd_o	51.3	51.0	51.5
	SATURATION, %	S_o	97.4	98.7	100+
	VOID RATIO	e_o	2.26	2.28	2.25
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.30	0.31	0.38
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	133	70	75
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),dark gray, contains seams of silty sand

LL 126	PL 30	PI 90	Gs 2.68	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS: Shear plane visible at approx. 7.2% axial strain on test no.1 and 6.2% axial strain on test no. 2 .

PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-1-U SAMPLE NO. 8-R

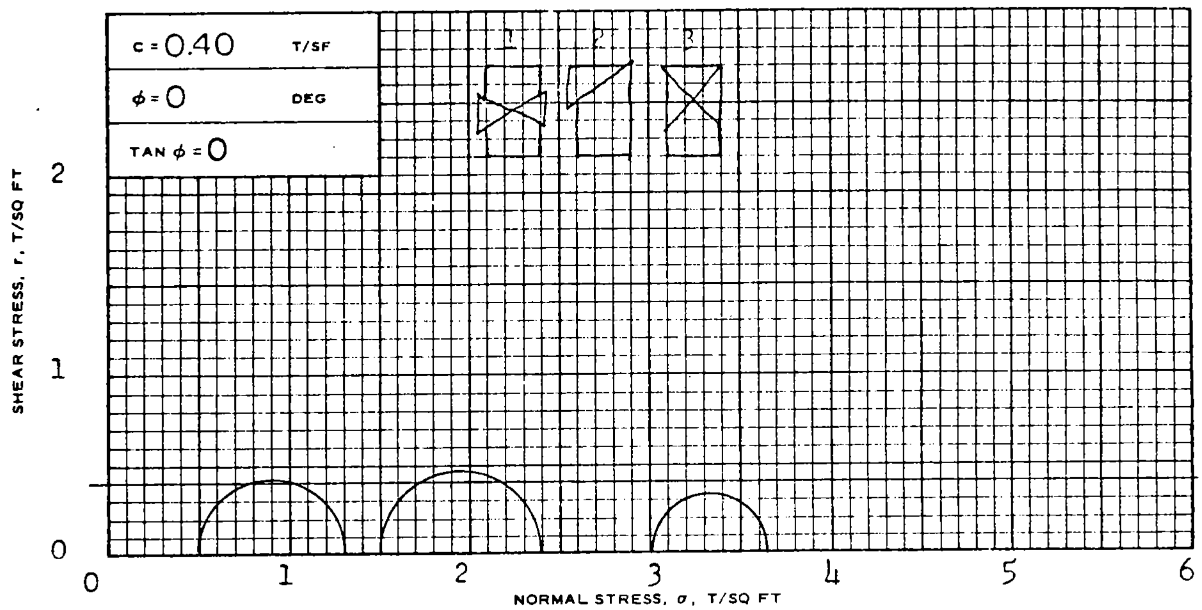
DEPTH/ELEV -50.1 MSL

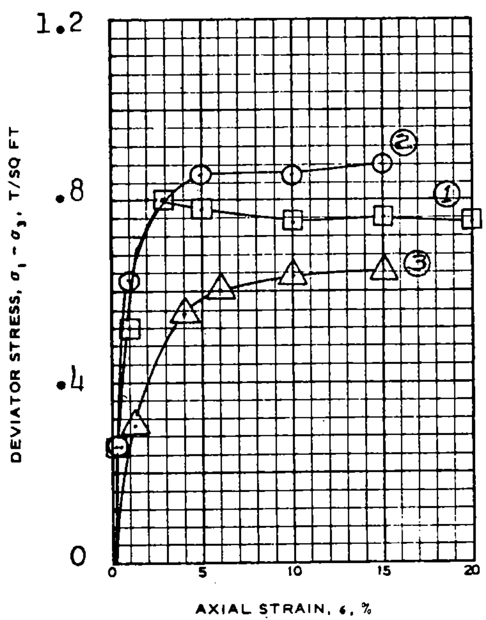
LABORATORY USAEWES DATE 22 October, 1971

GDA **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*
REV JUNE 1970

Fig. A - 45





			1	2	3
SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	38.4	37.6	38.8
	DRY DENSITY LB/CU FT	γ_{d_o}	82.9	83.5	81.4
	SATURATION, %	S_o	100+	100+	98.5
	VOID RATIO	e_o	1.02	1.01	1.06
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γ_{d_c}			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.80	0.88	0.64
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	16	110	103
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, with sand and organic matter

LL 55 | PL 17 | PI 38 | Gs 2.69 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q

REMARKS: Shear plane visible at approx. 2.2% axial strain on test no.3.

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

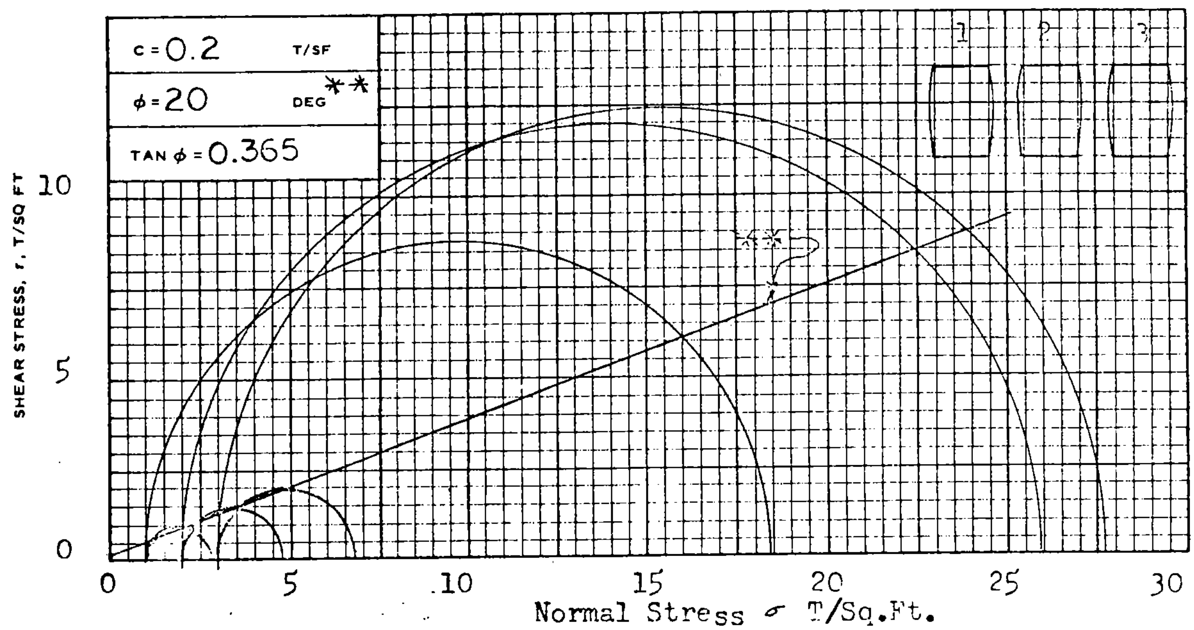
BORING NO. X-1-U | SAMPLE NO. 10-D

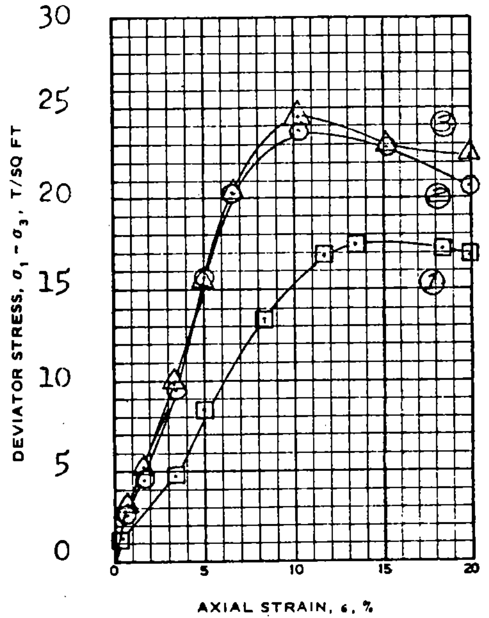
DEPTH/ELEV -61.0 MSL

LABORATORY USAEWES | DATE 21 October 1971

GDA **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	26.3	25.9	25.7	
	DRY DENSITY LB/CU FT	γd_o	97.4	97.8	97.8	
	SATURATION, %	S_o	98.8	98.1	97.4	
	VOID RATIO	e_o	0.711	0.705	0.701	
BEFORE SHEAR	WATER CONTENT, %	w_c	25.4	25.0	24.4	
	DRY DENSITY LB/CU FT	γd_c	99.0	100.1	100	
	SATURATION, %	S_c	99.1	100+	97.78	*
	VOID RATIO	e_c	0.684	0.666	0.666	
	FINAL BACK PRESSURE, T/SQ FT	u_o	4.68	4.68	4.68	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	17.41	23.84	24.62	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	150	113	113	
$(\sigma_1-\sigma_3)$ at max pore pressure **			1.7	2.8	3.7	
INITIAL DIAMETER, IN.		D_o	1.39	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS SILTY SAND(SM),tan

LL –	PL –	PI –	G_s 2.67	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS: Pore pressure response indicated 100% saturation.

PROJECT LK. PONT.,LA. & VICINITY-HURR.PROT. (1971)RIGOLETS CONT.STR.&CLOSURE DAM:DDM 6

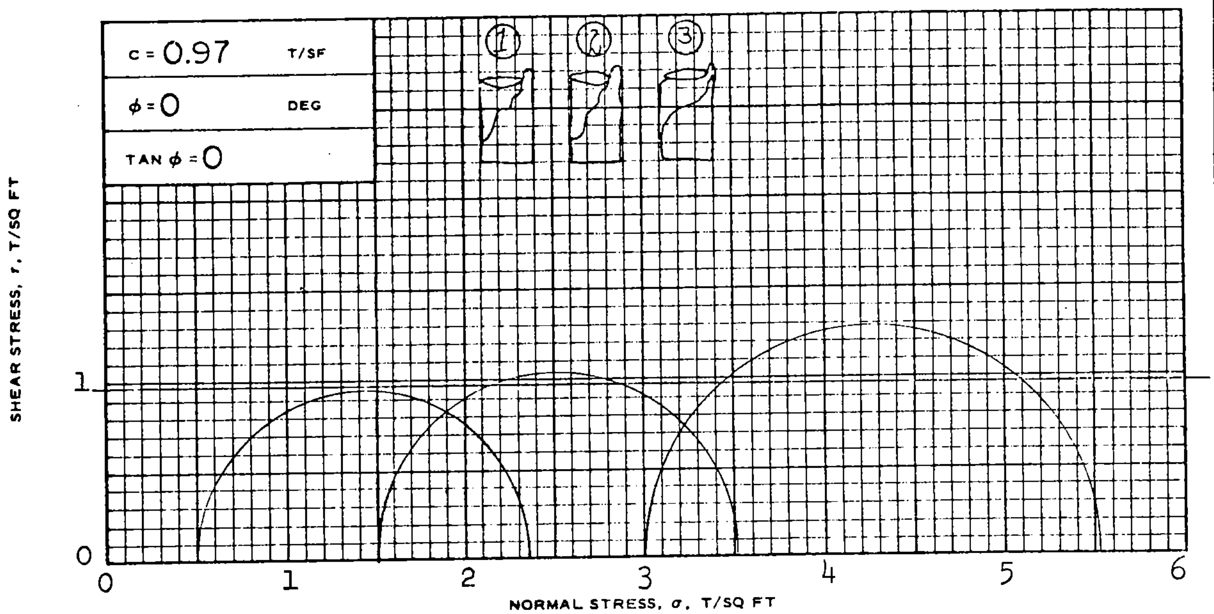
BORING NO. X-1U SAMPLE NO. 13-C

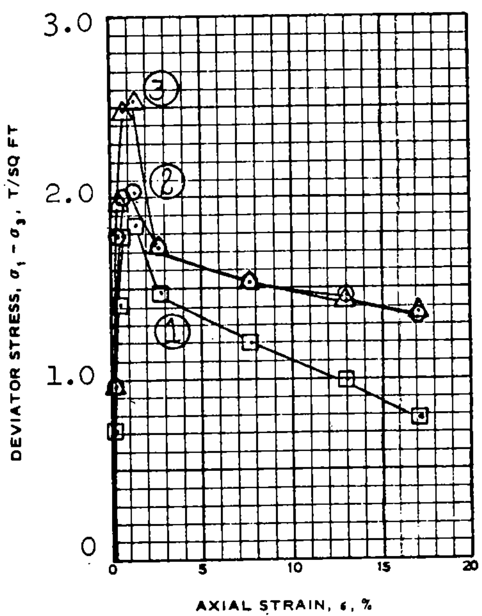
DEPTH/ELEV -71.0 MSL

LABORATORY USAEWES DATE 18 October 1971

JAL TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	55.0	54.5	54.3	
	DRY DENSITY LB/CU FT	γ_{d_o}	68.2	68.5	68.9	
	SATURATION, %	S_o	100+	99.8	100+	
	VOID RATIO	e_o	1.50	1.49	1.47	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.85	2.02	2.53	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	15	15	15	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), mottled gray, light gray and brown, fissured

LL 104	PL 29	PI 75	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. & VIC.-HURR. PROT. (1971)
RIGOLETS CONTROL STR. & CLOSURE DAM; DDM NO. 6

BORING NO. X-1-U SAMPLE NO. 16-B

~~DEPTH~~/ELEV -82.1 MSL

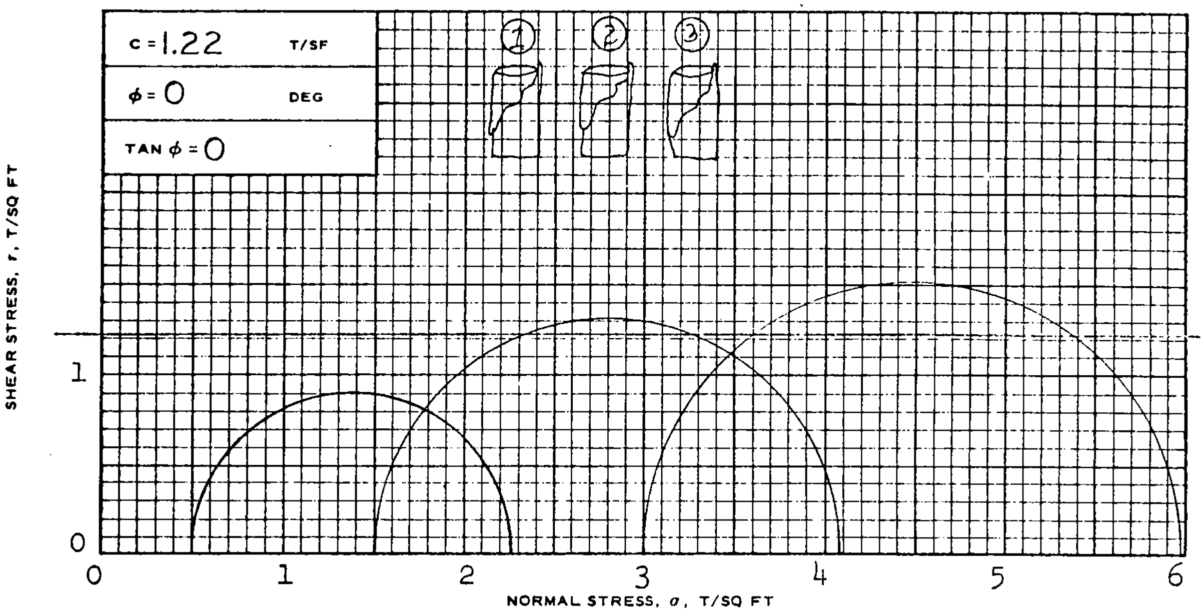
LABORATORY USAEWES DATE 13 October 1971

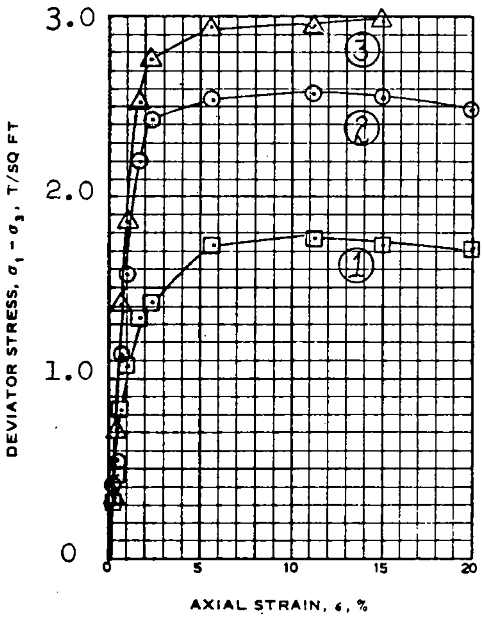
TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

Fig. A - 48

c = 1.22	T/SF
ϕ = 0	DEG
TAN ϕ = 0	





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	29.1	28.8	28.1	
	DRY DENSITY LB/CU FT	γ_{d_o}	91.0	94.1	94.4	
	SATURATION, %	S_o	91.4	97.4	95.6	
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.76	2.58	2.98	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	146	146	197	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS CLAY(CH), light gray, crumbly, contains strata of plastic clay

LL 50	PL 19	PI 31	G: 2.72

TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971) RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO.6

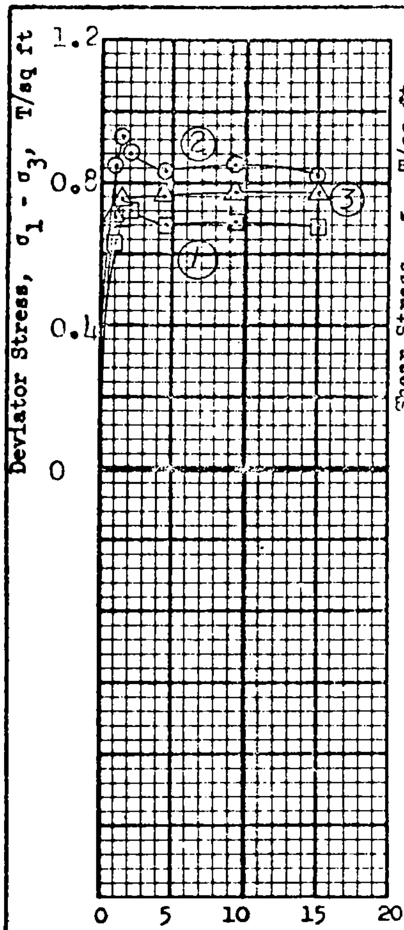
BORING NO. X-1-U | SAMPLE NO. 18-D

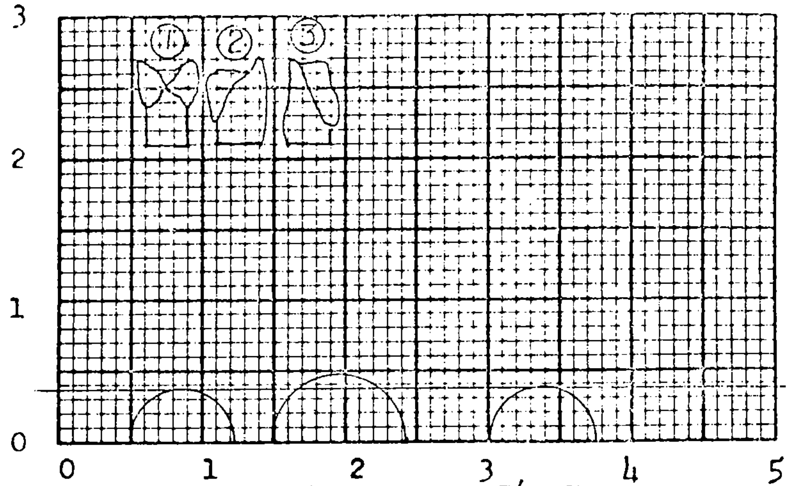
~~DEPTH~~/ELEV -92.0 MSL

LABORATORY USAEWES | DATE 13 Oct. 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.37 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	47.3%	46.4%	48.3%	%
	Void ratio	e_o	1.30	1.28	1.32	
	Saturation	S_o	99.3 %	99.0 %	99.7 %	%
	Dry density, lb/cu ft	γ_d	74.0	74.6	73.5	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.72	0.93	0.77	
Time to failure, min		t_f	13	10	63	
Rate of strain, percent/min			0.148	0.148	0.148	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | **Type of specimen** UNDISTURBED

Classification PLASTIC CLAY(CH),gray, contains shell fragments and pockets*

LL 67 | **PL** 18 | **PI** 49 | G_s 2.73

Remarks *of fine sand

Project LK.PONT.LA.,& VIC.- HURR. PROT.- RIGOLETS CONTROL STRUCT. & CLOSURE DAM

Area DDM # 6

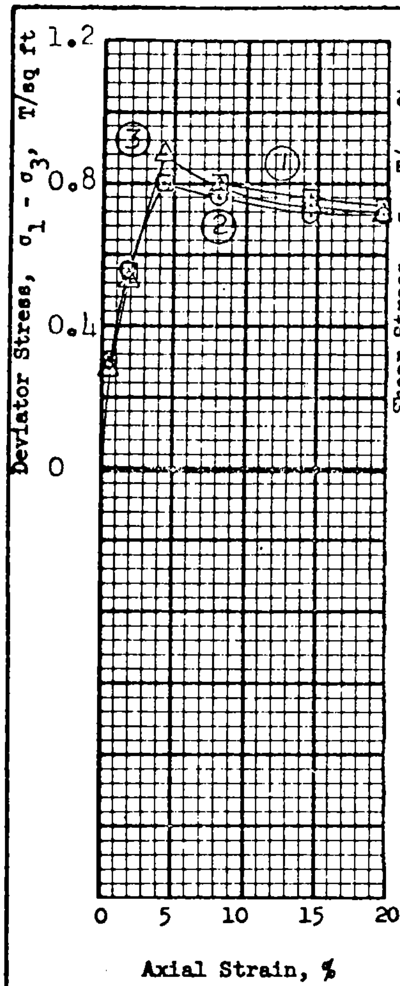
Boring No. X3-U | **Sample No.** 12-D

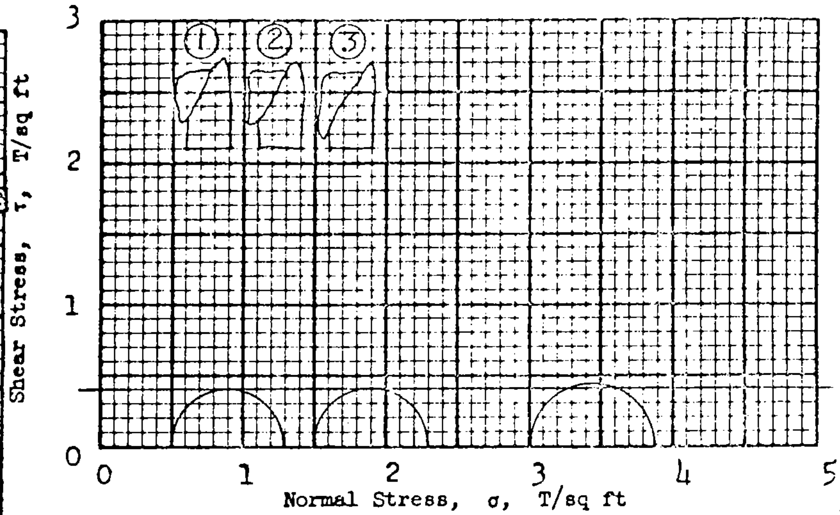
Depth El - 64.8 MSL | **Date** 8 June 1971

TES **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.40 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

				1	2	3	
Test No.				1	2	3	
Initial	Water content		w_o	45.6%	46.3%	47.5%	%
	Void ratio		e_o	1.25	1.26	1.29	
	Saturation		S_o	98.5 %	99.2%	99.4%	%
	Dry density, lb/cu ft		γ_d	75.0	74.6	73.6	
Before Shear	Water content		w_c	%	%	%	%
	Void ratio		e_c				
	Saturation		S_c	%	%	%	%
	Final back pressure, T/sq ft		u_o				
Final	Water content		w_f	%	%	%	%
	Void ratio		e_f				
Minor principal stress, T/sq ft			σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft			$(\sigma_1-\sigma_3)_{max}$	0.80	0.80	0.88	
Time to failure, min			t_f	33	33	33	
Rate of strain, percent/min				0.129	0.129	0.129	
Ult deviator stress, T/sq ft			$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.			D_o	1.40	1.41	1.40	
Initial height, in.			H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH), gray, contains numerous shell fragments

LL 63 | PL 18 | PI 45 | | G_s 2.70

Remarks

Project LK. PONT. LA., & VIC.-HURR. PROT.- RIGOLETS CONTROL STRUCT. & CLOSURE DAM

Area DDM # 6

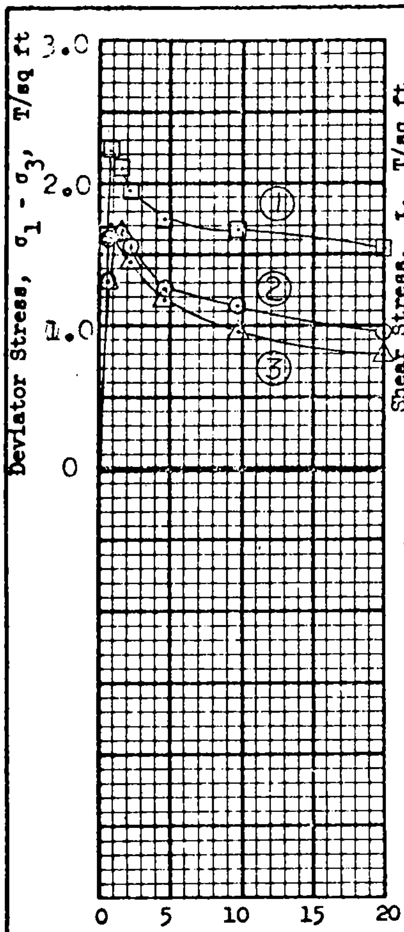
Boring No. X3-U | Sample No. 13-D

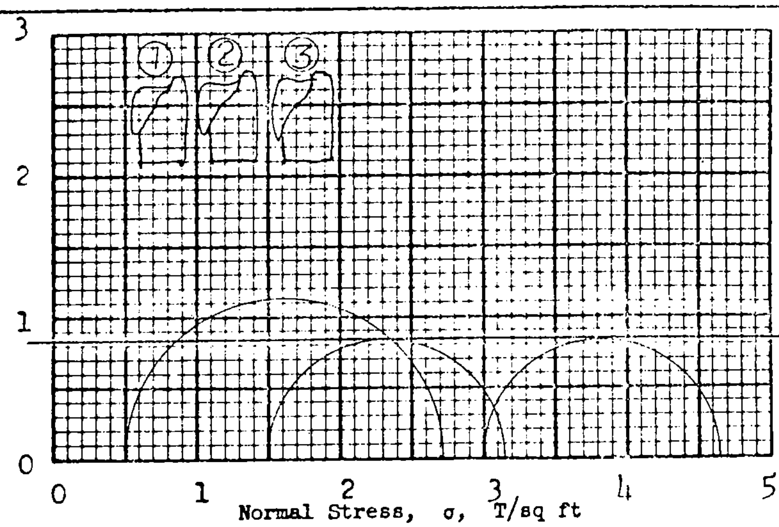
Depth El - 68.7 MSL | Date 8 June 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS





Test No.			1	2	3	
Initial	Water content	w_o	43.4 %	45.8 %	45.6 %	%
	Void ratio	e_o	1.21	1.28	1.28	
	Saturation	S_o	99.0 %	98.8 %	98.3 %	%
	Dry density, lb/cu ft	γ_d	78.1	75.7	75.7	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.23	1.65	1.67	
Time to failure, min		t_f	6	12	12	
Rate of strain, percent/min			0.124	0.124	0.124	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.83 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q	Type of specimen UNDISTURBED			
Classification PLASTIC CLAY(CH),gray, contains shell fragments				
LL 82	PL 23	PI 59		G_s 2.76

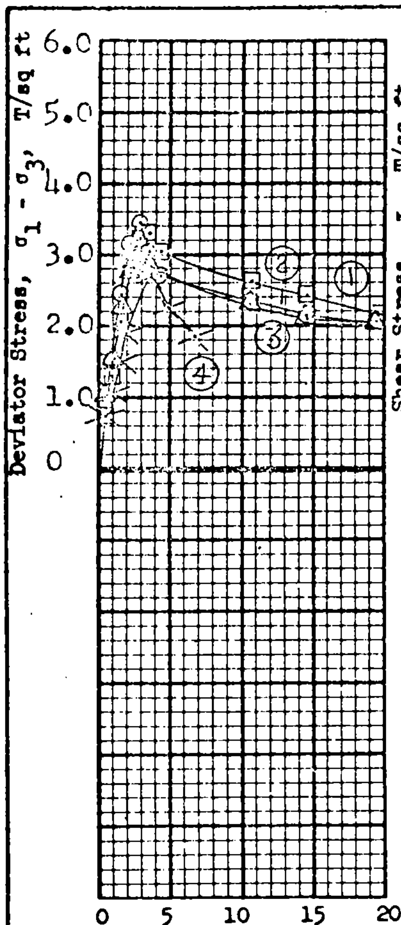
Remarks ______

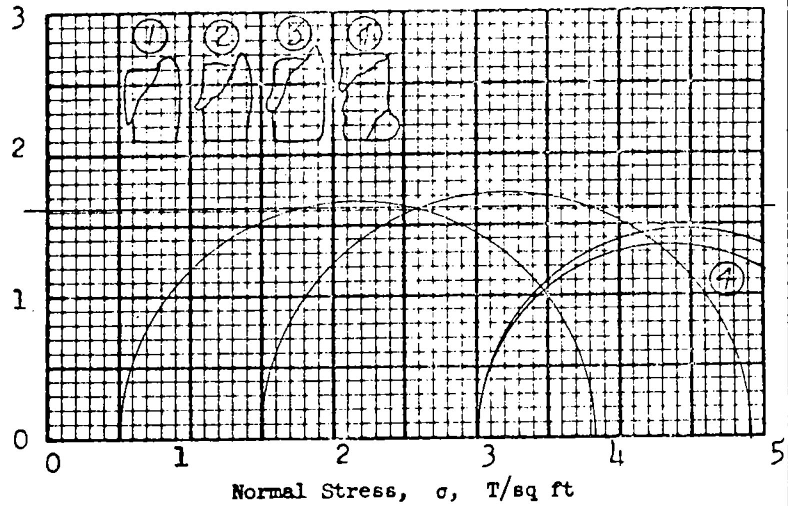
Project LK.PONT.LA.,&VIC.- HURR.PROT.- RIGOLETS CONTROL STRUCT. AND CLOSURE DAM	
Area	
Boring No. X3-U	Sample No. 17-B
Depth El - 82.8 MSL	Date 9 June 1971
TES	TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOIL

Fig. A - 52





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.61 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	4
Initial	Water content	w_o	65.1 %	64.7 %	72.8 %	77.8 %
	Void ratio	e_o	1.60	1.60	1.79	1.92
	Saturation	S_o	98.9 %	98.3 %	98.8 %	98.5 %
	Dry density, lb/cu ft	γ_d	58.4	58.4	54.3	51.9
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	3.0
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	3.34	3.42	2.91	2.69
Time to failure, min		t_f	30	24	24	51
Rate of strain, percent/min			0.121	0.121	0.121	0.073
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.41	1.40	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Type of test Q	Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),dark brown, contains finely divided organic*

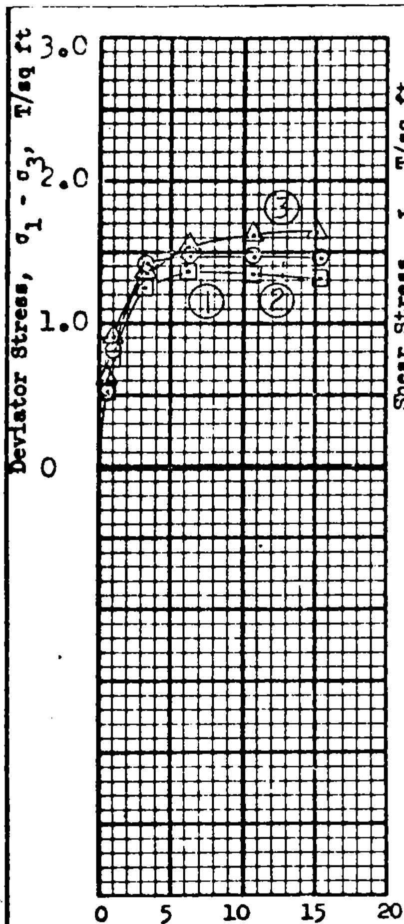
LL 115	PL 38	PI 77		G_s 2.43

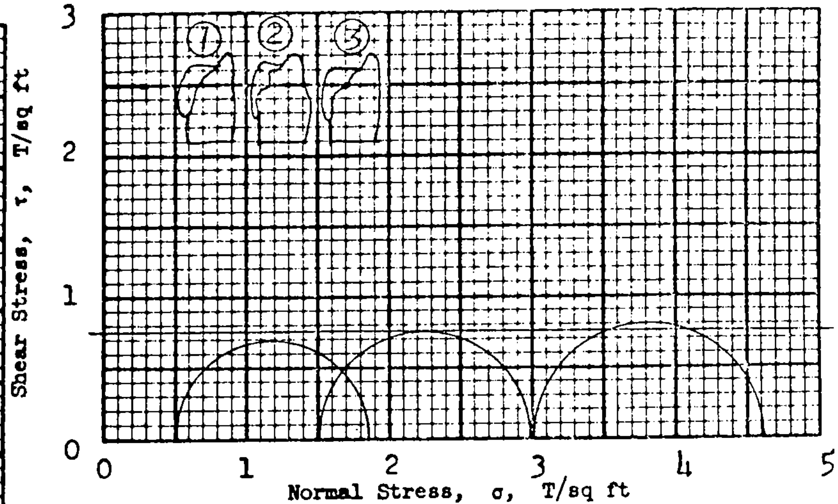
Remarks *matter and root fragments

Project LK.PONT.LA.,& VIC.- HURR. PROT. - RIGOLETS CONTROL STRUCT. AND CLOSURE DAM	
Area	
Boring No. X 3 - U	Sample No. 20 - B
Depth El - 94.9 MSL	Date 9 June 1971
TES TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOIL





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.75 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	27.0 %	28.1 %	27.2 %	%
	Void ratio	e_o	0.743	0.775	0.748	
	Saturation	S_o	99.2 %	99.0 %	99.3 %	%
	Dry density, lb/cu ft	γ_d	97.8	96.0	97.5	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.36	1.49	1.63	
Time to failure, min		t_f	43	43	106	
Rate of strain, percent/min			0.144	0.144	0.144	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q	Type of specimen UNDISTURBED		
Classification PLASTIC CLAY(CH), gray, contains small sand pockets and *			
LL 54	PL 16	PI 38	G_s 2.73

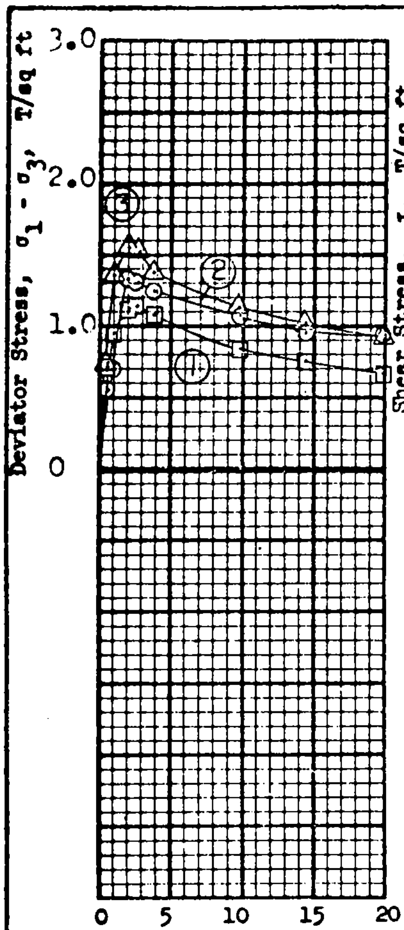
Remarks *decayed root fragments

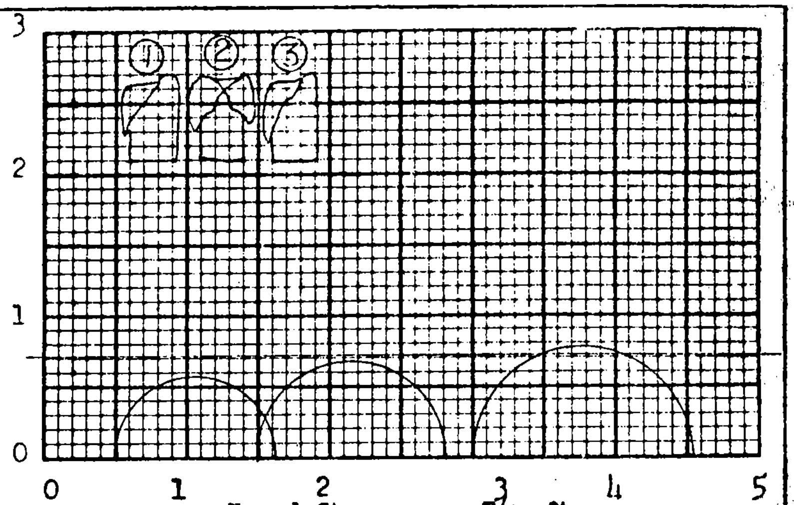
Project LK PONT. LA., & VIC. – HURR. PROT. – RIGOLETS CONTROL STURCT. AND CLOSURE DAM	
Area	
Boring No. X 5 – U	Sample No. 13-B
~~Depth~~ El – 69.6 MSL	Date 10 June 1971
TES	TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE *TRANSLUCENT*

USAEWES SOILS TEST SECTION

Fig. A - 54





Test No.			1	2	3	
Initial	Water content	w_o	52.4 %	54.6%	53.9%	%
	Void ratio	e_o	1.46	1.52	1.51	
	Saturation	S_o	98.7 %	98.8 %	98.2 %	%
	Dry density, lb/cu ft	γ_d	69.8	68.2	68.5	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.13	1.32	1.55	
Time to failure, min		t_f	15	11	11	
Rate of strain, percent/min			0.182	0.182	0.182	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.72 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q	Type of specimen UNDISTURBED
Classification PLASTIC CLAY(CH),gray	

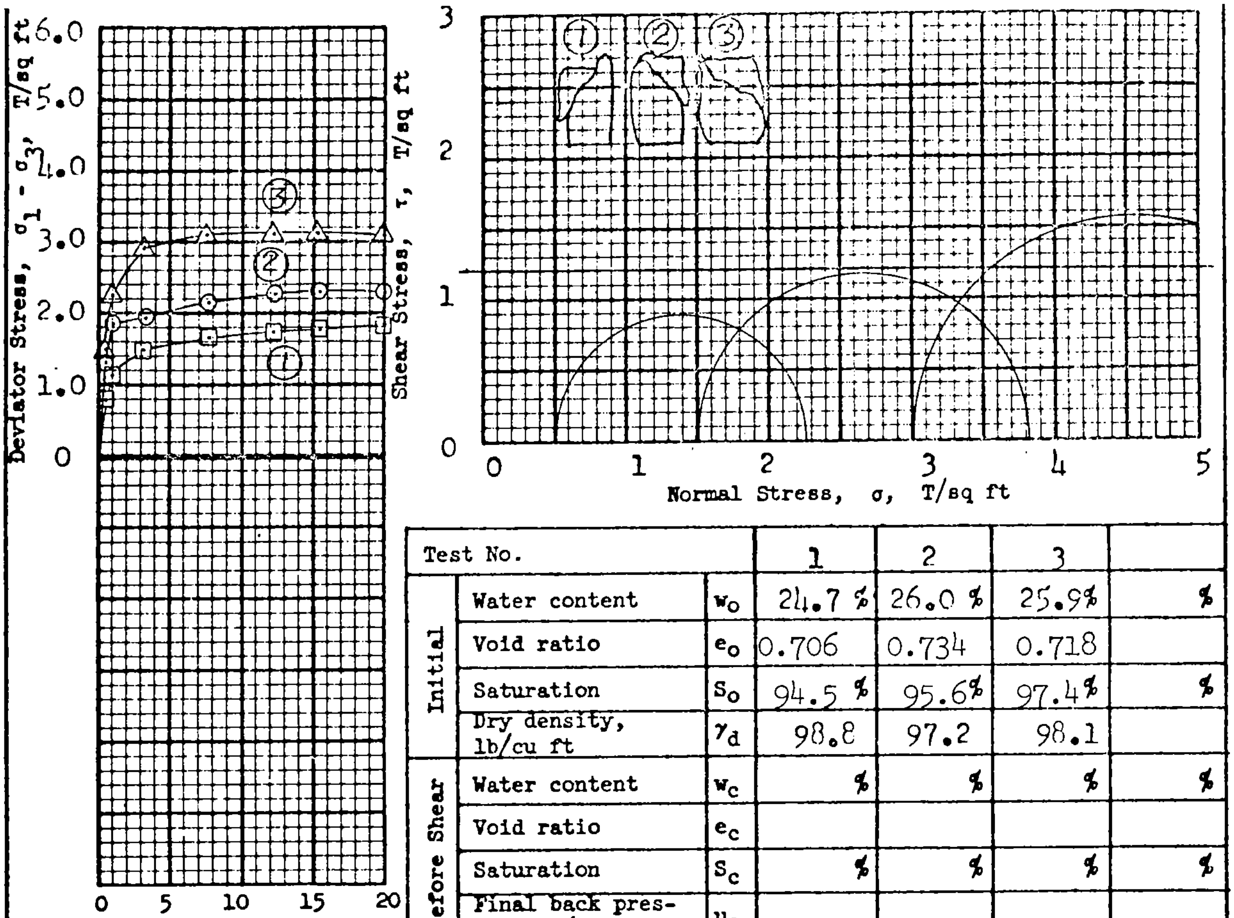
LL 96	PL 26	PI 70		G_s 2.75

Remarks ______

Project LK.PONT.LA.,& VIC.- HURR. PROT.- RIGOLETS CONTROL STRUCT. AND CLOSURE DAM	
Area	
Boring No. X5-U	Sample No. 15-D
~~Depth~~ El - 79.7 MSL	Date 10 June 1971
TES TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS [illegible]



Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.20 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

			1	2	3	
Test No.			1	2	3	
Initial	Water content	w_o	24.7 %	26.0 %	25.9%	%
Initial	Void ratio	e_o	0.706	0.734	0.718	
Initial	Saturation	S_o	94.5 %	95.6%	97.4%	%
Initial	Dry density, lb/cu ft	γ_d	98.8	97.2	98.1	
Before Shear	Water content	w_c	%	%	%	%
Before Shear	Void ratio	e_c				
Before Shear	Saturation	S_c	%	%	%	%
Before Shear	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
Final	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.76	2.31	3.10	
Time to failure, min		t_f	103	103	103	
Rate of strain, percent/min			0.149	0.149	0.149	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.41	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q	Type of specimen UNDISTURBED			
Classification PLASTIC CLAY(CH), gray				
LL 50	PL 19	PI 31		G_s 2.70

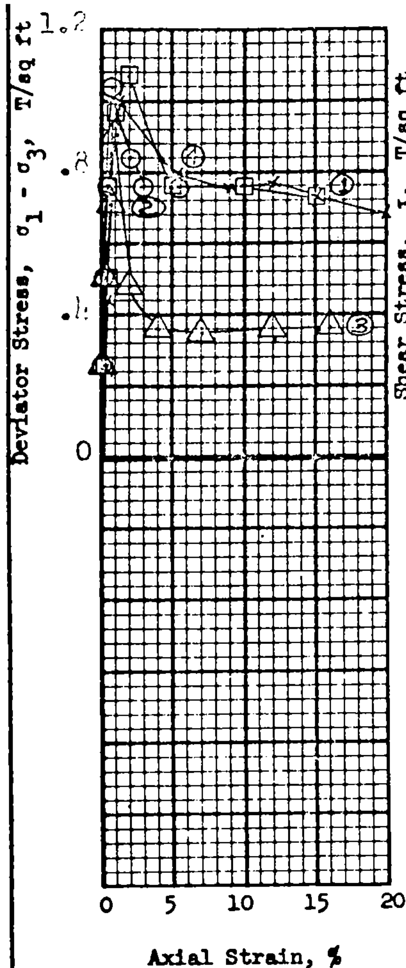
Remarks ______

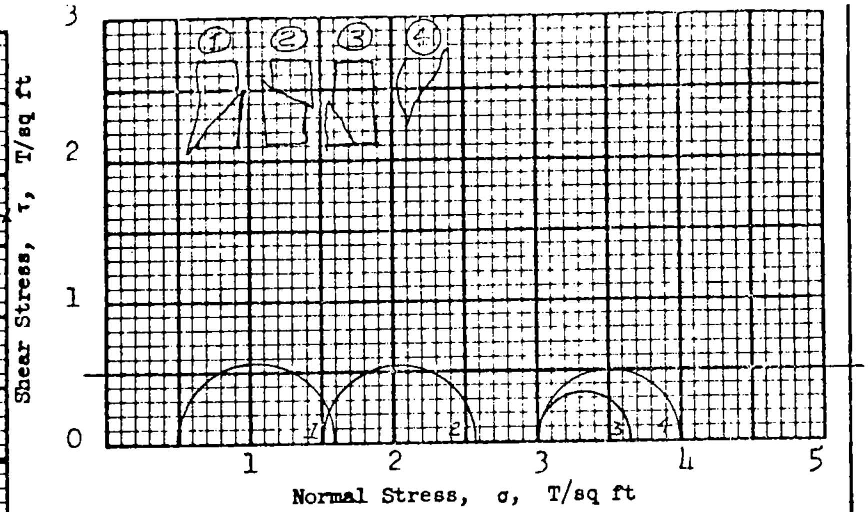
Project LK. PONT. LA., & VIC.- HURR. PROT.- RIGOLETS CONTROL STURCT. AND CLOSURE DAM	
Area	
Boring No. X 5 U	Sample No. 18-B
Depth El - 89.8 MSL	Date 15 June 1971
TES TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS TEST SECTION

Fig. A - 56





			1	2	3	4
Test No.			1	2	3	4
Initial	Water content	w_o	50.6 %	51.7 %	52.3 %	52.1 %
Initial	Void ratio	e_o	1.42	1.45	1.46	1.50
Initial	Saturation	S_o	98.7 %	98.8 %	99.2 %	96.2 %
Initial	Dry density, lb/cu ft	γ_d	71.4	70.6	70.2	69.2
Before Shear	Water content	w_c	%	%	%	%
Before Shear	Void ratio	e_c				
Before Shear	Saturation	S_c	%	%	%	%
Before Shear	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
Final	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	3.0
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.07	1.04	0.66	1.00
Time to failure, min		t_f	8	16	20	13
Rate of strain, percent/min			0.158	0.496	0.060	0.090
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.50 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH), gray, containing shell fragments.

LL 88 | PL 25 | PI 63 | G_s 2.77

Remarks ________

Project LK. PONT., LA. HURR. PROT. RIGOLETS CONT. & STR. ℄ CLOSURE DAM

Area

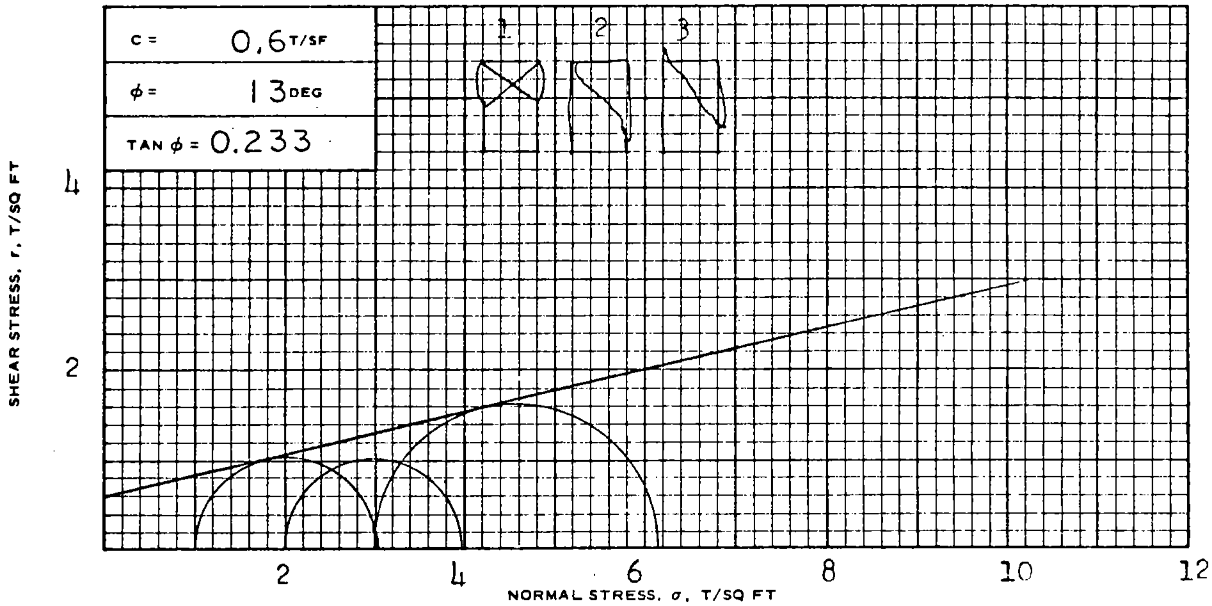
Boring No. X-7-U | Sample No. 8-D

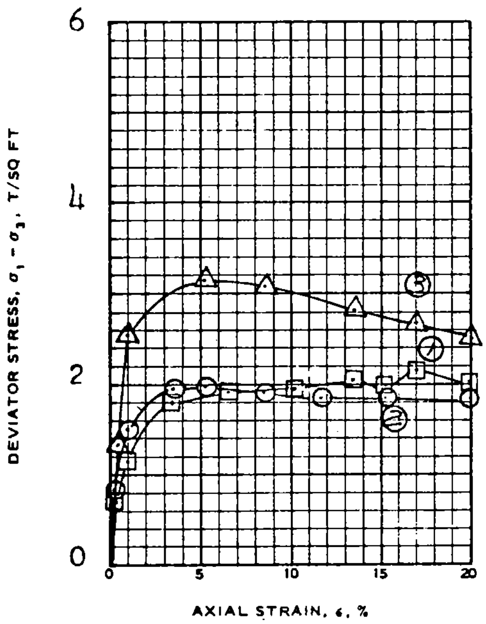
Depth El -64.7 MSL | Date 4 October 1971

GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE *TRANSLUCENT*

USAEWES SOILS TEST SECTION





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	34.0	35.0	33.6
	DRY DENSITY LB/CU FT	γd_o	86.7	85.5	87.2
	SATURATION, %	S_o	96.1	96.2	96.2
	VOID RATIO	e_o	0.966	0.993	0.951
BEFORE SHEAR	WATER CONTENT, %	w_c	33.0	31.8	31.6
	DRY DENSITY LB/CU FT	γd_c	89.7	91.5	92.8
	SATURATION, %	S_c	100+	100+	100+
	VOID RATIO	e_c	0.900	0.863	0.836
	FINAL BACK PRESSURE, T/SQ FT	u_o	4.68	4.68	4.68
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.03	1.99	3.15
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	243	92	92
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.41	1.42	1.42
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Alternate seams of PLASTIC CLAY (CH) and CLAYEY SILT (ML), gray.

LL 48 | PL 19 | PI 29 | Gs 2.73 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST R

REMARKS: Atterberg limits on mixture of materials.

PROJECT LK. PONT., LA. & VIC. - HURR. PROT.
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

BORING NO. Y-7U | SAMPLE NO. 12-A

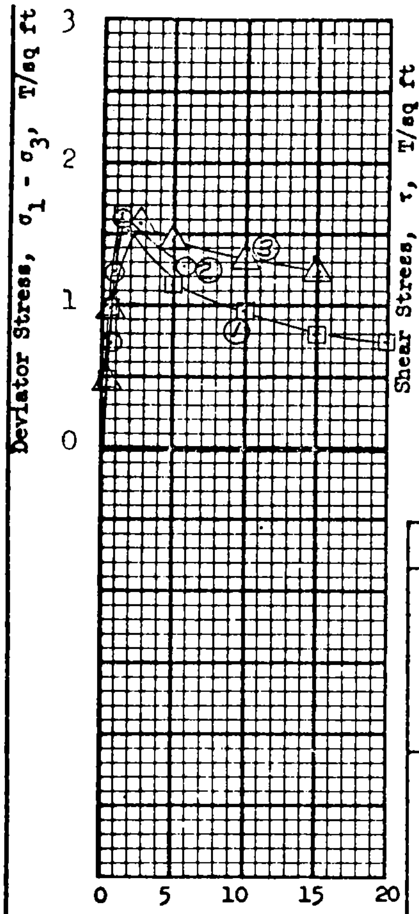
DEPTH/ELEV -78.1 MSL

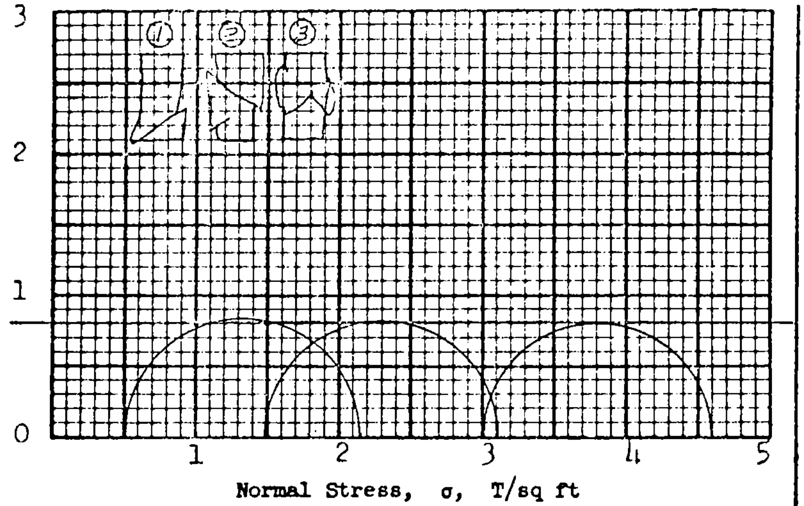
LABORATORY USAEWES | DATE 13 October 1973

JAL TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

USAEWES SOILS TEST SECTION





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.80 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	49.2 %	50.0 %	51.2 %	%
	Void ratio	e_o	1.37	1.39	1.43	
	Saturation	S_o	98.8 %	98.9 %	98.5 %	%
	Dry density, lb/cu ft	γ_d	72.3	71.9	70.6	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.64	1.61	1.59	
Time to failure, min		t_f	7	32	50	
Rate of strain, percent/min			0.175	0.411	0.545	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.41	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q **Type of specimen** UNDISTURBED

Classification PLASTIC CLAY (CH), bluish gray.

LL 96 **PL** 27 **PI** 69 G_s 2.75

Remarks ______

Project LK. PONT., LA. HURR. PROT. RIGOLETS CONT. & STR. & CLOSURE DAM 1971

Area

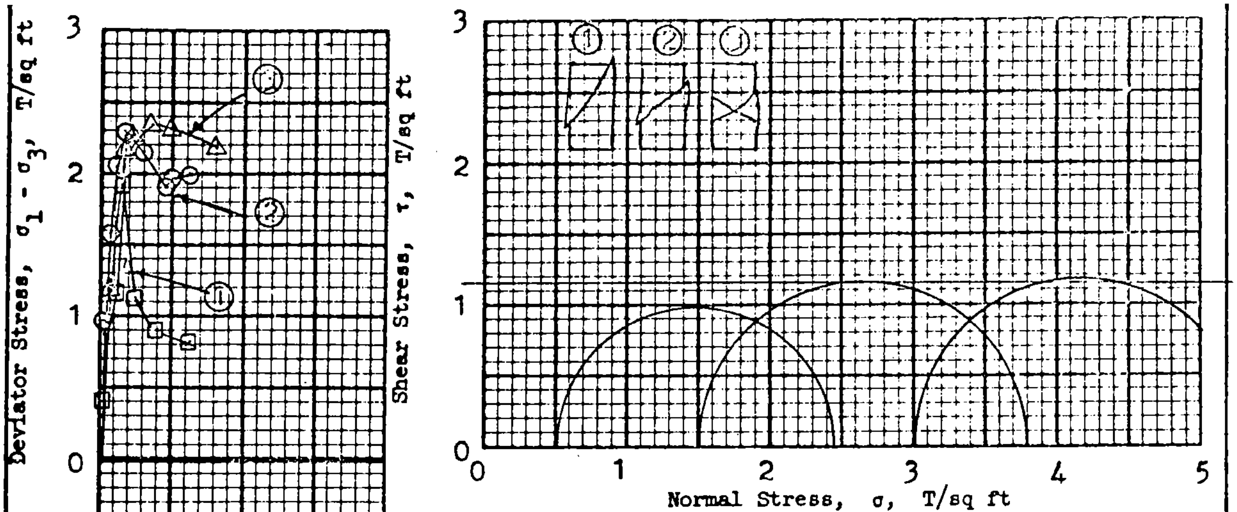
Boring No. X-7-U **Sample No.** 13-D

~~Depth~~ El -84.7 MSL **Date** 8 OCTOBER 1971

FAM TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE *TRANSLUCENT* USAEWES SOILS TEST SECTION

Fig. A - 59



Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.15 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	31.3 %	30.7 %	32.0 %	%
	Void ratio	e_o	0.888	0.884	0.909	
	Saturation	S_o	96.6 %	95.2 %	96.4 %	%
	Dry density, lb/cu ft	γ_d	90.6	90.8	89.6	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.96	2.30	2.34	
Time to failure, min		t_f	18	23	44	
Rate of strain, percent/min			0.107	0.073	0.082	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, crumbly; contains silt seams.

LL 66 | PL 21 | PI 45 | | G_s 2.74

Remarks ________

Project LK. PONT., LA.& VIC.-HURR. PROTECTION
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

Area DDM NO. 6 (1971)

Boring No. X-7U | Sample No. 16-C

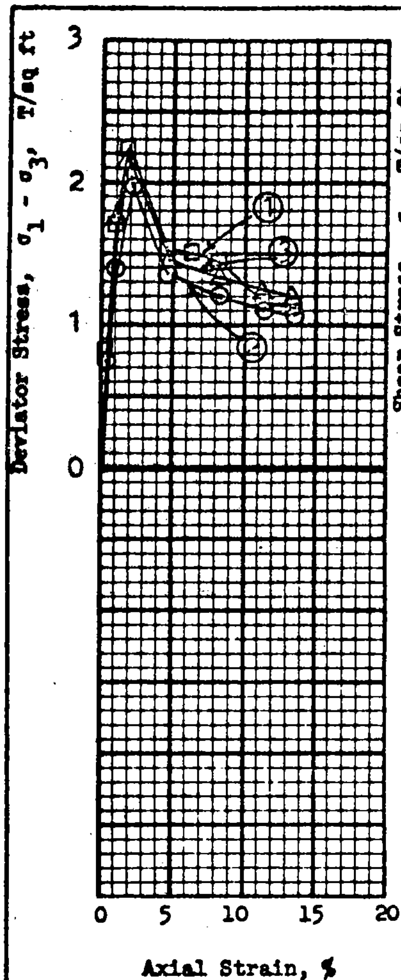
~~Depth~~ El -95.5 MSL | Date 9 September, 1971

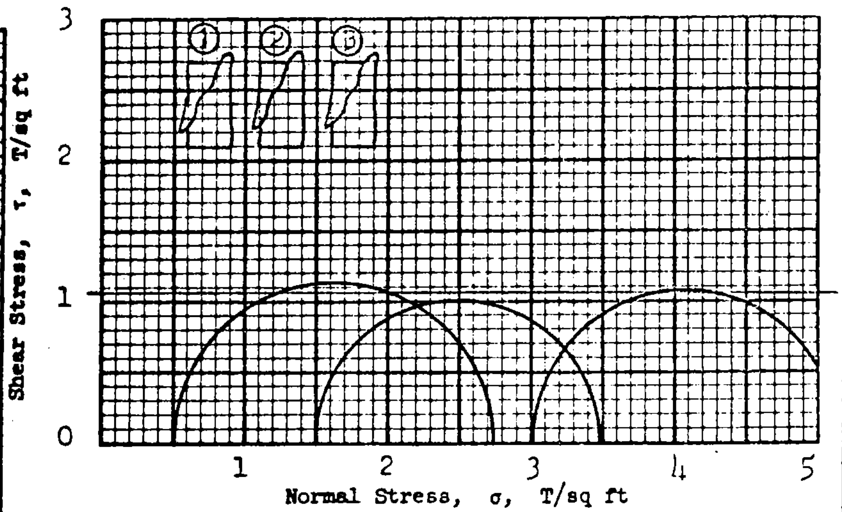
GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE USAEWES SOILS TEST SECTION

TRANSLUCENT

Fig. A - 60





Test No.			1	2	3	
Initial	Water content	w_o	44.9 %	44.7%	45.3 %	%
	Void ratio	e_o	1.23	1.23	1.24	
	Saturation	S_o	100 %	99.6 %	100+ %	%
	Dry density, lb/cu ft	γ_d	76.7	76.8	76.2	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.23	1.98	2.17	
Time to failure, min		t_f	20	25	20	
Rate of strain, percent/min			0.087	0.087	0.087	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.41	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.08 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, fissured.

LL 80 | PL 25 | PI 55 | G_s 2.74

Remarks ______

Project LK. PONT., LA.& VIC.-HURR. PROT.
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

Area DDM NO. 6

Boring No. X-7U | Sample No. 22-D

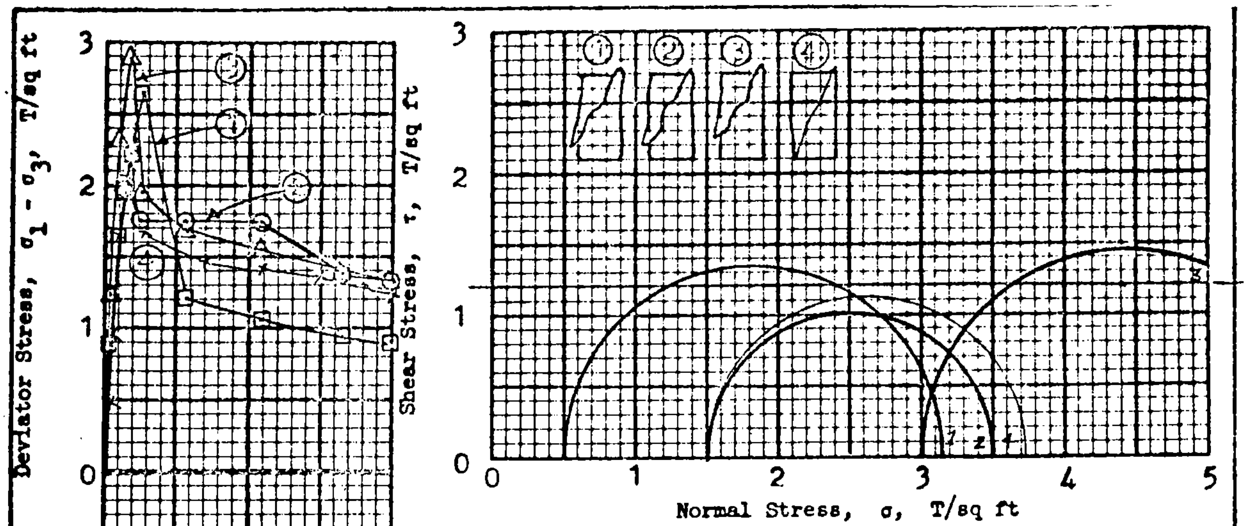
~~Depth~~ El -120.7 MSL | Date 30 September, 1971

TFS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE *TRANSLUCENT*

USAEWES SOILS TEST SECTION

Fig. A - 61



Test No.			1	2	3	4
Initial	Water content	w_o	50.8 %	50.0 %	49.8 %	49.9%
	Void ratio	e_o	1.39	1.36	1.36	1.40
	Saturation	S_o	99.8 %	100+ %	100 %	97.3%
	Dry density, lb/cu ft	γ_d	71.3	72.2	72.3	71.0
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	1.5
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.65	1.99	2.89	2.23
Time to failure, min		t_f	29	15	19	21
Rate of strain, percent/min			0.086	0.086	0.086	0.094
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.20 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q	Type of specimen UNDISTURBED		
Classification PLASTIC CLAY (CH), gray, slickensided.			
LL 92	PL 28	PI 64	G_s 2.73

Remarks ______

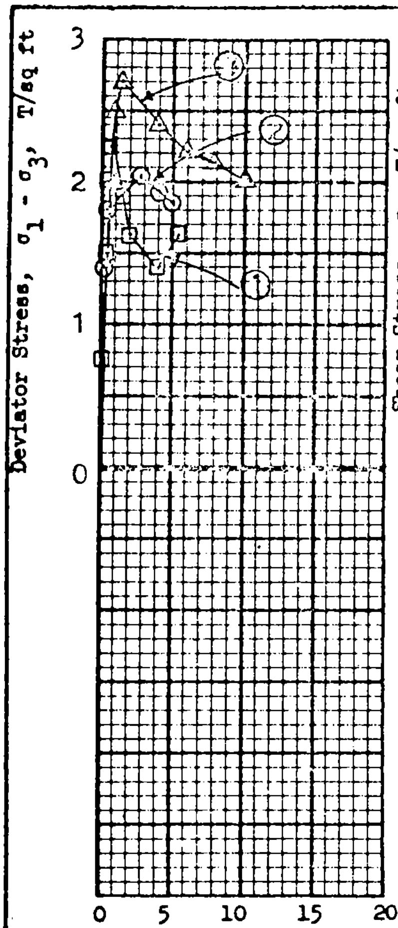
Project LK. PONT., LA.& VIC.-HURR. PROT.
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

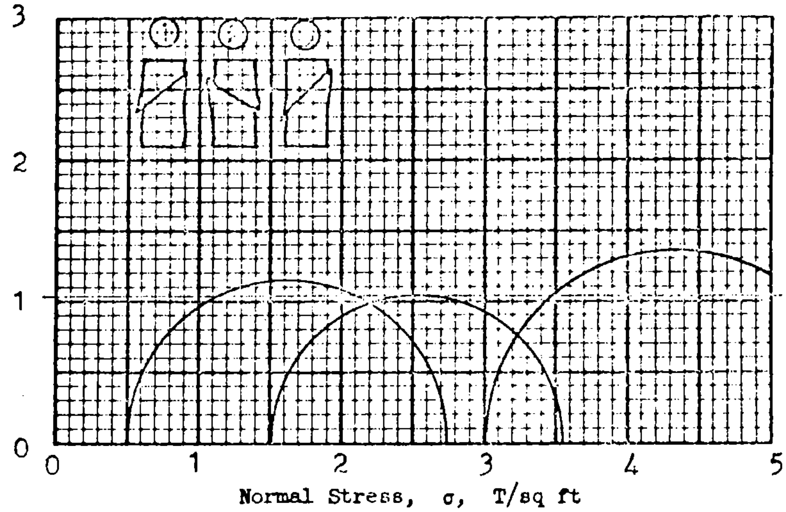
Area DDM NO.6

Boring No. X-7U — Sample No. 29-D

~~Depth~~ El -148.7 MSL — Date 30 September, 1971

TES TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.02 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	36.1 %	36.1 %	38.0 %	%
	Void ratio	e_o	1.02	1.02	1.08	
	Saturation	S_o	98.4 %	98.4%	97.8%	%
	Dry density, lb/cu ft	γ_d	86.1	86.0	83.3	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.24	2.04	2.70	
Time to failure, min		t_f	91	96	18	
Rate of strain, percent/min			0.050	0.029	0.072	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray.

LL 66	PL 24	PI 42		G_s 2.78

Remarks ______

Project LK.PONT.,LA.& VIC.-HURR. PROTECTION
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

Area DDM NO. 6

Boring No. X-7-U | Sample No. 34-D

~~Depth~~ El -168.5 MSL | Date 1 October,1971

GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE TRANSLUCENT

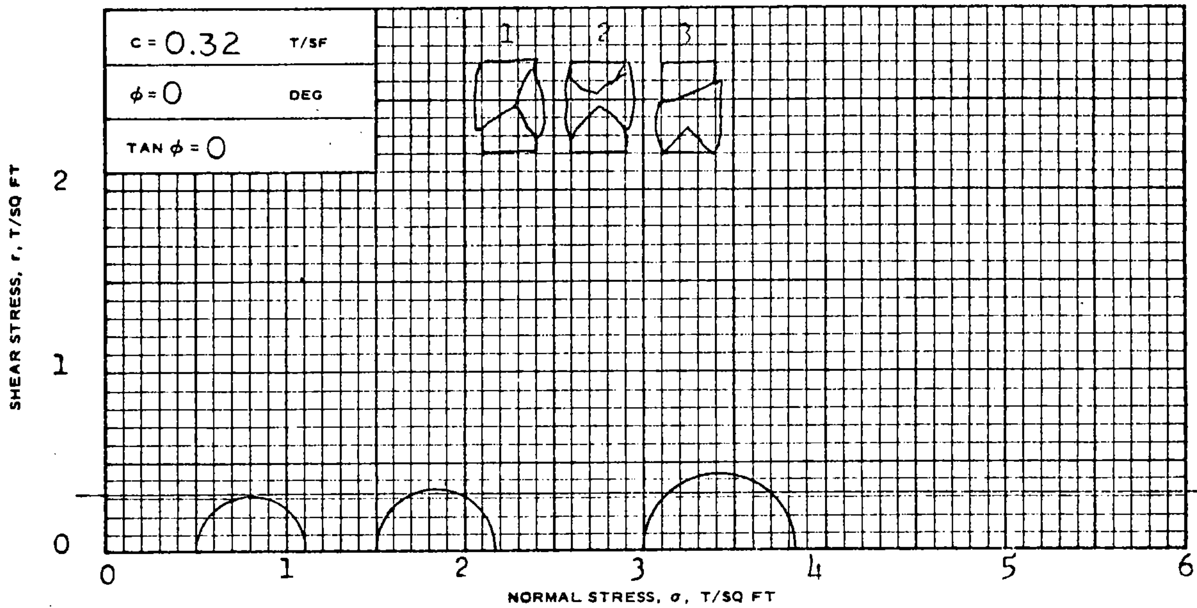
USAEWES SOILS TEST SECTION

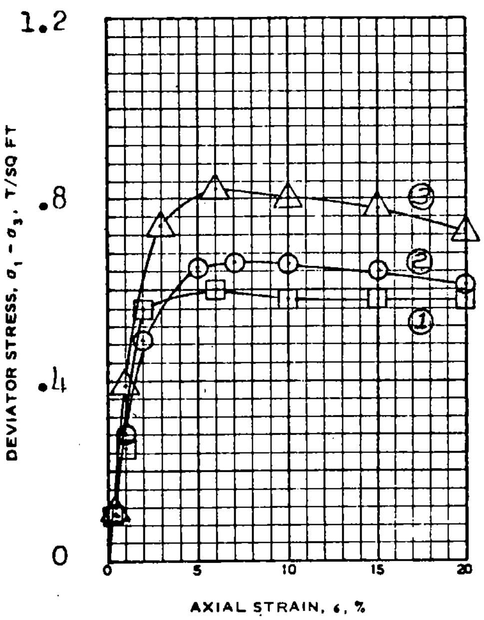
Fig. A - 63

c = 0.32 T/SF

ϕ = 0 DEG

TAN ϕ = 0





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	36.3	36.0	33.0	
	DRY DENSITY LB/CU FT	γd_o	84.0	84.0	88.0	
	SATURATION, %	s_o	97.8	96.0	96.5	
	VOID RATIO	e_o	1.01	1.02	0.930	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	s_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.60	0.66	0.82	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	24	55	27	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.41	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains silty sand pockets and lenses, and scattered shells 1/8" to 1/2" in diameter

LL 41	PL 16	PI 25	Gs 2.72	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS: Atterberg limits performed on mixture of clay and sand.

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM , 1971

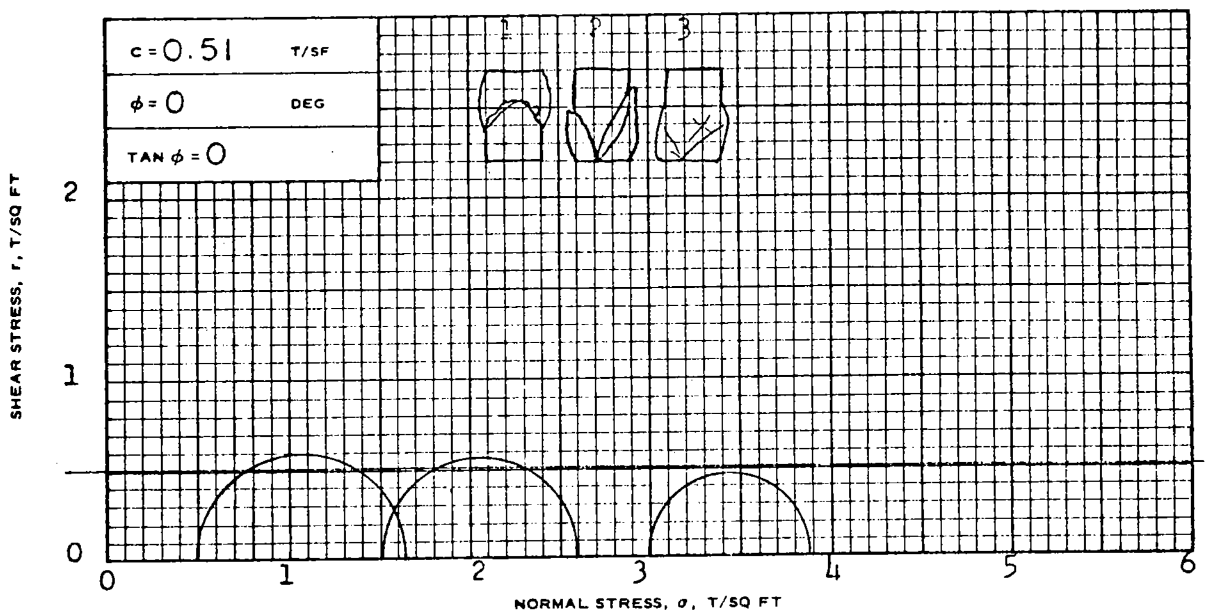
BORING NO. X-8-U SAMPLE NO. 10-C

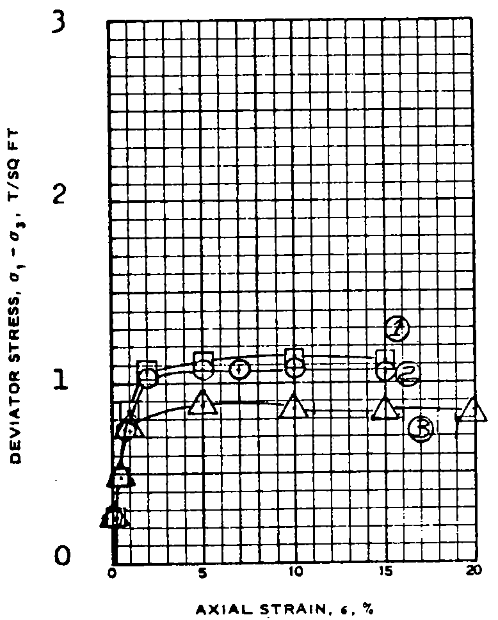
DEPTH/ELEV -59.6 MSL

LABORATORY USAEWES DATE 20 October, 1971

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	29.2	29.2	29.7	
	DRY DENSITY LB/CU FT	γd_o	94.5	94.3	92.9	
	SATURATION, %	S_o	100+	100+	98.9	
	VOID RATIO	e_o	0.777	0.781	0.808	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.13	1.08	0.88	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	97	46	68	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.41	1.41	1.41	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS LEAN CLAYC(CL),gray, contains silt and fine sand

LL 51	PL 16	PI 35	Gs 2.69	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

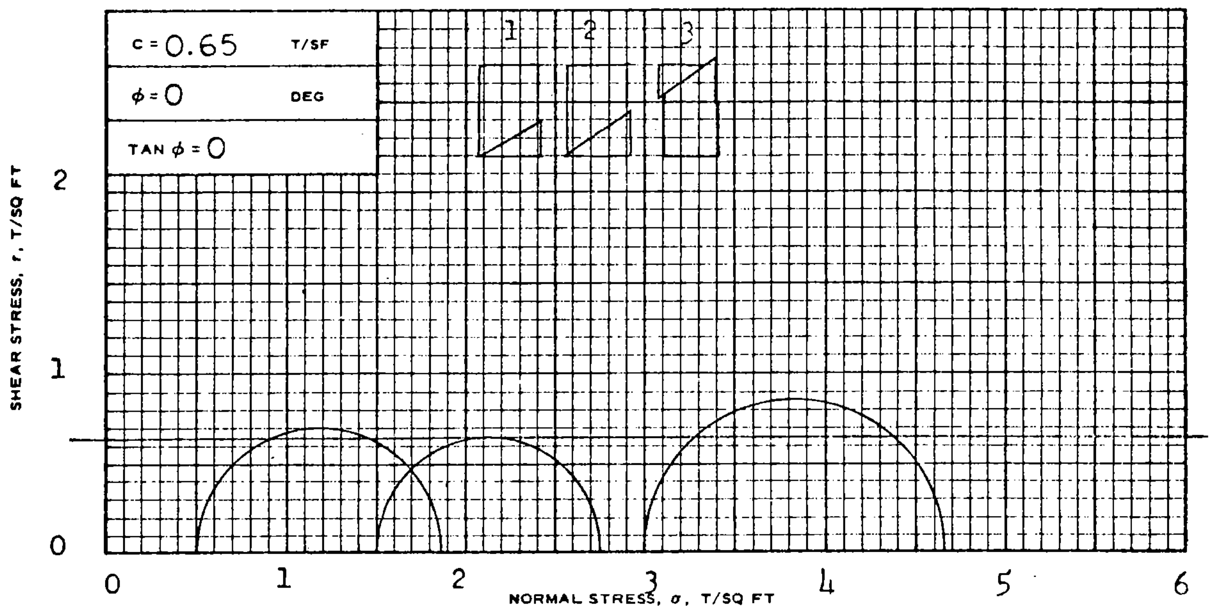
BORING NO. X-8-U SAMPLE NO. 12-D

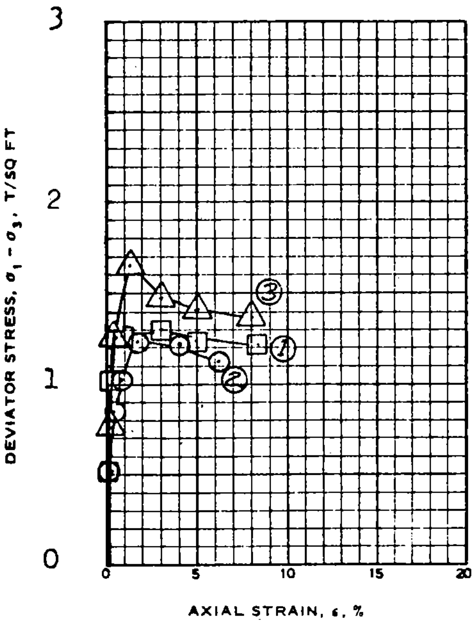
~~DEPTH~~/ELEV -68.5 MSL

LABORATORY USAEWES DATE 21 October, 1971

FAM TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	48.0	49.8	52.7	
	DRY DENSITY LB/CU FT	γd_o	74.9	72.9	70.1	
	SATURATION, %	S_o	100+	100+	99.9	
	VOID RATIO	e_o	1.29	1.36	1.45	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.36	1.24	1.65	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	5	15	27	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

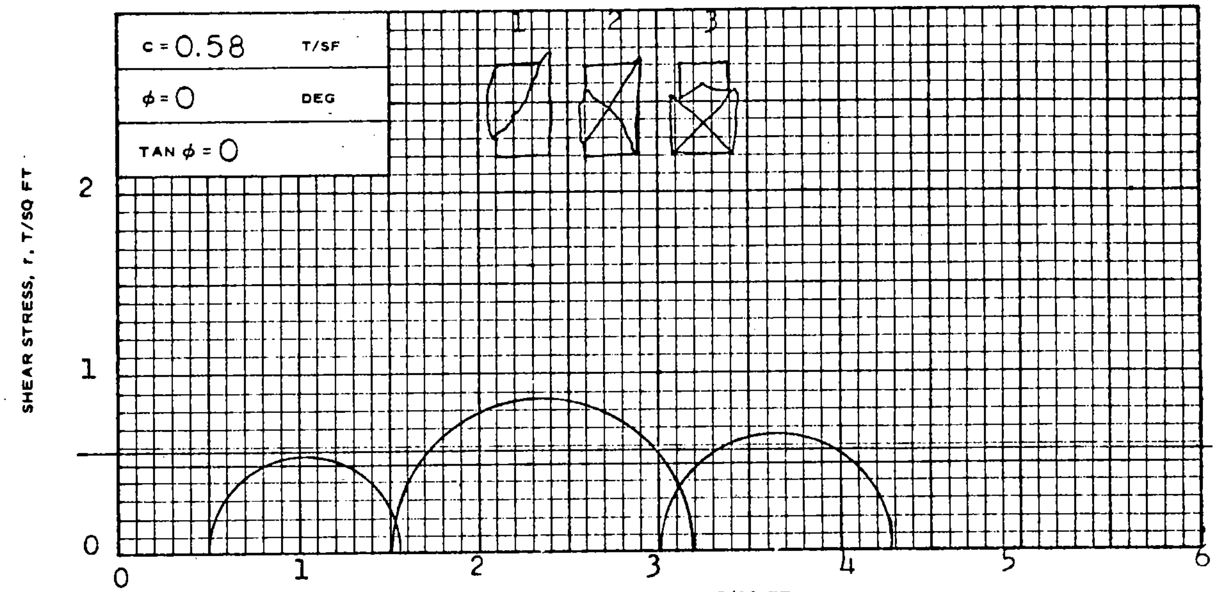
CONTROLLED- Strain TEST

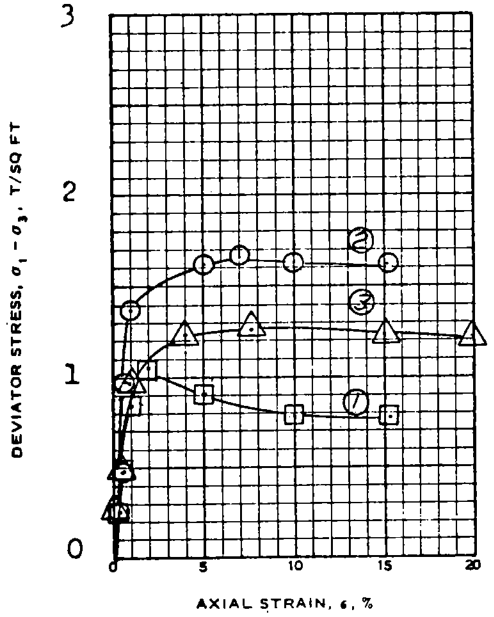
DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, contains 1/16" thick vertical seam of cemented silty sand; concretions

LL 92	PL 24	PI 68	Gs 2.75	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:	PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971
	BORING NO. X-8-U / SAMPLE NO. 15-D
	~~DEPTH~~/ELEV -80.5 MSL
	LABORATORY USAEWES / DATE 1 November 1971
	GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





			1	2	3
SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	25.8	23.5	26.3
	DRY DENSITY LB/CU FT	γ_{d_o}	98.5	103.7	97.7
	SATURATION, %	S_o	98.0	100+	97.9
	VOID RATIO	e_o	0.711	0.625	0.725
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γ_{d_c}			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.05	1.67	1.26
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	7	67	36
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.41	1.41	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS LEAN CLAY(CL), light gray, contains a few small pockets of sand

LL 43 | PL 16 | PI 27 | Gs 2.70 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-8-U | SAMPLE NO. 17-D

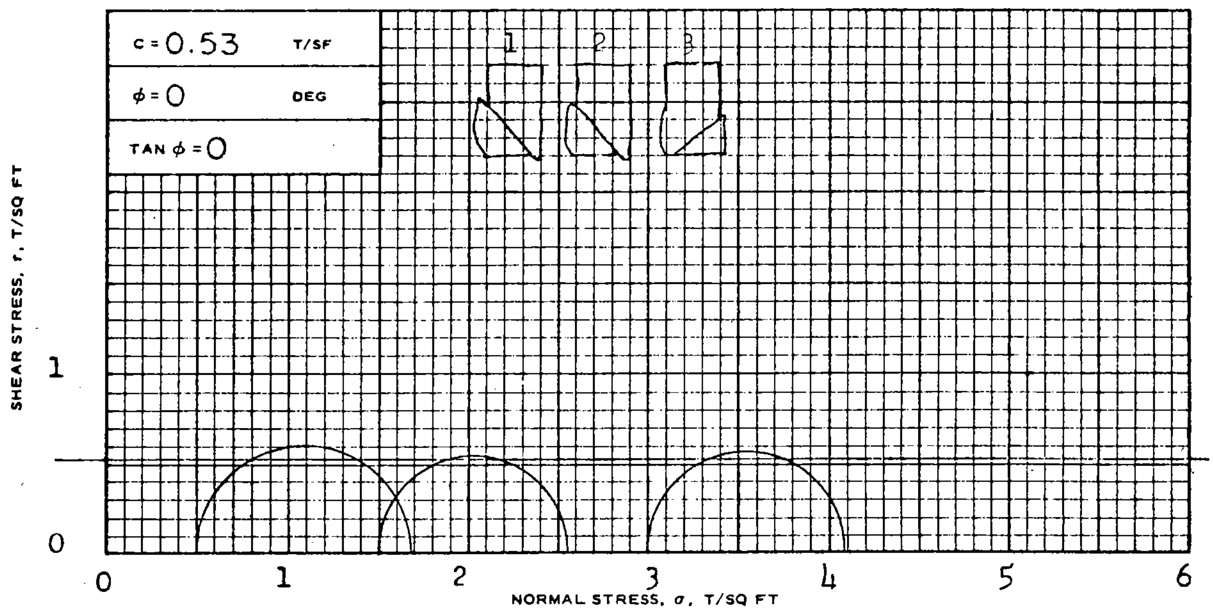
~~DEPTH~~/ELEV -88.5 MSL

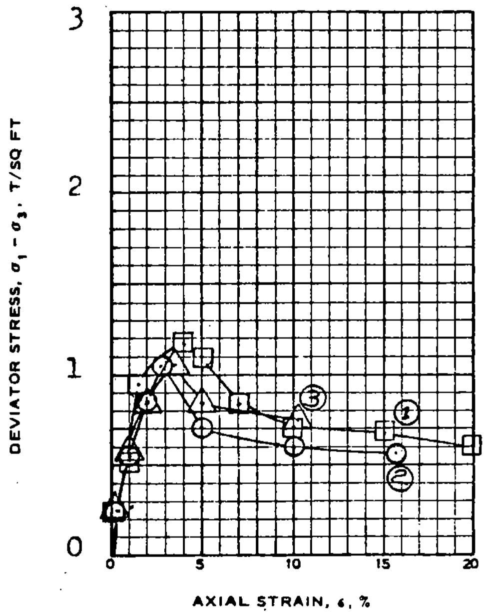
LABORATORY USAEWES | DATE 20 October, 1971

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*

Fig. A - 67





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	51.3	49.6	50.3
	DRY DENSITY LB/CU FT	γd_o	71.2	71.6	71.7
	SATURATION, %	S_o	100+	98.5	99.9
	VOID RATIO	e_o	1.38	1.37	1.37
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.18	1.05	1.07
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	12	12	12
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.41	1.40	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains sandy silt lenses

LL 84 | PL 24 | PI 60 | Gs 2.72 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-9-U SAMPLE NO. 9-B

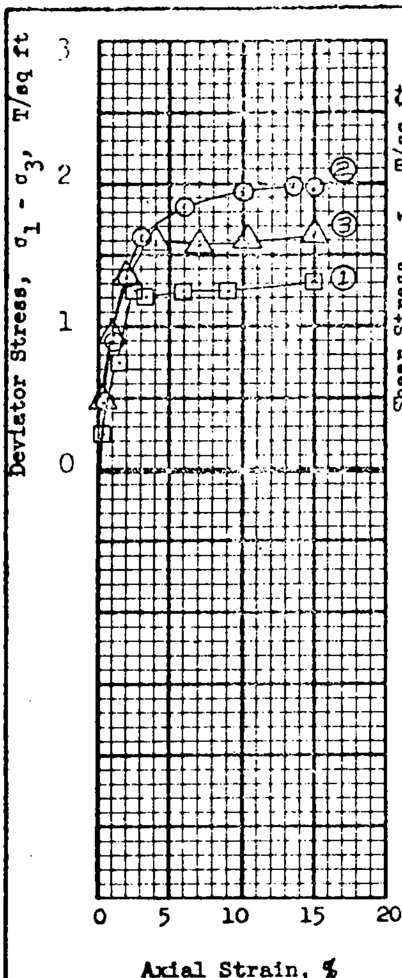
DEPTH/ELEV -62.5 MSL

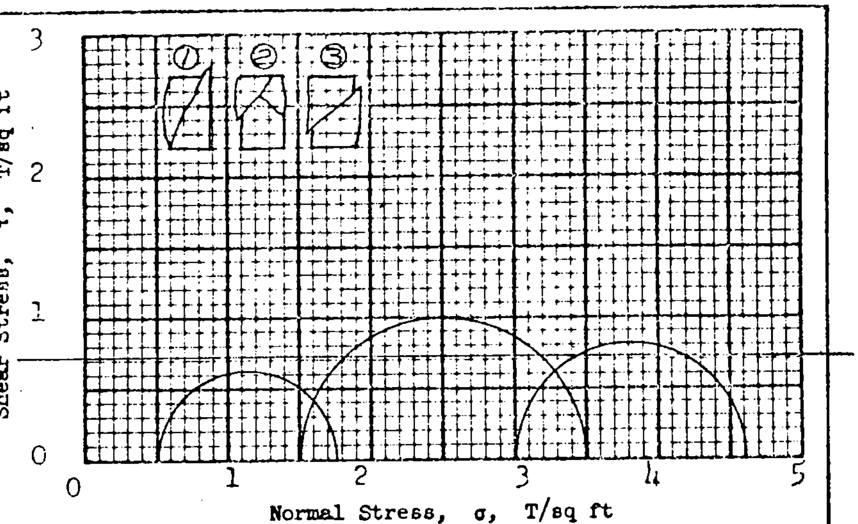
LABORATORY USAWES DATE 21 October, 1971

FAM **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

Fig. A - 68





Test No.			1	2	3	
Initial	Water content	w_o	27.9 %	28.0 %	31.4 %	%
	Void ratio	e_o	0.765	0.748	0.831	
	Saturation	S_o	98.1 %	100+ %	100+ %	%
	Dry density, lb/cu ft	γ_d	96.3	96.1	91.7	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.26	1.99	1.62	
Time to failure, min		t_f	74	80	60	
Rate of strain, percent/min			0.204	0.168	0.252	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.72 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, contains concretions

LL 53 | PL 20 | PI 33 | G_s 2.69

Remarks

Project LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

Area

Boring No. X-9U | Sample No. 11-B

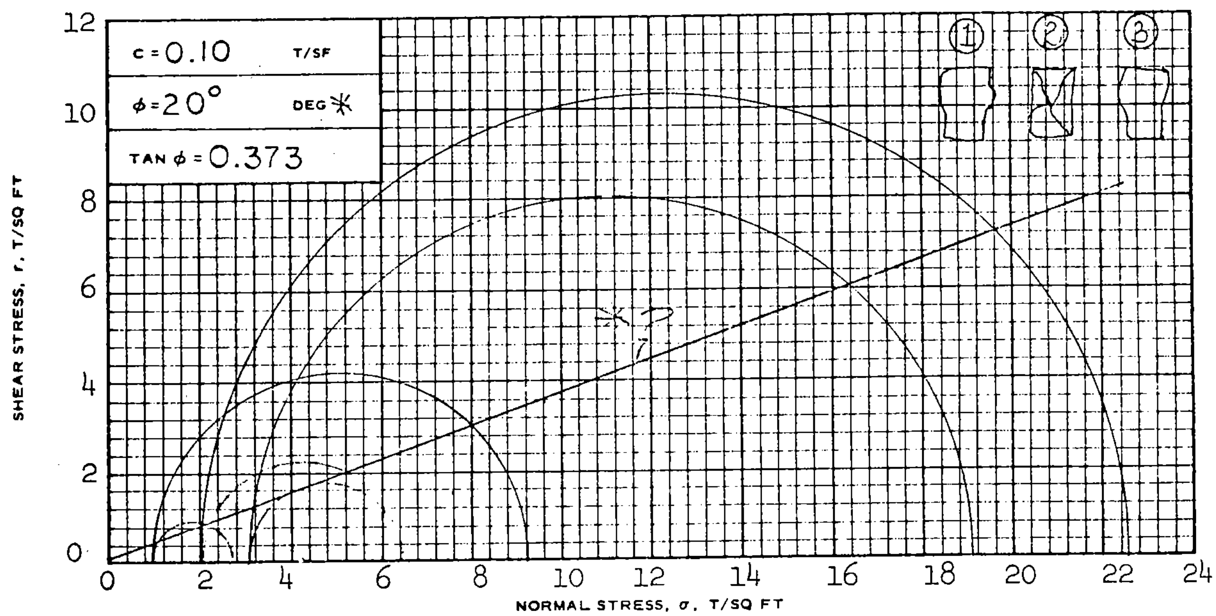
Depth El -70.6 MSL | Date 20 October, 1971

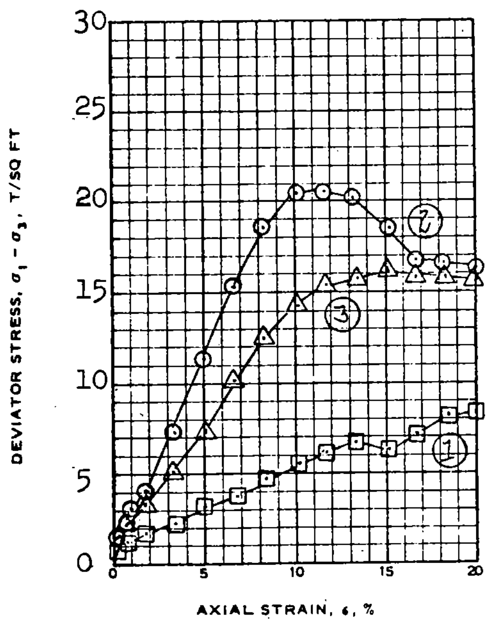
TRIAXIAL COMPRESSION TEST REPORT

JMS

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	30.0	27.4	28.2	
	DRY DENSITY LB/CU FT	γd_o	94.1	95.5	95.0	
	SATURATION, %	S_o	100+	97.1	98.7	
	VOID RATIO	e_o	0.784	0.759	0.768	
BEFORE SHEAR	WATER CONTENT, %	w_c	27.6	26.6	27.0	
	DRY DENSITY LB/CU FT	γd_c	95.8	97.2	97.3	
	SATURATION, %	S_c	98.5	98.4	100+	
	VOID RATIO	e_c	0.754	0.727	0.725	
	FINAL BACK PRESSURE, T/SQ FT	u_o	3.96	3.96	3.96	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	8.29	20.56	16.06	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	216	189	242	
$(\sigma_1 - \sigma_3)$ at max pore pressure *			1.8	4.3	3.5	
INITIAL DIAMETER, IN.		D_o	1.39	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS SILTY SAND(SM),gray

LL NP	PL NP	PI NP	G_s 2.69	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS:

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR.&CLOSURE DAM;DDM # 6

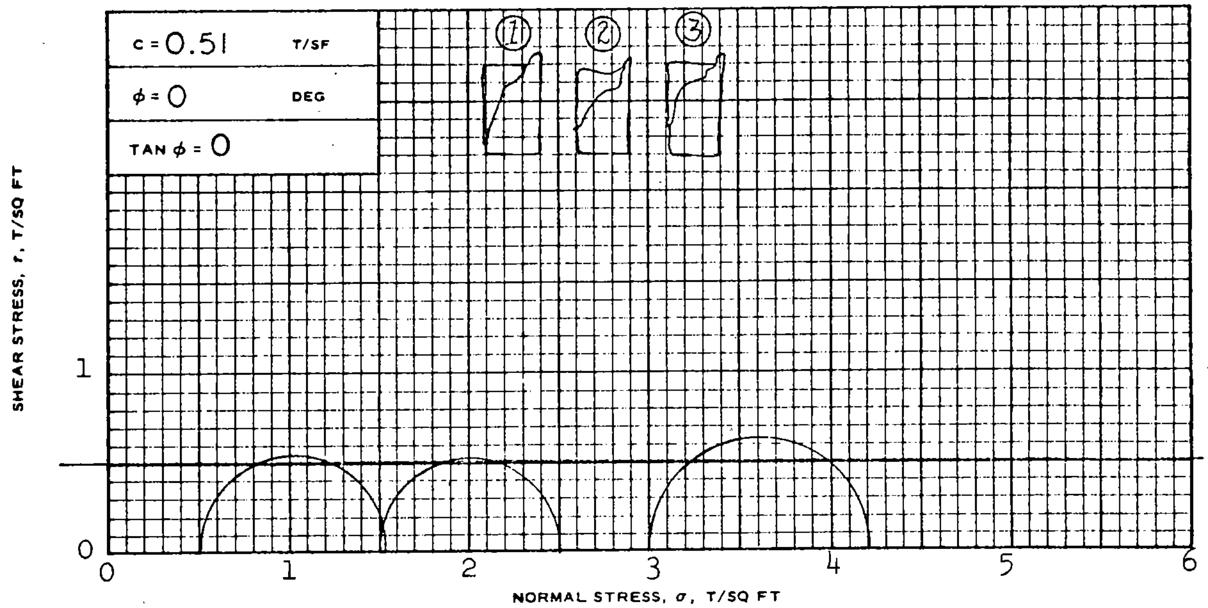
BORING NO. X-9-U | SAMPLE NO. 12-B

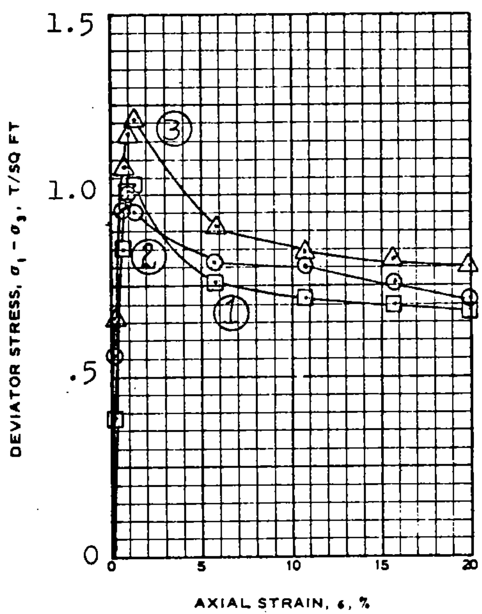
~~DEPTH~~/ELEV -74.3 MSL

LABORATORY USAWES | DATE 18 Oct. 1971

JAL **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	56.8	56.3	58.0	
	DRY DENSITY LB/CU FT	γd_o	66.7	66.9	65.9	
	SATURATION, %	S_o	99.4	99.2	99.6	
	VOID RATIO	e_o	1.56	1.55	1.59	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.03	1.00	1.21	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	148	114	148	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains scattered small concretions

LL 90	PL 22	PI 68	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT.,LA.-HURR. PROT. RIGOLETS CONTROL STR.&CLOSURE DAM DDM NO. 6.

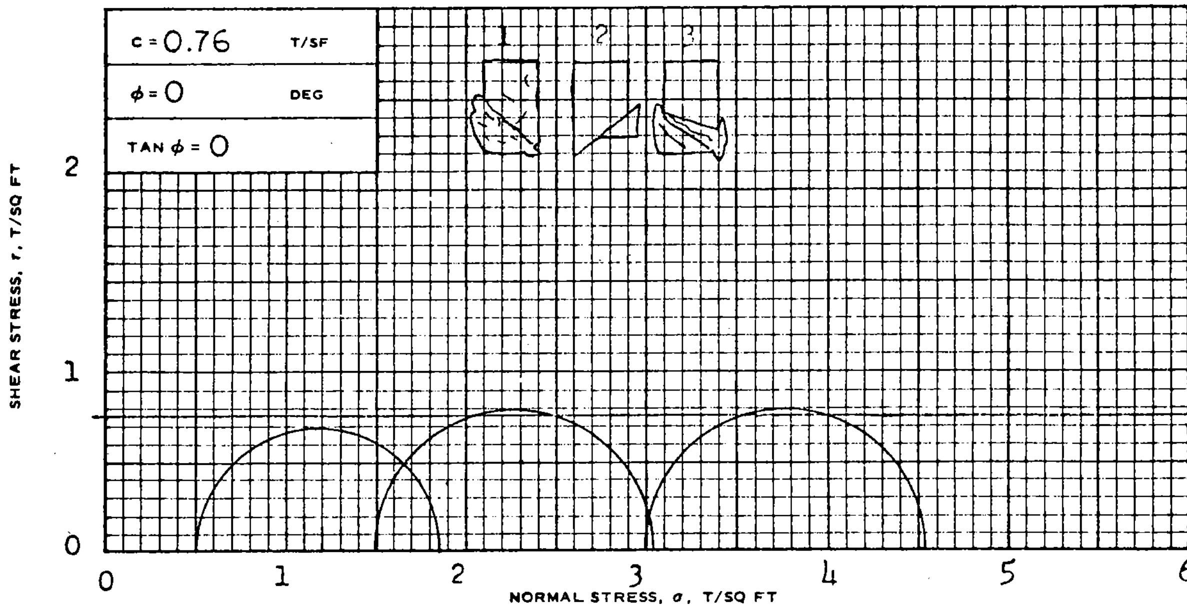
BORING NO. X-9-U SAMPLE NO. 15-D

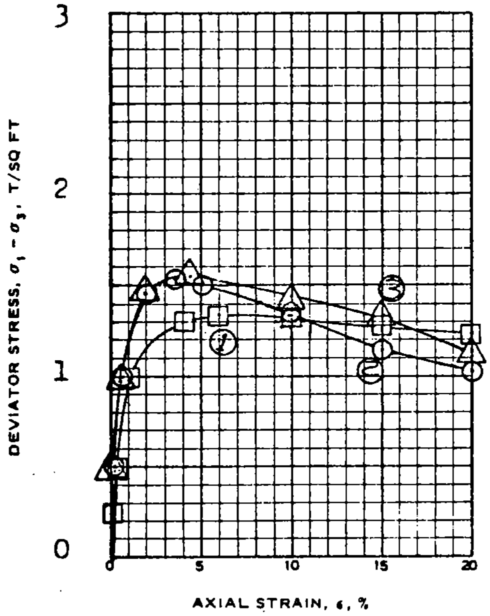
~~DEPTH~~/ELEV -88.4 MSL

LABORATORY USAEWES DATE 18 October 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)
REV JUNE 1970





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	36.7	37.1	37.7
	DRY DENSITY LB/CU FT	γ_{d_o}	83.5	83.5	82.4
	SATURATION, %	S_o	96.9	98.0	96.7
	VOID RATIO	e_o	1.03	1.03	1.06
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γ_{d_c}			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.34	1.52	1.52
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	28	14	27
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.41	1.40	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray

LL 69	PL 24	PI 45	Gs 2.72	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-9-U SAMPLE NO. 17-D

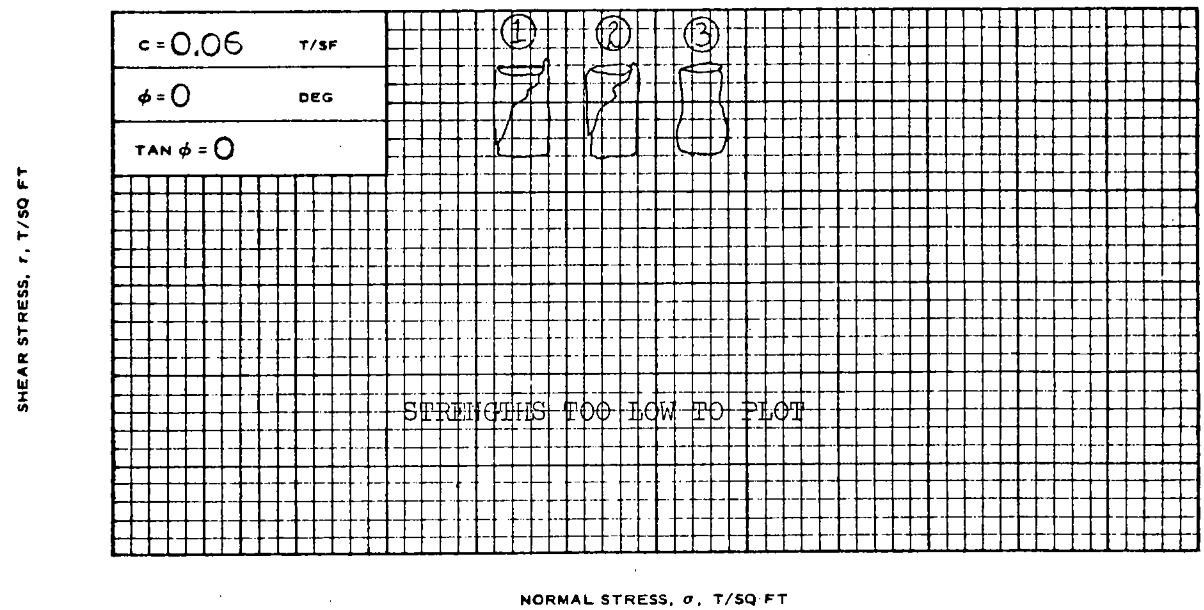
~~DEPTH~~/ELEV -96.2 MSL

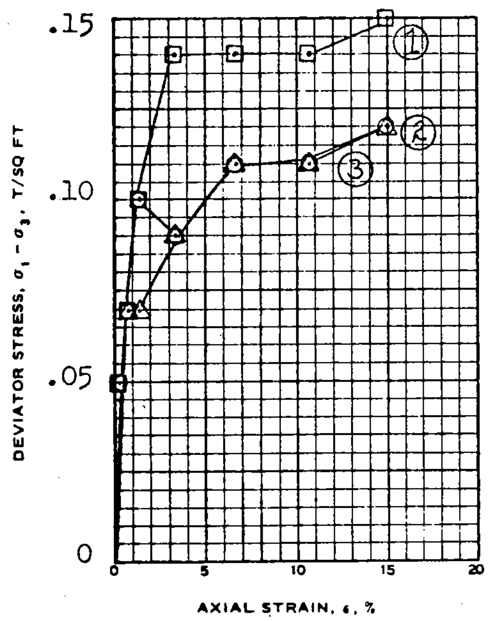
LABORATORY USAEWES DATE 22 October, 1971

FAM **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

Fig. A - 72





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	65.7	70.0	57.4
	DRY DENSITY LB/CU FT	γ_{d_o}	62.7	59.8	67.4
	SATURATION, %	S_o	100+	100+	100+
	VOID RATIO	e_o	1.66	1.79	1.47
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γ_{d_c}			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.15	0.12	0.12
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	125	125	125
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.37	1.38	1.38
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains large deposits of fine sand

LL 51	PL 15	PI 36	Gs 2.67	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK.PONT.,LA.& VIC.-HURR. PROT.(1971) RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO6

BORING NO. X10-U SAMPLE NO. 2-D

DEPTH/ELEV -36.1 MSL

LABORATORY USAEWES DATE 18 Oct. 1971

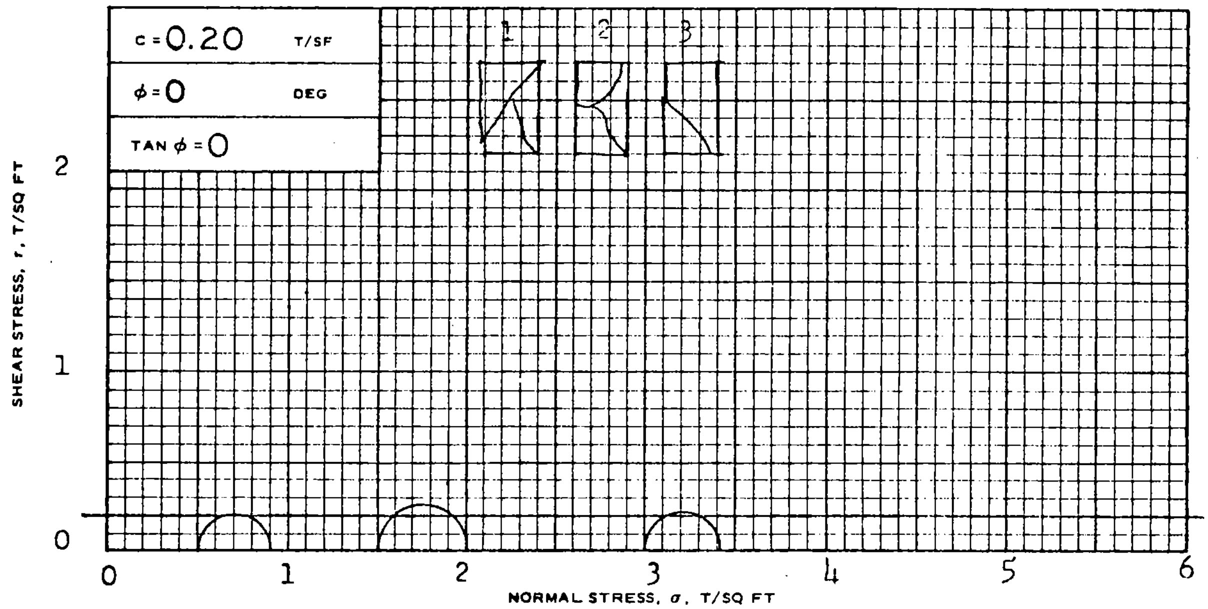
TES TRIAXIAL COMPRESSION TEST REPORT

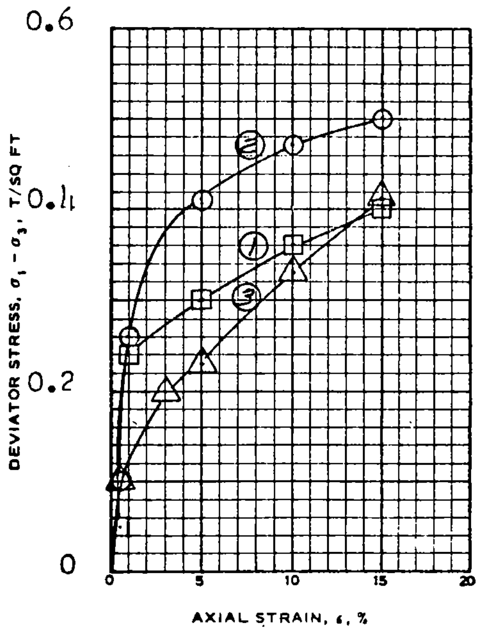
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

c = 0.20 T/SF

φ = 0 DEG

TAN φ = 0





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	42.8	40.8	40.6	
	DRY DENSITY LB/ CU FT	γd_o	78.7	80.9	80.6	
	SATURATION, %	S_o	100	100+	100	
	VOID RATIO	e_o	1.16	1.09	1.10	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.40	0.50	0.41	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	59	30	30	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.39	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains silt lenses

LL 50	PL 21	PI 29	Gs 2.71	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-10-U SAMPLE NO. 5-B

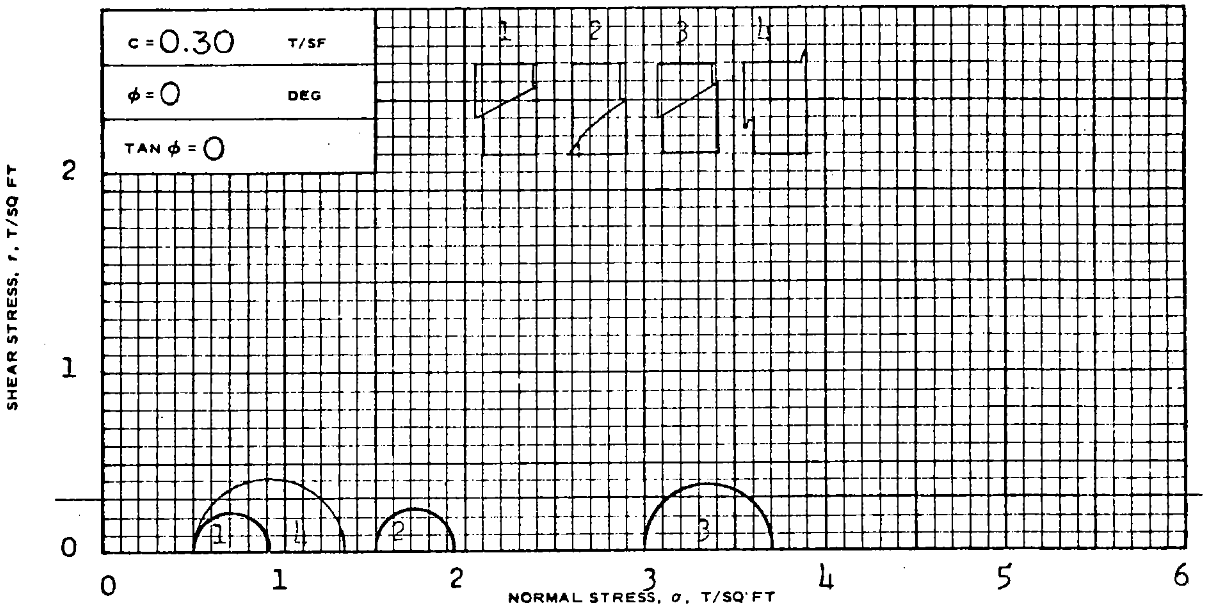
~~DEPTH~~/ELEV -45.7 MSL

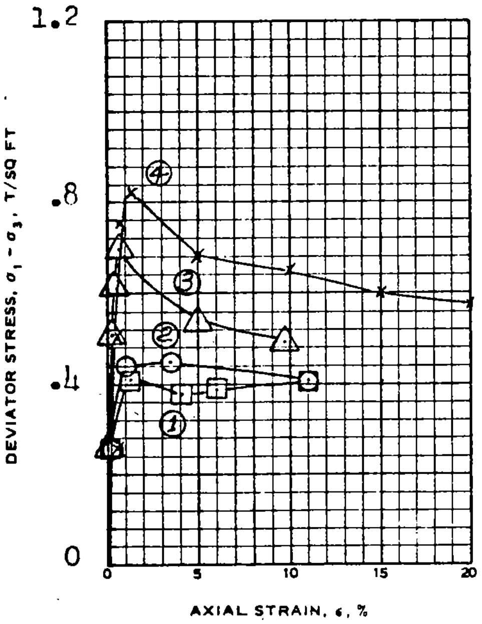
LABORATORY USAEWES DATE 20 October, 1971

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

c = 0.30	T/SF
ϕ = 0	DEG
TAN ϕ = 0	





			1	2	3	4
SPECIMEN NO.			1	2	3	4
INITIAL	WATER CONTENT, %	w_o	49.0	45.2	48.0	45.9
	DRY DENSITY LB/CU FT	γ_{d_o}	73.1	76.3	73.9	75.7
	SATURATION, %	S_o	100+	99.9	100+	99.8
	VOID RATIO	e_o	1.34	1.24	1.31	1.26
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	0.5
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.41	0.45	0.70	0.82
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	231	73	13	10
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray

LL 71	PL 20	PI 51	Gs 2.74	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

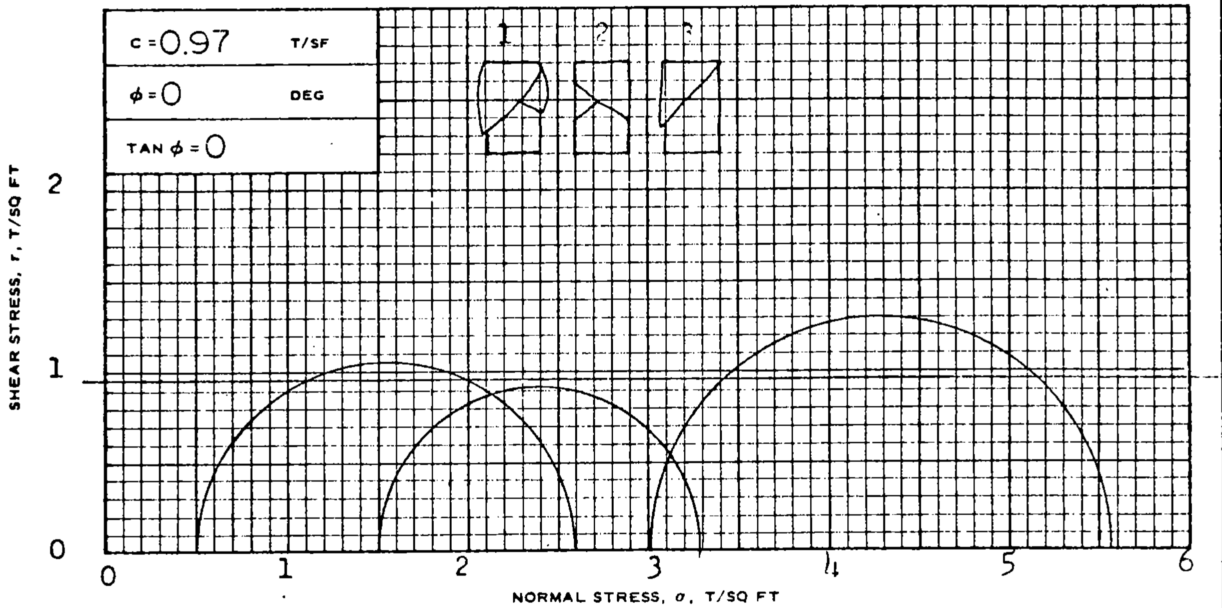
BORING NO. X-10-U — SAMPLE NO. 8-D

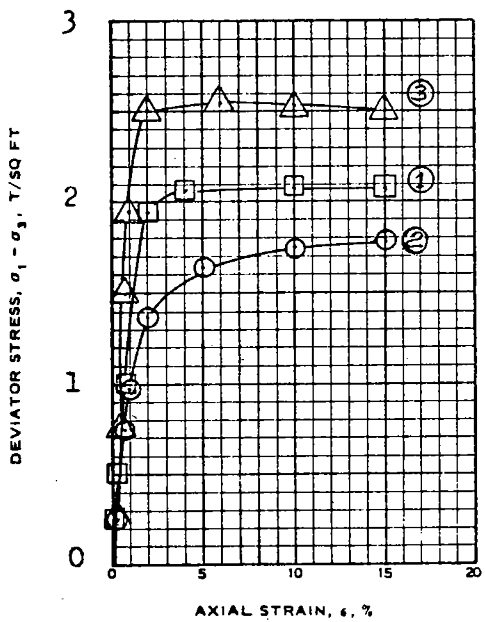
~~DEPTH~~/ELEV -60.0 MSL

LABORATORY USAEWES — DATE 1 November 1971

GDA — **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 — PREVIOUS EDITION IS OBSOLETE — TRANSLUCENT — *(EM 1110-2-1906)*





SPECIMEN NO.		1	2	3
INITIAL: WATER CONTENT, %	w_o	22.3	25.3	24.5
INITIAL: DRY DENSITY LB/CU FT	γd_o	102.3	98.2	98.9
INITIAL: SATURATION, %	S_o	93.4	95.8	94.4
INITIAL: VOID RATIO	e_o	0.642	0.710	0.698
BEFORE SHEAR: WATER CONTENT, %	w_c			
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c			
BEFORE SHEAR: SATURATION, %	S_c			
BEFORE SHEAR: VOID RATIO	e_c			
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	2.08	1.78	2.56
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	60	92	35
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.	D_o	1.40	1.41	1.40
INITIAL HEIGHT, IN.	H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),bluish gray

LL 56	PL 17	PI 39	Gs 2.69	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

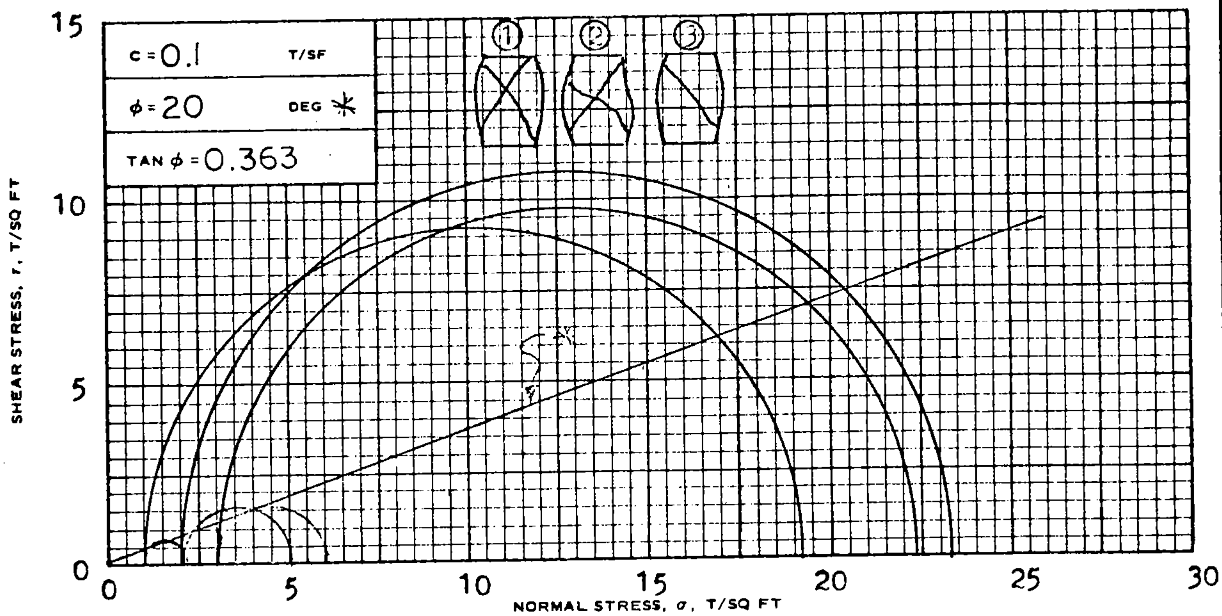
BORING NO. X-10-U SAMPLE NO. 10-D

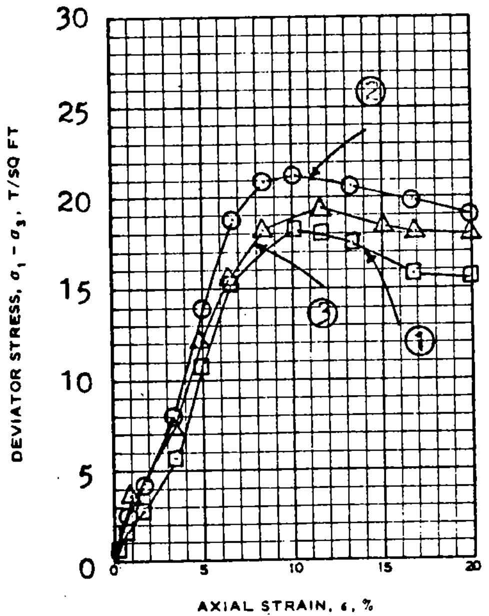
~~DEPTH~~/ELEV -68.0 MSL

LABORATORY USAEWES DATE 21, October, 1971

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.		1	2	3	
INITIAL: WATER CONTENT, %	w_o	26.9	26.5	28.7	
INITIAL: DRY DENSITY LB/CU FT	γd_o	96.4	97.0	94.4	
INITIAL: SATURATION, %	S_o	98.4	98.4	100	
INITIAL: VOID RATIO	e_o	0.730	0.719	0.766	
BEFORE SHEAR: WATER CONTENT, %	w_c	26.3	25.7	27.0	
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c	97.8	98.6	97.4	
BEFORE SHEAR: SATURATION, %	S_c	99.7	99.4	100+	
BEFORE SHEAR: VOID RATIO	e_c	0.705	0.690	0.712	
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o	3.96	3.96	3.96	
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	18.16	21.32	19.38	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	129	129	151	
$(\sigma_1 - \bar{\sigma}_3)$ at max pore pressure *		1.2	3.2	3.2	
INITIAL DIAMETER, IN.	D_o	1.38	1.38	1.40	
INITIAL HEIGHT, IN.	H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS SILTY SAND (SM), gray

LL –	PL –	PI –	Gs 2.67	TYPE OF SPECIMEN UNDIST.	TYPE OF TEST R

REMARKS: Portion of sample allowed to drain before trimming

PROJECT LK. PONT., LA. & VICINITY-HURR. PROT. (1971) RIGOLETS CONT. STR. & CLOSURE DAM; DDM#6

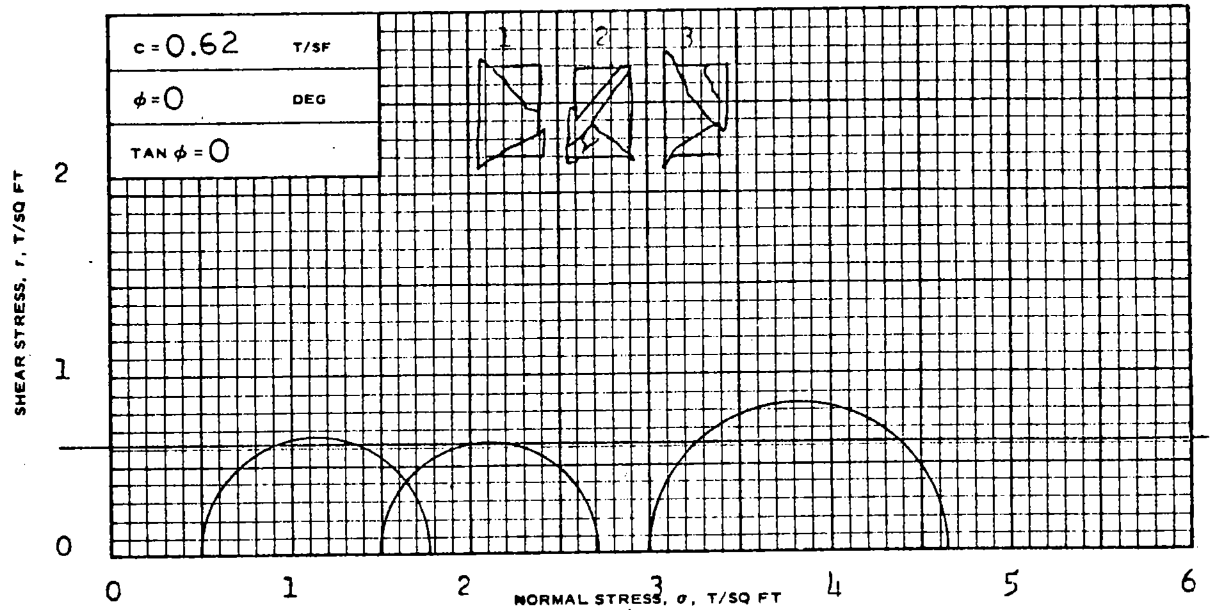
BORING NO. X-10U SAMPLE NO. 11-D

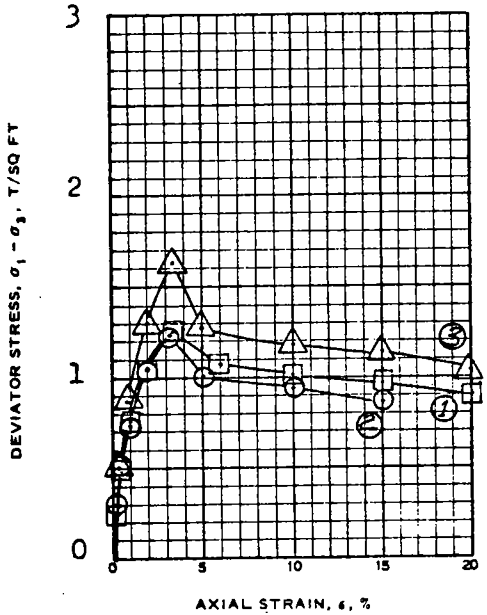
~~DEPTH~~/ELEV -72.0 MSL

LABORATORY USAEWES DATE 28 October, 1971

JAL **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	41.2	42.4	41.0
	DRY DENSITY LB/CU FT	γd_o	79.8	82.6	79.9
	SATURATION, %	S_o	98.7	100+	99.0
	VOID RATIO	e_o	1.14	1.06	1.13
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.27	1.21	1.62
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	13	31	17
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.41	1.40	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), dark gray, contains a few shell fragments

LL 73	PL 20	PI 53	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-10-U SAMPLE NO. 15-B

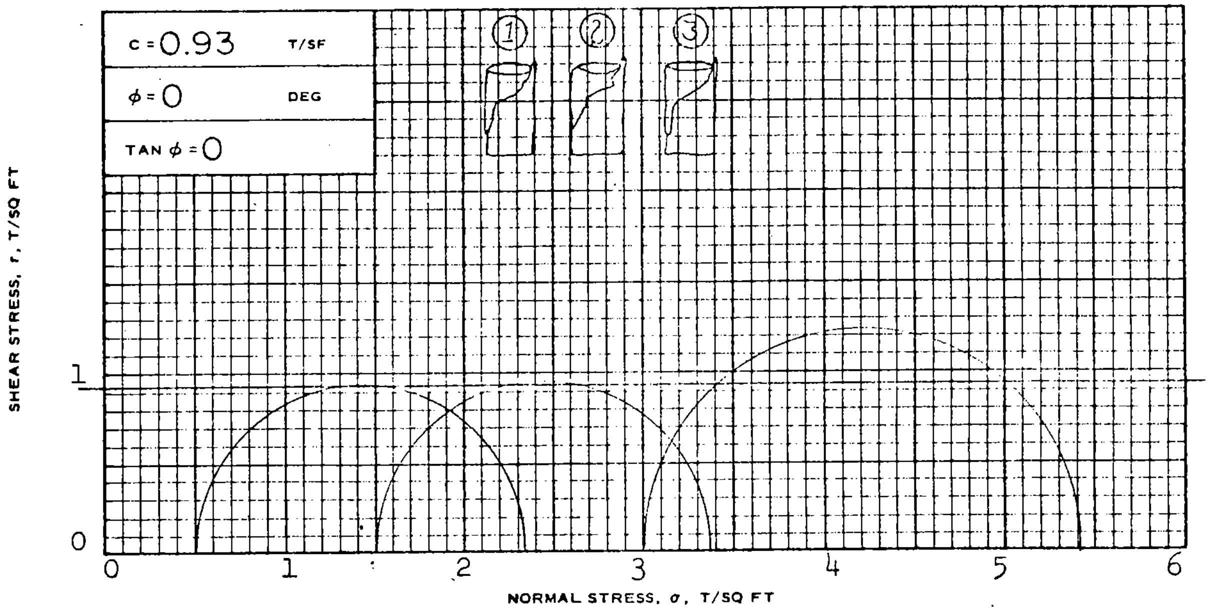
DEPTH/ELEV -85.8 MSL

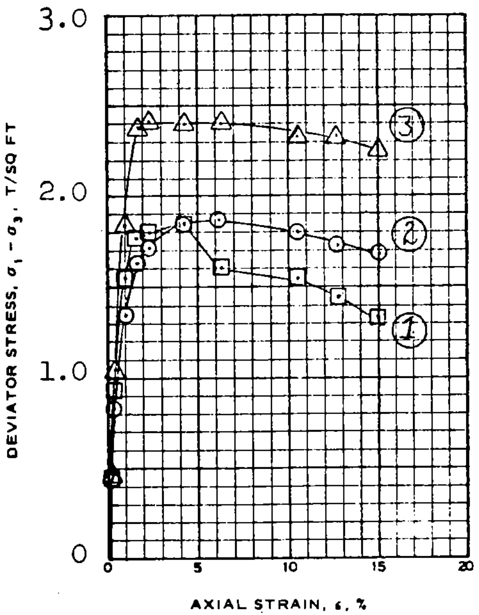
LABORATORY USAEWES DATE 21 October 1971

FAM TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

Fig. A - 78





			SPECIMEN NO.	1	2	3	
INITIAL	WATER CONTENT, %	w_o	34.7	34.4	33.6	
	DRY DENSITY LB/CU FT	γd_o	87.1	84.7	87.9	
	SATURATION, %	S_o	100+	93.8	98.8	
	VOID RATIO	e_o	0.935	0.990	0.918	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.84	1.87	2.41	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	58	85	31	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.40	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains numerous small concretions 1/8" to 1/4" in diameter,silt pockets and lenses;crumbly

LL 69	PL 19	PI 50	Gs 2.70	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK.PONT.,LA.& VIC.-HURR. PROT. (1971) RIGOLETS CONTROL STR.&CLOSURE DAM;DDM NO.6

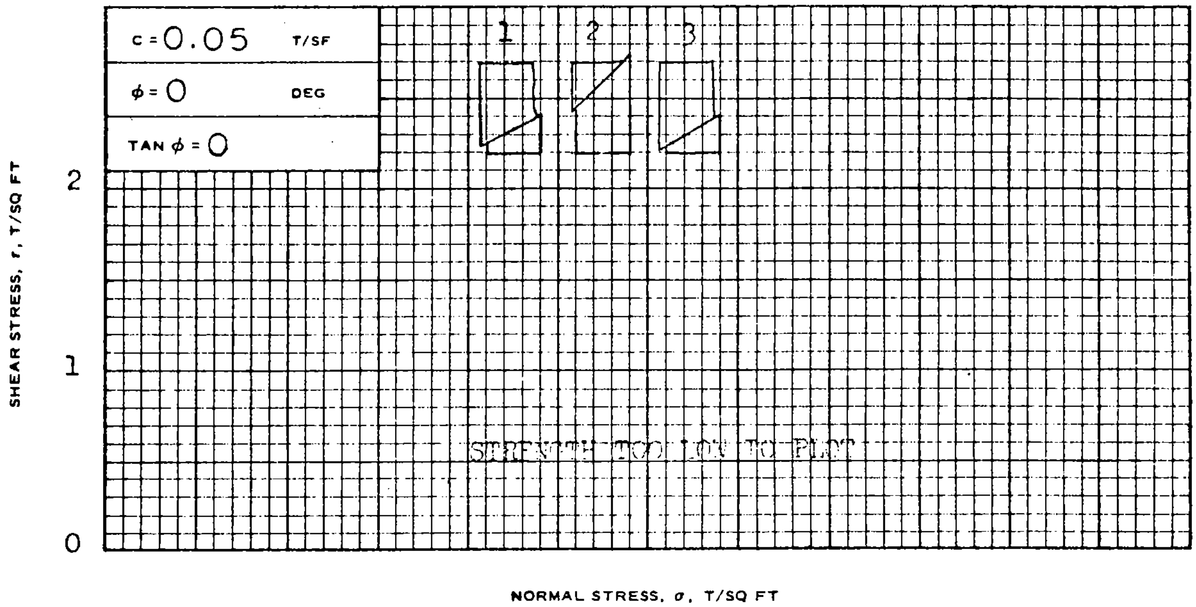
BORING NO. X-10-U SAMPLE NO. 17-C

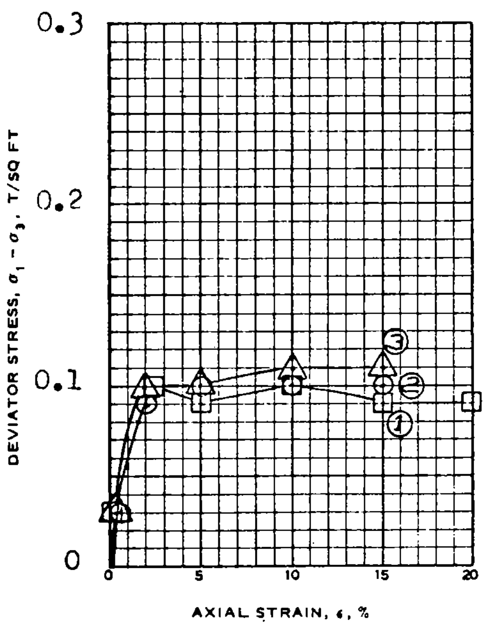
~~DEPTH~~/ELEV -94.9 MSL

LABORATORY USAEWES DATE 19 Oct. 1971

TES **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	119.5	119.9	118.8
	DRY DENSITY LB/CU FT	γd_o	39.6	38.8	39.8
	SATURATION, %	S_o	99.6	97.2	99.7
	VOID RATIO	e_o	3.19	3.28	3.17
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.10	0.10	0.11
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	14	76	138
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.38	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, fissured

LL 125	PL 27	PI 98	Gs 2.66	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

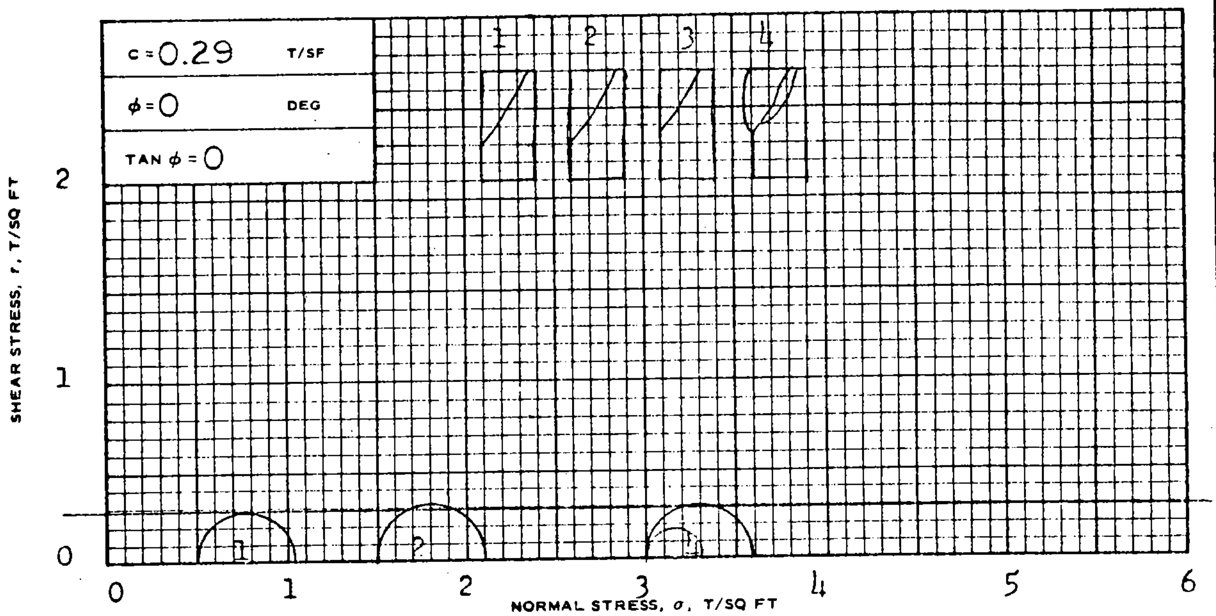
BORING NO. X-11-U SAMPLE NO. 2-D

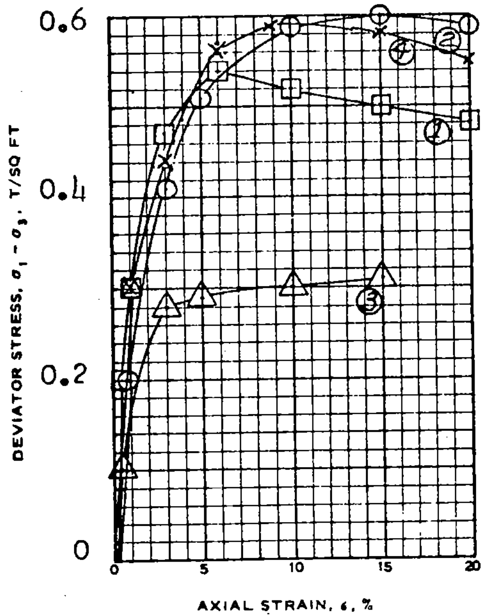
~~DEPTH~~/ELEV -38.8 MSL.

LABORATORY USAEWES DATE 2 November 1971

GDA **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	4
INITIAL	WATER CONTENT, %	w_o	44.4	42.7	47.5	47.2
	DRY DENSITY LB/CU FT	γd_o	76.1	76.6	72.4	73.6
	SATURATION, %	S_o	98.6	95.6	96.1	98.4
	VOID RATIO	e_o	1.22	1.21	1.34	1.30
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.54	0.60	0.31	0.59
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	23	72	61	18
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.41	1.41	1.40	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains sand lenses and a few shells up to 4" diameter

LL 67	PL 18	PI 49	Gs 2.71	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

Atterberg limits taken from mixture of clay and sand.

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

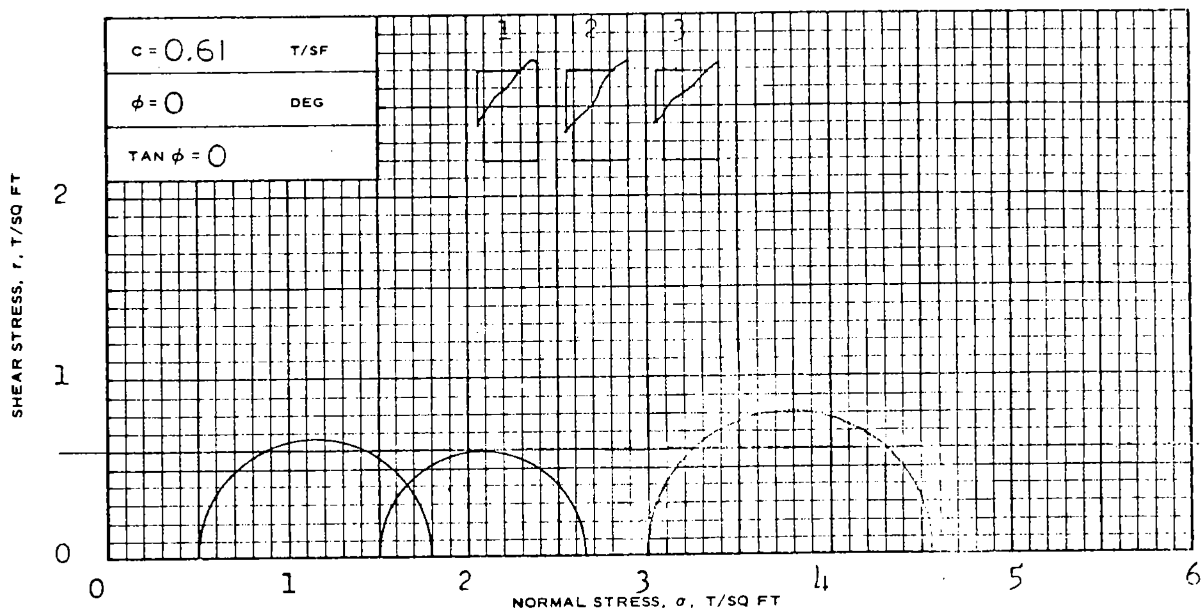
BORING NO. X-11-U SAMPLE NO. 8-A

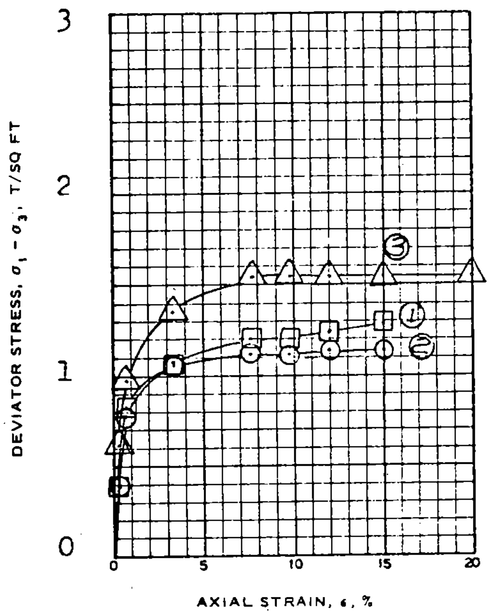
DEPTH/ELEV -60.3 MSL

LABORATORY USAEWES DATE 22 October, 1971

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.		1	2	3
INITIAL: WATER CONTENT, %	w_o	27.8	27.8	28.6
INITIAL: DRY DENSITY LB/CU FT	γd_o	96.5	96.2	95.1
INITIAL: SATURATION, %	S_o	99.5	98.8	99.0
INITIAL: VOID RATIO	e_o	0.760	0.765	0.786
BEFORE SHEAR: WATER CONTENT, %	w_c			
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c			
BEFORE SHEAR: SATURATION, %	S_c			
BEFORE SHEAR: VOID RATIO	e_c			
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	1.29	1.13	1.54
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	208	208	166
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.	D_o	1.39	1.39	1.39
INITIAL HEIGHT, IN.	H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray;contains a few tiny decayed roots; crumbly

LL 56	PL 17	PI 39	Gs 2.72	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

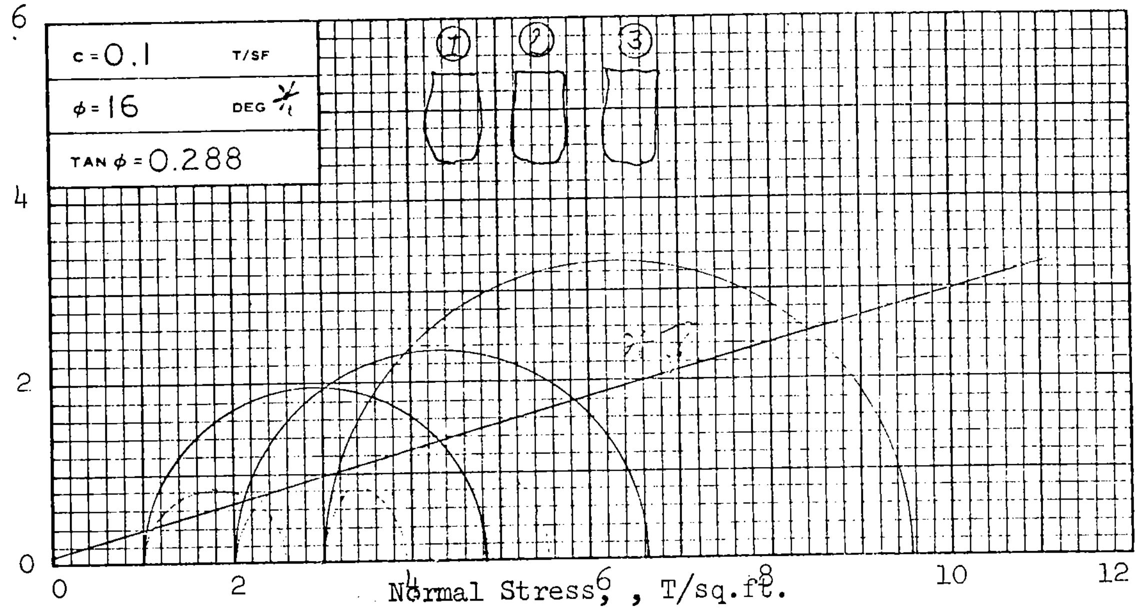
BORING NO. X-11-U SAMPLE NO. 9-D

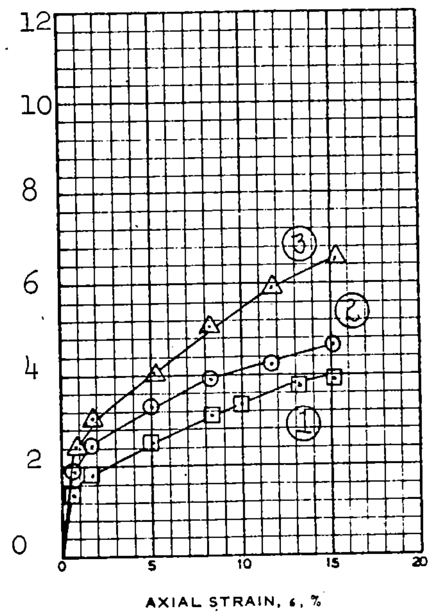
DEPTH/ELEV -66.9 MSL

LABORATORY USAEWES DATE 9 November 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	26.8	27.8	26.4	
	DRY DENSITY LB/CU FT	γd_o	96.8	95.4	97.0	
	SATURATION, %	S_o	98.0	98.3	97.2	
	VOID RATIO	e_o	0.736	0.712	0.731	
BEFORE SHEAR	WATER CONTENT, %	w_c	26.0	25.9	24.4	
	DRY DENSITY LB/CU FT	γd_c	98.4	98.1	100.2	
	SATURATION, %	S_c	98.9	97.8*	97.1*	
	VOID RATIO	e_c	0.707	0.712	0.676	
	FINAL BACK PRESSURE, T/SQ FT	u_o	3.96	3.96	3.96	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	3.85	4.65	6.57	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	213	213	214	
$(\sigma_1 - \sigma_3)$ at max pore pressure			1.6	1.9	2.5	
INITIAL DIAMETER, IN.		D_o	1.39	1.38	1.41	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS SILTY CLAY(CL-ML),gray; lightly cemented

LL 28	PL 21	PI 7	G 2.69	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS: * Pore pressure response indicated 100% saturation.

PROJECT LK. PONT.,LA.&VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR.&CLOSURE DAM;DDM #6

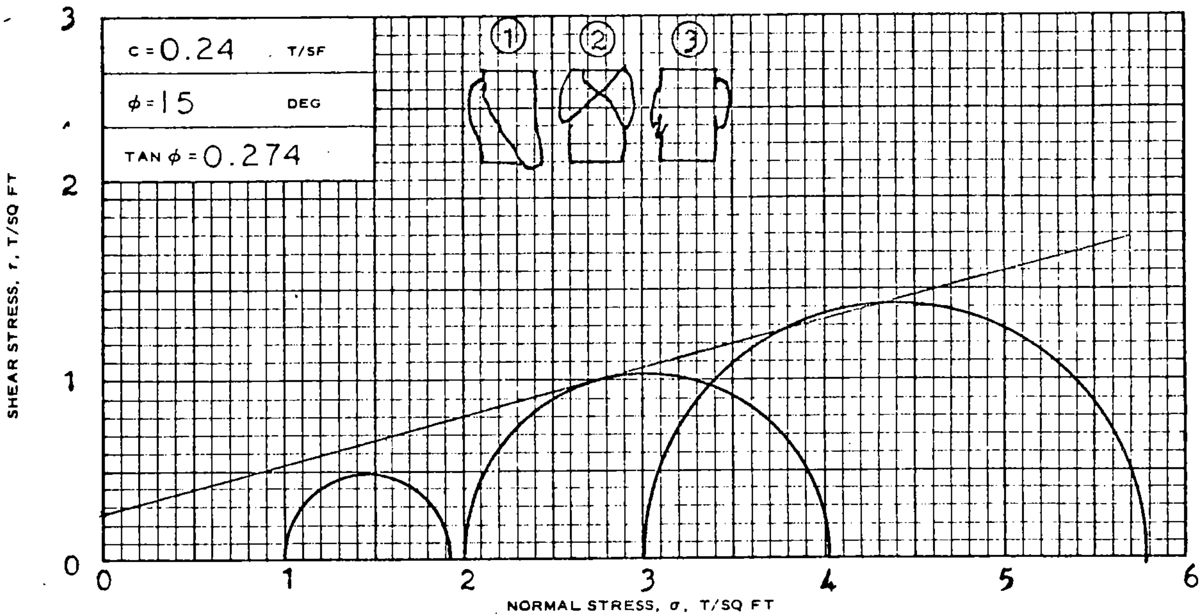
BORING NO. X-11-U — SAMPLE NO. 10-D

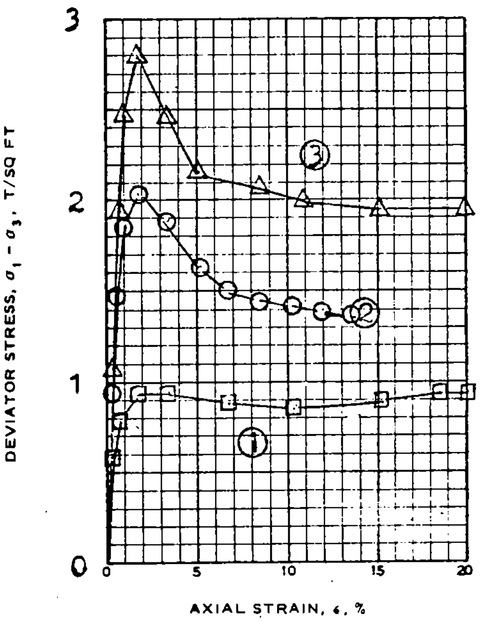
DEPTH/ELEV -70.8 MSL

LABORATORY USAEWES — DATE 17 Nov. 1971

JAL — TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 — PREVIOUS EDITION IS OBSOLETE — TRANSLUCENT — *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	51.7	52.8	51.1
	DRY DENSITY LB/CU FT	γd_o	70.3	70.0	71.1
	SATURATION, %	S_o	98.9	100+	99.5
	VOID RATIO	e_o	1.43	1.44	1.41
BEFORE SHEAR	WATER CONTENT, %	w_c	52.9	54.7	50.0
	DRY DENSITY LB/CU FT	γd_c	71.8	73.3	73.7
	SATURATION, %	S_c	100+	100+	100+
	VOID RATIO	e_c	1.38	1.33	1.32
	FINAL BACK PRESSURE, T/SQ FT	u_o	4.68	4.68	4.68
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.93	2.03	2.78
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	32	32	31
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), greenish gray, fissured

LL 95	PL 27	PI 68	Gs 2.74	TYPE OF SPECIMEN UNDIST.	TYPE OF TEST R

REMARKS:

PROJECT LK. PONT., LA, & VICINITY-HURR. PROT. (71)
RIGOLETS CONTROL STR. & CLOSURE DDM #6

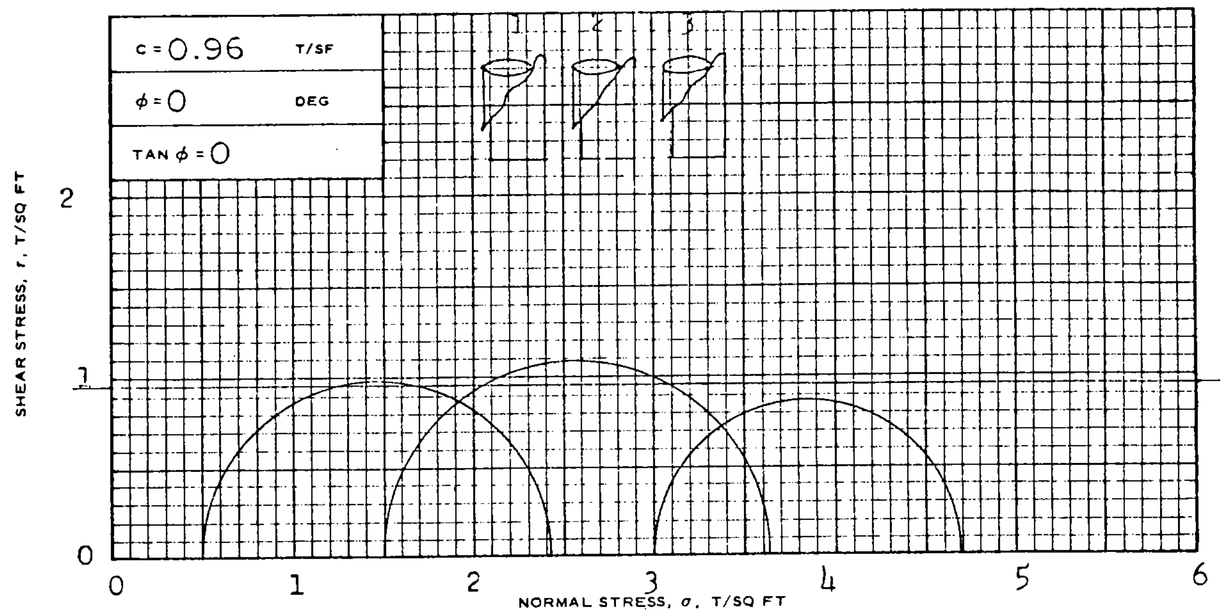
BORING NO. X-11U — SAMPLE NO. 12-B

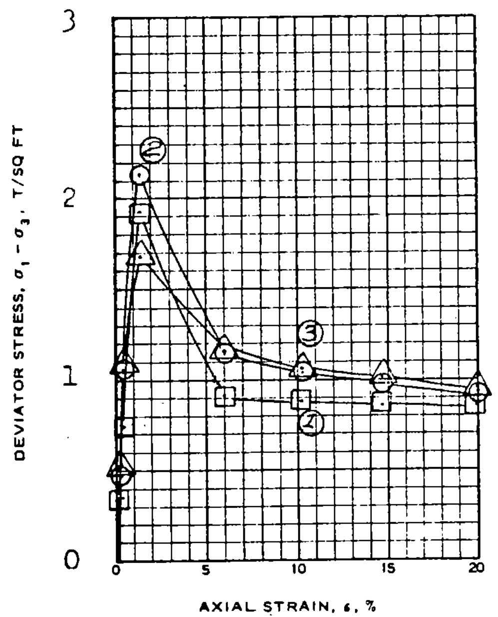
~~DEPTH~~/ELEV -77.2 MSL

LABORATORY USAEWES — DATE 11 Nov., 1971

JAL TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	55.4	55.7	55.3
	DRY DENSITY LB/CU FT	γd_o	68.0	67.9	68.0
	SATURATION, %	S_o	100+	100	100+
	VOID RATIO	e_o	1.51	1.51	1.51
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.92	2.13	1.69
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	18	18	18
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

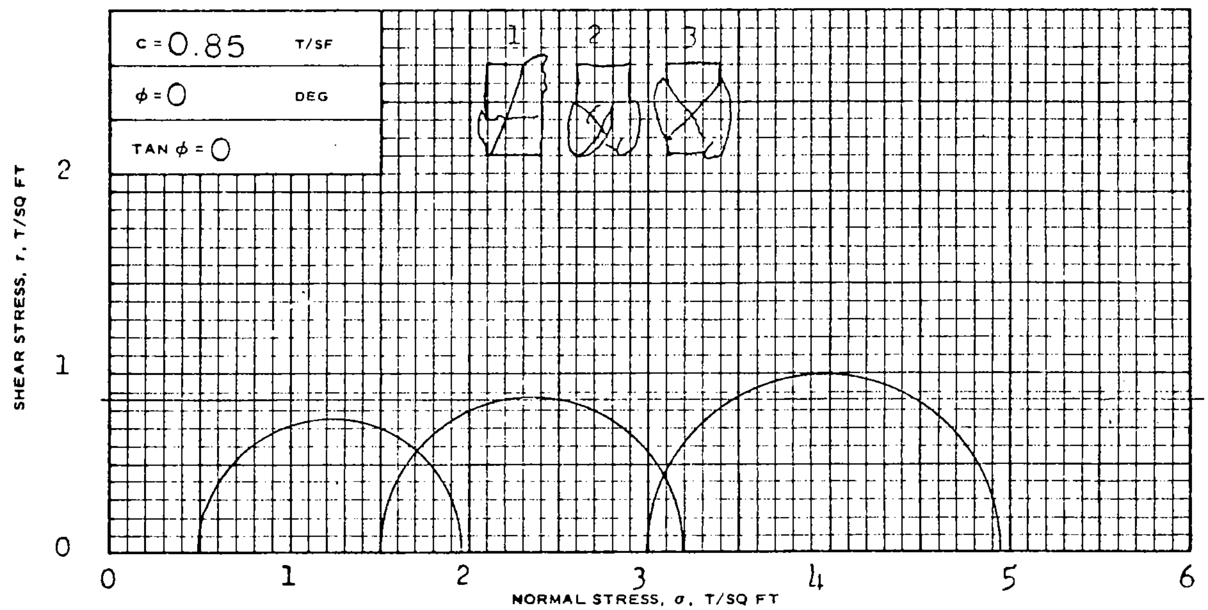
CONTROLLED- Strain TEST

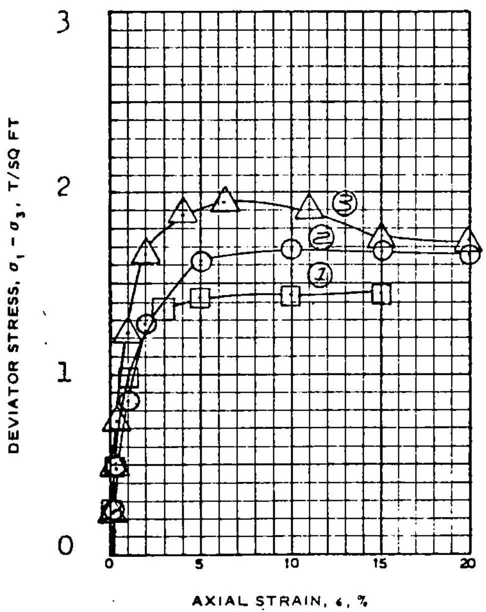
DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, contains scattered decayed rootlets

LL 101	PL 25	PI 76	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:	PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971
	BORING NO. X-11-U / SAMPLE NO. 13-B
	~~DEPTH~~/ELEV -81.3 MSL
	LABORATORY USAEWES / DATE 8 November 1971
	TES — TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





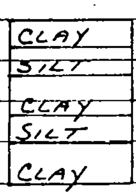
SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	33.8	34.4	33.7	
	DRY DENSITY LB/CU FT	γd_o	87.4	86.6	87.0	
	SATURATION, %	S_o	97.1	97.0	95.9	
	VOID RATIO	e_o	0.950	0.968	0.959	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.44	1.69	1.94	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	57	40	23	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.41	1.40	1.42	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, contains two layers of sandy silt

LL 61	PL 22	PI 39	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

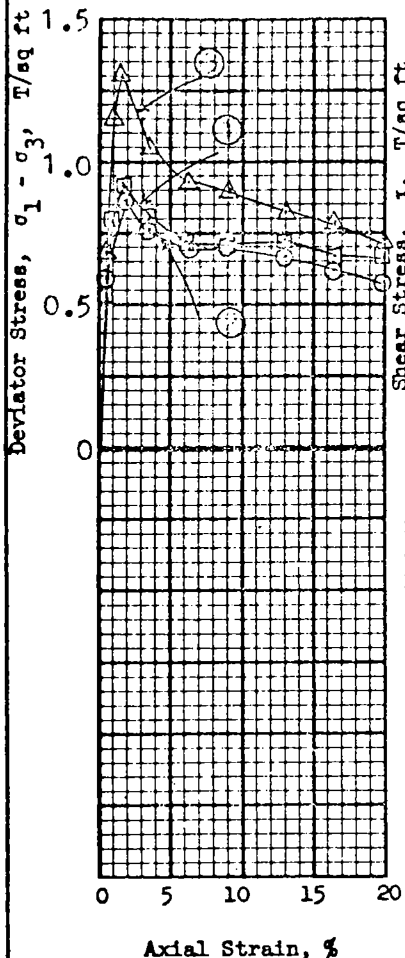


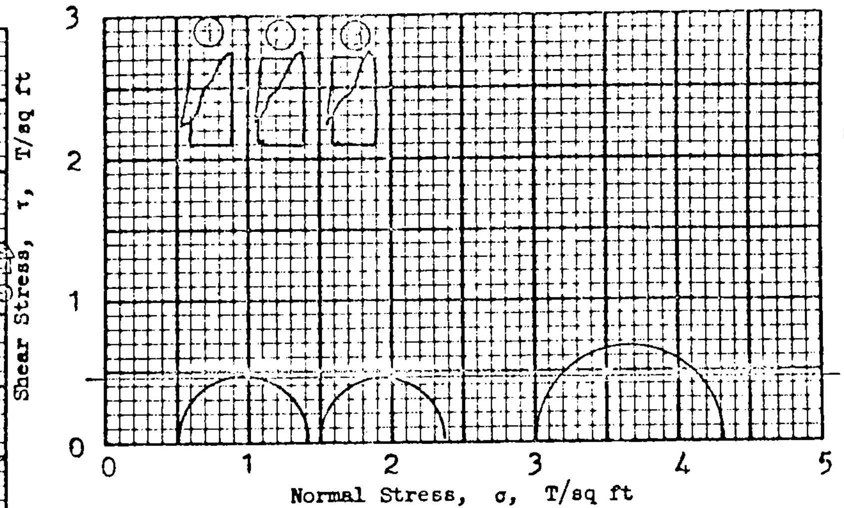
Typical specimen

PROJECT	LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971
BORING NO. X-11-U	SAMPLE NO. 16-C
DEPTH/ELEV	-94.0 MSL
LABORATORY USAEWES	DATE 22 October, 1971

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





Test No.			1	2	3	
Initial	Water content	w_o	51.4 %	53.0 %	47.3 %	%
	Void ratio	e_o	1.44	1.48	1.34	
	Saturation	S_o	99.2 %	99.6 %	98.1 %	%
	Dry density, lb/cu ft	γ_d	71.2	69.9	74.2	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.91	0.86	1.31	
Time to failure, min		t_f	17	17	14	
Rate of strain, percent/min			0.110	0.110	0.110	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

φ = 0°

tan φ = 0

c = 0.455 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, containing shell fragments, decayed *

LL 80 | PL 24 | PI 56 | G_s 2.78

Remarks *wood and strata of fine sand.

Project LK. PONT.,LA.& VIC.-HURR. PROT.

RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

Area DDM NO.6

Boring No. X-12U | Sample No. 8-D

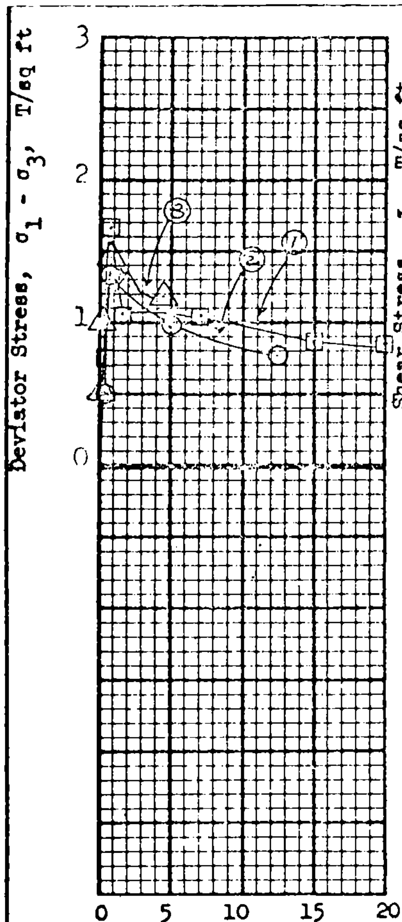
~~Depth~~ El -63.8 MSL | Date 27 September,1971

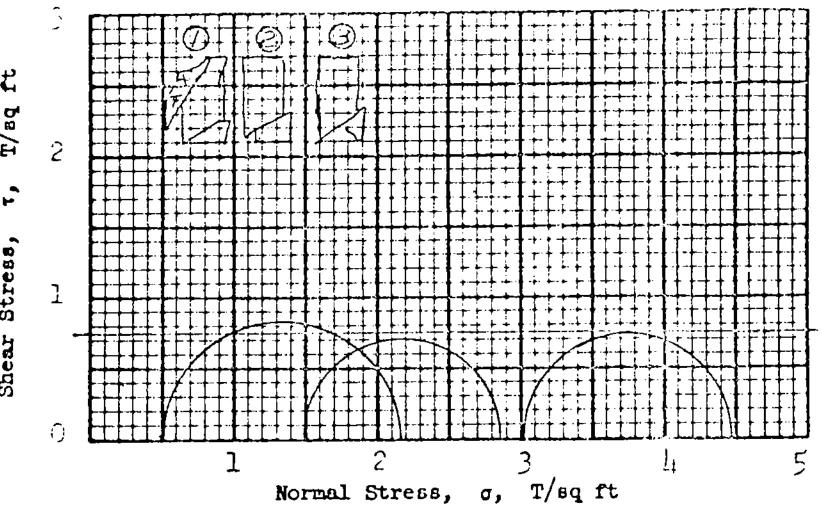
TFS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE *TRANSLUCENT*

USAEWES SOILS TEST SECTION

Fig. A - 87





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.74 T/sq ft

Method of saturation ______

[ ] Controlled stress

[X] Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	59.9 %	60.2 %	60.4 %	%
	Void ratio	e_o	1.65	1.66	1.67	
	Saturation	S_o	99.8 %	99.7 %	99.5 %	%
	Dry density, lb/cu ft	γ_d	64.8	64.5	64.3	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.66	1.34	1.47	
Time to failure, min		t_f	5	10	33	
Rate of strain, percent/min			0.174	0.083	0.018	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

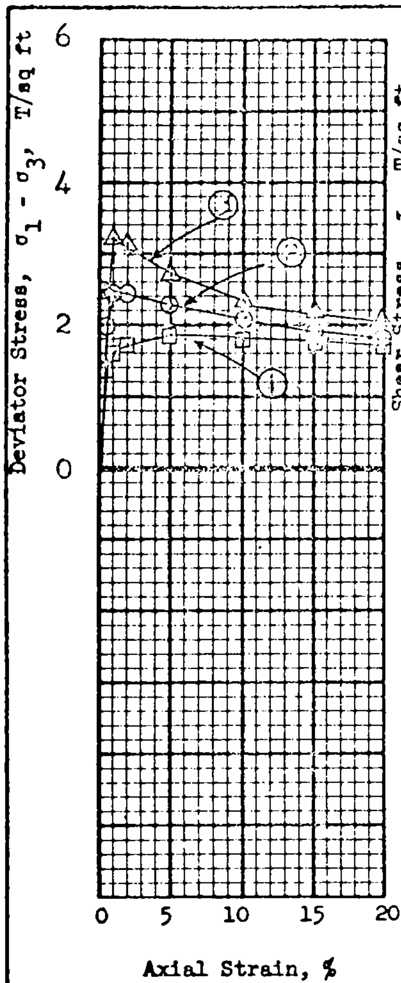
Type of test Q	Type of specimen UNDISTURBED			
Classification PLASTIC CLAY (CH), GRAY.				
LL 107	PL 28	PI 79		G_s 2.75

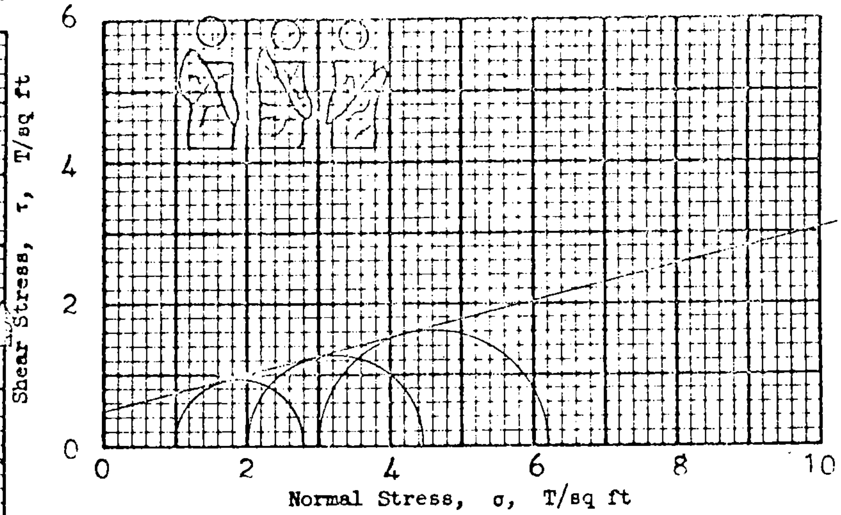
Remarks ______

Project LK. PONT., LA. HURR. PROT. RIGOLETS CONT. & STR. & CLOSURE DAM 1971	
Area	
Boring No. X-12-U	Sample No. 14-C
Depth El -86.9 MSL	Date 7 October 1971
FAM TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SCF





Shear Strength Parameters

ϕ = 14°

tan ϕ = 0.25

c = 0.50 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	36.0 %	39.1 %	39.1 %	%
	Void ratio	e_o	0.996	1.06	1.06	
	Saturation	S_o	96.1 %	98.1 %	98.1 %	%
	Dry density, lb/cu ft	γ_d	83.2	80.8	80.7	
Before Shear	Water content	w_c	37.7 %	39.6 %	39.3 %	%
	Void ratio	e_c	0.968	1.00	0.991	
	Saturation	S_c	100+ %	100+ %	100+ %	%
	Final back pressure, PSI	u_o	70	70	70	
	Dry Density Lbs/cu.ft.	γ_d	84.4	82.9	83.4	
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	1.0	2.0	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.81	2.45	3.21	
Time to failure, min		t_f	59	12	12	
Rate of strain, percent/min			0.085	0.085	0.086	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.41	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test R | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),light gray, slickensided

LL 63 | PL 23 | PI 40 | G_s 2.66

Remarks ________

Project LK.PONT.,LA.&VIC.-HURR. PROT.-(1971)

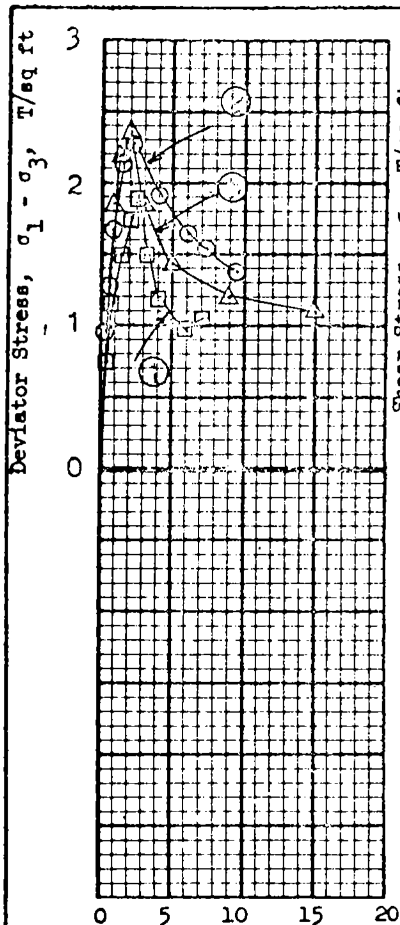
RIGOLETS CONTROL STRUCTURE&CLOSURE DAM;DDM#6

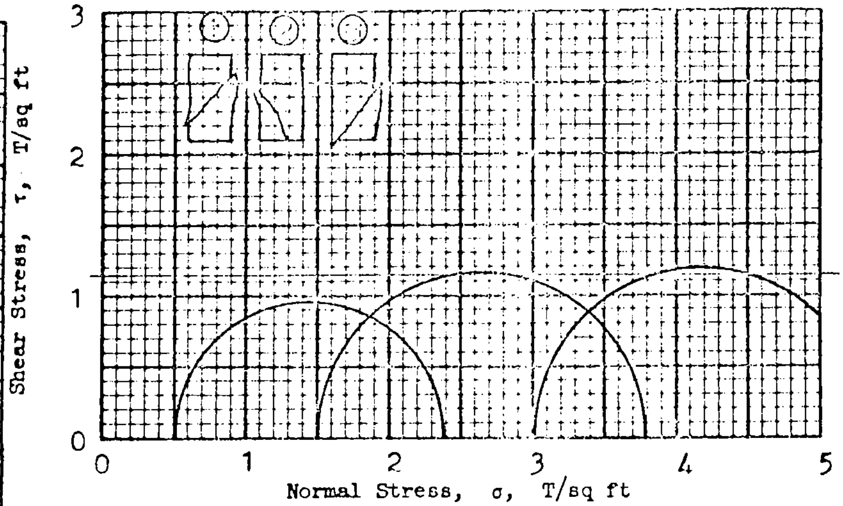
Area

Boring No. X-12U | Sample No. 17-D

~~Depth~~ El -99.8 MSL | Date 14 September,1971

PJR TRIAXIAL COMPRESSION TEST REPORT





Test No.			1	2	3	
Initial	Water content	w_o	49.9 %	49.7%	49.4%	%
	Void ratio	e_o	1.37	1.36	1.36	
	Saturation	S_o	97.6 %	97.9 %	97.3 %	%
	Dry density, lb/cu ft	γ_d	70.6	70.9	71.0	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.88	2.29	2.35	
Time to failure, min		t_f	21	26	24	
Rate of strain, percent/min			0.124	0.079	0.085	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.39	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.14 T/sq ft

Method of saturation _______

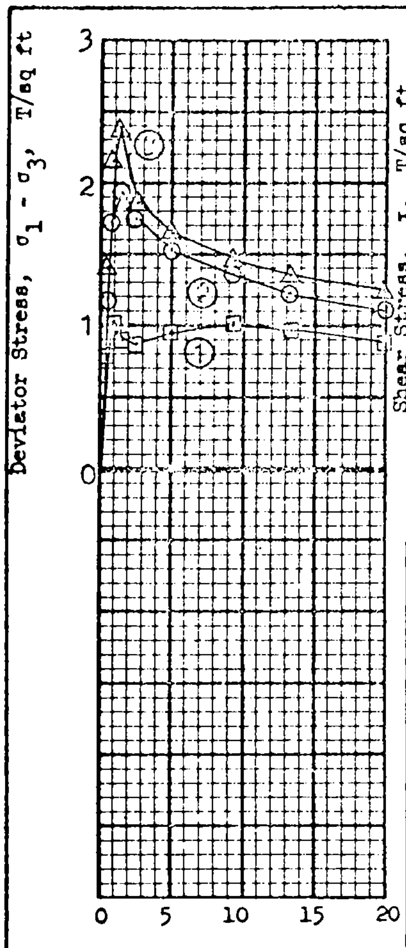
☐ Controlled stress

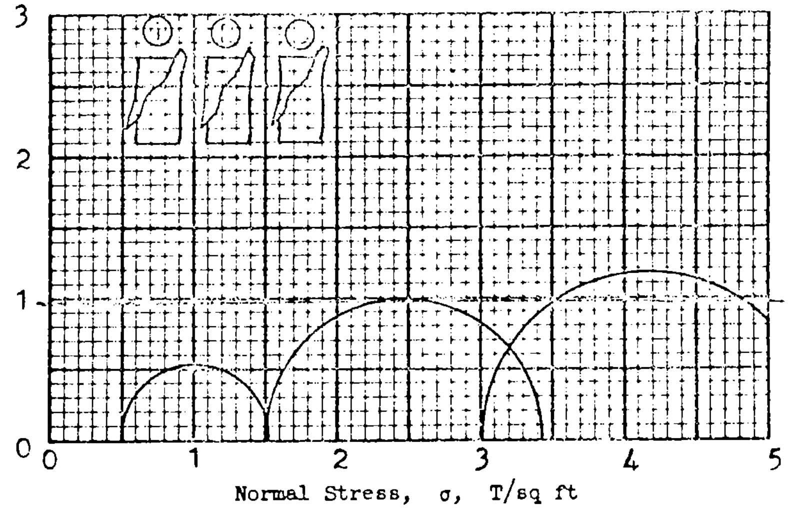
☒ Controlled strain

Type of test Q	Type of specimen UNDISTURBED		
Classification PLASTIC CLAY (CH), gray and brown with organic matter.			
LL 82	PL 26	PI 56	G_s 2.68

Remarks

Project LK. PONT., LA. & VIC-HURRICANE PROTECTION	
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM	
Area DDM NO. 6	
Boring No. X-12U	Sample No. 21-C
~~Depth~~ El -114.9 MSL	Date 28 September, 1971
GDA TRIAXIAL COMPRESSION TEST REPORT	





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.97 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	48.4 %	49.1 %	48.6 %	%
	Void ratio	e_o	1.38	1.38	1.37	
	Saturation	S_o	96.8 %	98.2 %	97.9 %	%
	Dry density, lb/cu ft	γ_d	72.5	72.4	72.8	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.01	1.94	2.37	
Time to failure, min		t_f	10	16	14	
Rate of strain, percent/min			0.094	0.094	0.094	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, containing fine sand pockets & lense

LL 66 | PL 22 | PI 44 | | G_s 2.76

Remarks Insufficient material for check test.

Project LK. PONT., LA. & VIC.-HURR. PROT.

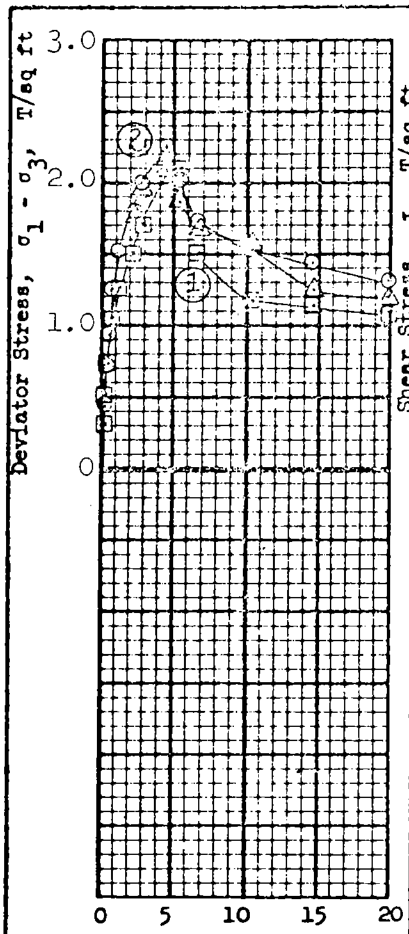
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM

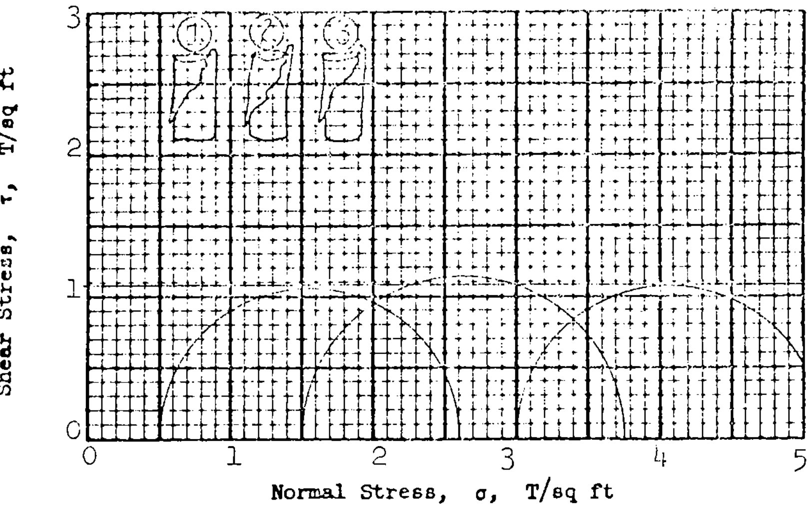
Area DDM NO.6

Boring No. X-12U | Sample No. 29-D

~~Depth~~ El -147.8 MSL | Date 29 September, 1971

TES TRIAXIAL COMPRESSION TEST REPORT





Sheer Strength Parameters

φ = 0°

tan φ = 0

c = 1.08 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	40.9 %	40.8 %	41.6 %	%
Initial	Void ratio	e_o	1.08	1.09	1.13	
Initial	Saturation	S_o	100+ %	100+ %	99.8 %	%
Initial	Dry density, lb/cu ft	γ_d	81.3	81.0	79.4	
Before Shear	Water content	w_c	%	%	%	%
Before Shear	Void ratio	e_c				
Before Shear	Saturation	S_c	%	%	%	%
Before Shear	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
Final	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.10	2.26	2.16	
Time to failure, min		t_f	52	52	52	
Rate of strain, percent/min			0.087	0.087	0.087	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.41	1.42	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY (CH), gray, containing scattered small concretions.

LL 72 | PL 24 | PI 48 | | G_s 2.71

Remarks ______

Project LK. PONT.,LA.&VIC.-HURR. PROTECTION

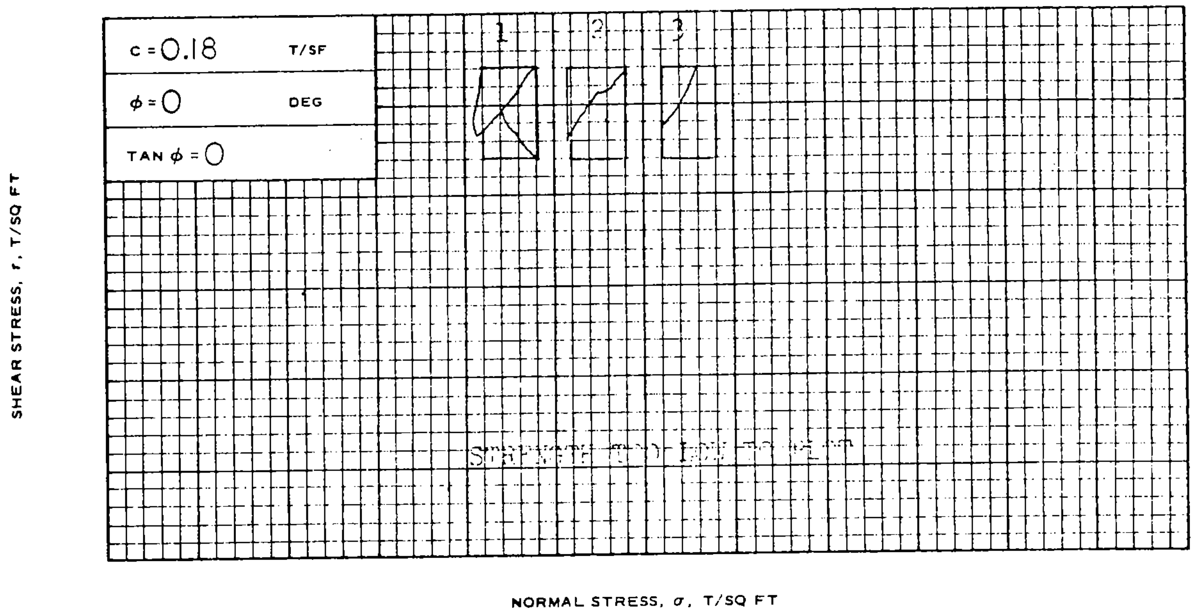
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM;

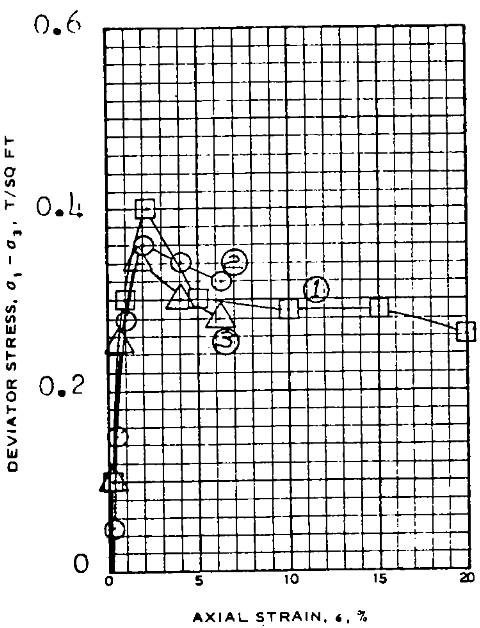
Area DDM NO. 6 (1971)

Boring No. X-12-U | Sample No. 34-B

Depth El -166.1 MSL | Date 29 Sept. 1971

TES TRIAXIAL COMPRESSION TEST REPORT





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	81.9	80.3	79.9
	DRY DENSITY LB/CU FT	γd_o	52.1	52.8	53.0
	SATURATION, %	S_o	99.3	99.2	99.1
	VOID RATIO	e_o	2.21	2.17	2.16
BEFORE SHEAR	WATER CONTENT, %	w_c			
	DRY DENSITY LB/CU FT	γd_c			
	SATURATION, %	S_c			
	VOID RATIO	e_c			
	FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.40	0.36	0.34
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	9	16	25
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains a few sand lenses

LL 103 | PL 25 | PI 78 | Gs 2.68

TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. -HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

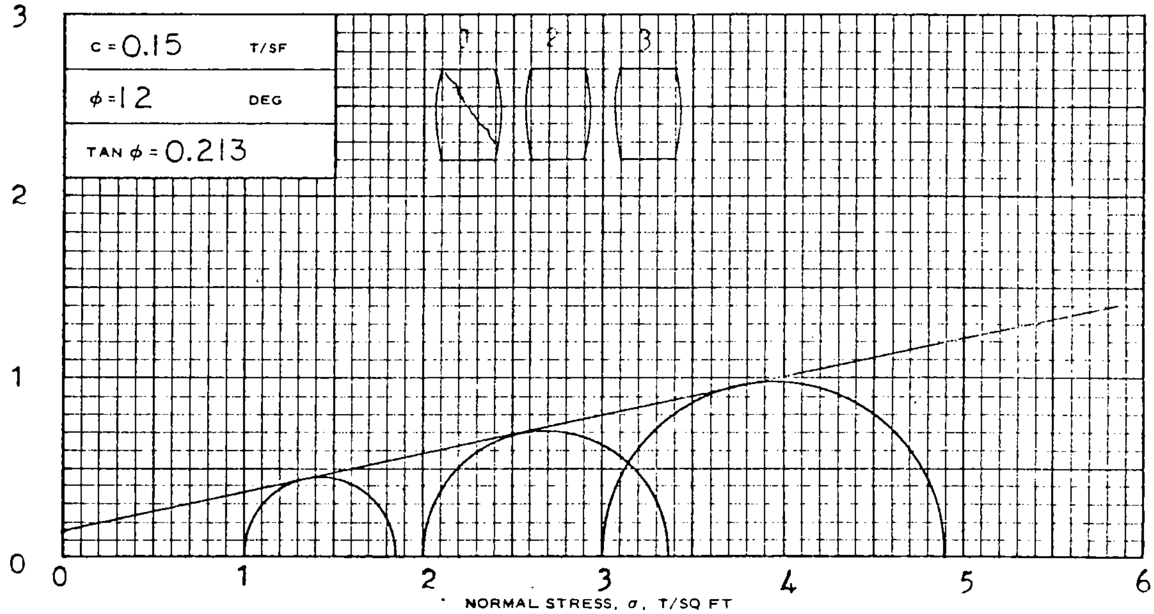
BORING NO. X-13-U | SAMPLE NO. 3-D

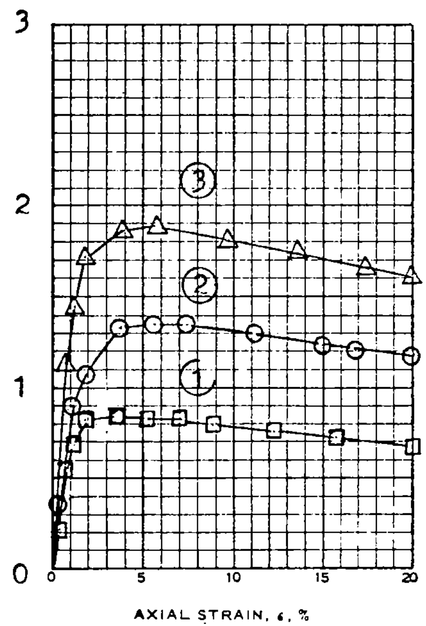
DEPTH/ELEV -35.7 MSL

LABORATORY USAEWES | DATE 1 november 1971

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	81.1	80.6	78.4
	DRY DENSITY LB/CU FT	γd_o	52.4	52.6	53.8
	SATURATION, %	S_o	98.8	98.6	99.1
	VOID RATIO	e_o	2.22	2.21	2.14
BEFORE SHEAR	WATER CONTENT, %	w_c	64.4	60.0	52.1
	DRY DENSITY LB/CU FT	γd_c	61.6	69.2	75.2
	SATURATION, %	S_c	100+	100+	100+
	VOID RATIO	e_c	1.74	1.44	1.24
	FINAL BACK PRESSURE, T/SQ FT	u_o	4.68	4.68	4.68
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.84	1.36	1.88
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	70	112	116
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),dark gray, contains fine sand lenses and fine sand seam

LL 114	PL 25	PI 89	Gs 2.70	TYPE OF SPECIMEN UNDIST.	TYPE OF TEST R

REMARKS:

PROJECT LK.PONT.,LA.& VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR.&CLOSURE DAM;DDM#6

BORING NO. X-13U SAMPLE NO. 4-C

~~DEPTH~~/ELEV - 38.9 MSL

LABORATORY USAEWES DATE 12 Nov.,1971

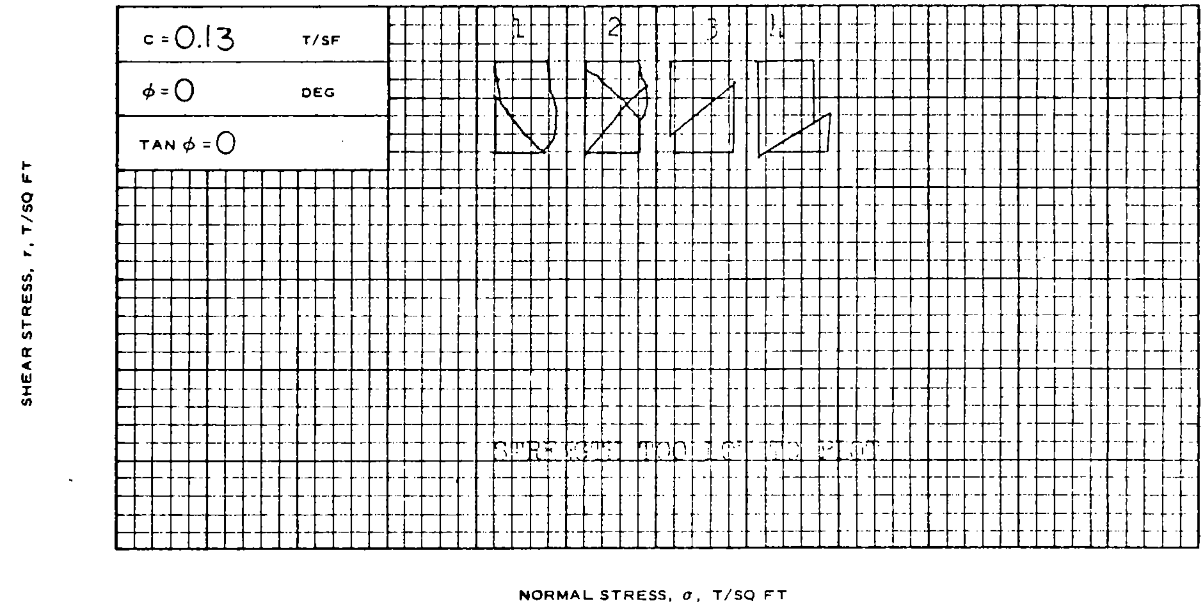
JAL TRIAXIAL COMPRESSION TEST REPORT

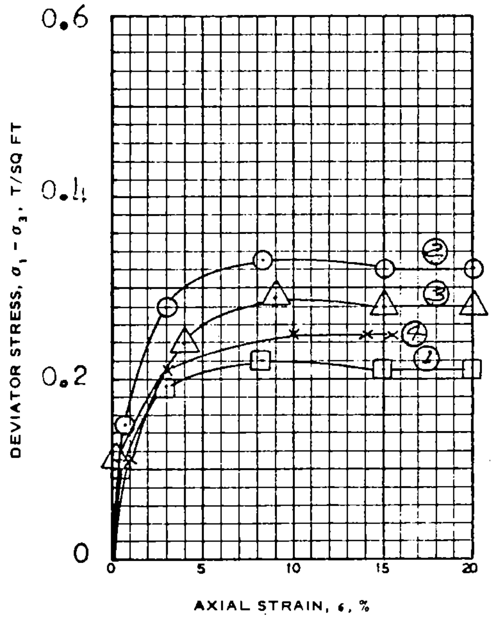
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

c = 0.13 T/SF

ϕ = 0 DEG

TAN ϕ = 0





SPECIMEN NO.			1	2	3	4
INITIAL	WATER CONTENT, %	w_o	59.2	53.3	55.2	69.6
	DRY DENSITY LB/CU FT	γ_{d_o}	63.7	67.0	66.8	58.8
	SATURATION, %	S_o	97.8	96.2	99.2	100+
	VOID RATIO	e_o	1.61	1.48	1.48	1.82
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	1.5
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.22	0.33	0.29	0.25
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	25	29	26	33
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	1.38
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains pockets of sand

LL 74	PL 20	PI 50	Gs 2.66	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA. - HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

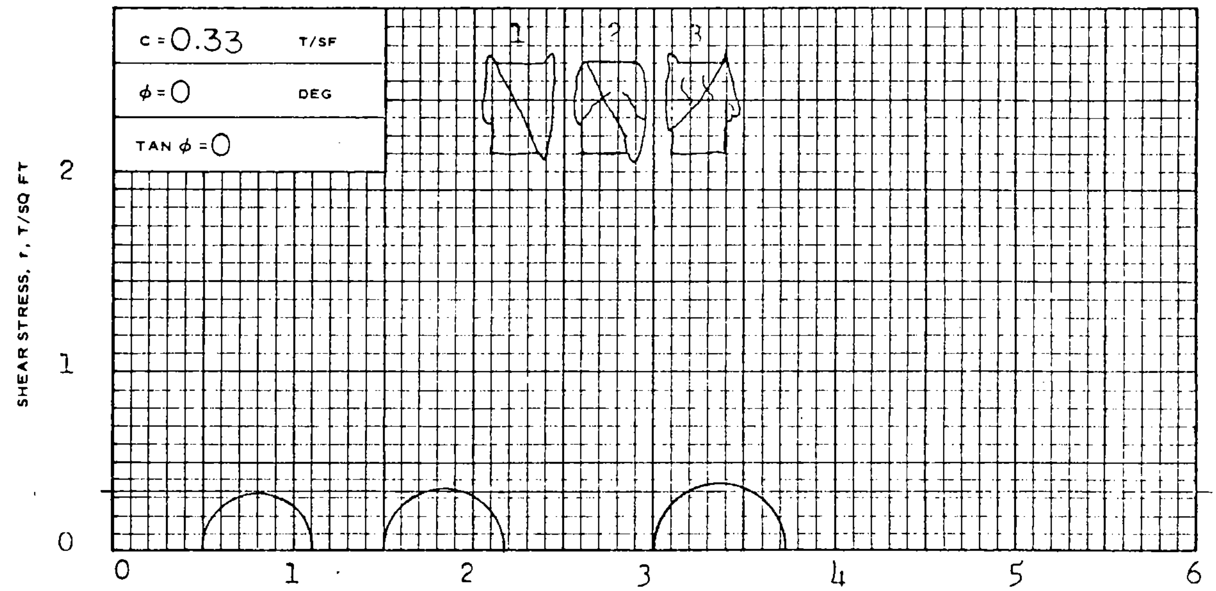
BORING NO. X-13-U | SAMPLE NO. 7-A

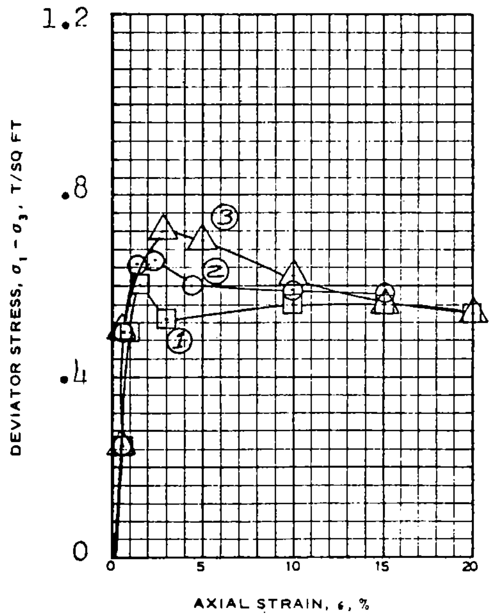
~~DEPTH~~/ELEV -49.1 MSL

LABORATORY USAEWES | DATE 3 November 1971

GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	54.2	48.2	48.2	
	DRY DENSITY LB/CU FT	γd_o	69.0	73.8	73.7	
	SATURATION, %	S_o	100+	100+	100+	
	VOID RATIO	e_o	1.47	1.31	1.31	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.60	0.66	0.72	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	12	22	34	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAH(CH),gray, contains fine sand

LL 79	PL 21	PI 58	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

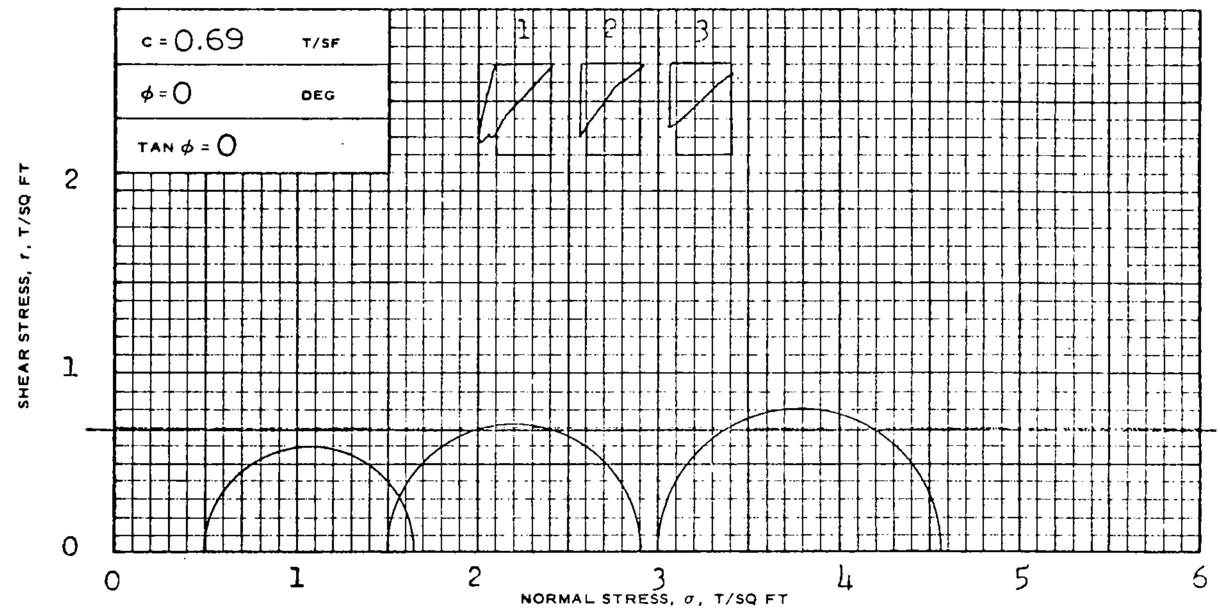
BORING NO. X-13-U SAMPLE NO. 10-C

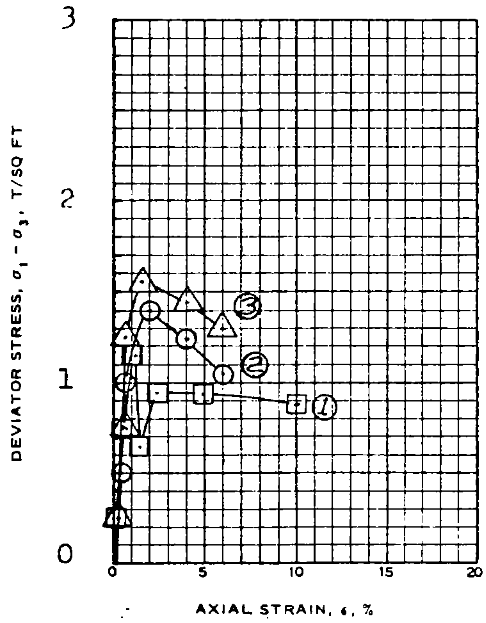
~~DEPTH~~/ELEV -62.8 MSL

LABORATORY USAEWES DATE 28 October 1971

FAM TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.		1	2	3	
INITIAL: WATER CONTENT, %	w_o	47.6	50.8	48.7	
INITIAL: DRY DENSITY LB/CU FT	γd_o	73.2	70.9	72.3	
INITIAL: SATURATION, %	S_o	97.3	98.7	98.1	
INITIAL: VOID RATIO	e_o	1.34	1.41	1.36	
BEFORE SHEAR: WATER CONTENT, %	w_c				
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c				
BEFORE SHEAR: SATURATION, %	S_c				
BEFORE SHEAR: VOID RATIO	e_c				
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	1.15	1.40	1.56	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	7	16	22	
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.	D_o	1.40	1.40	1.40	
INITIAL HEIGHT, IN.	H_o	3.00	3.00	3.00	

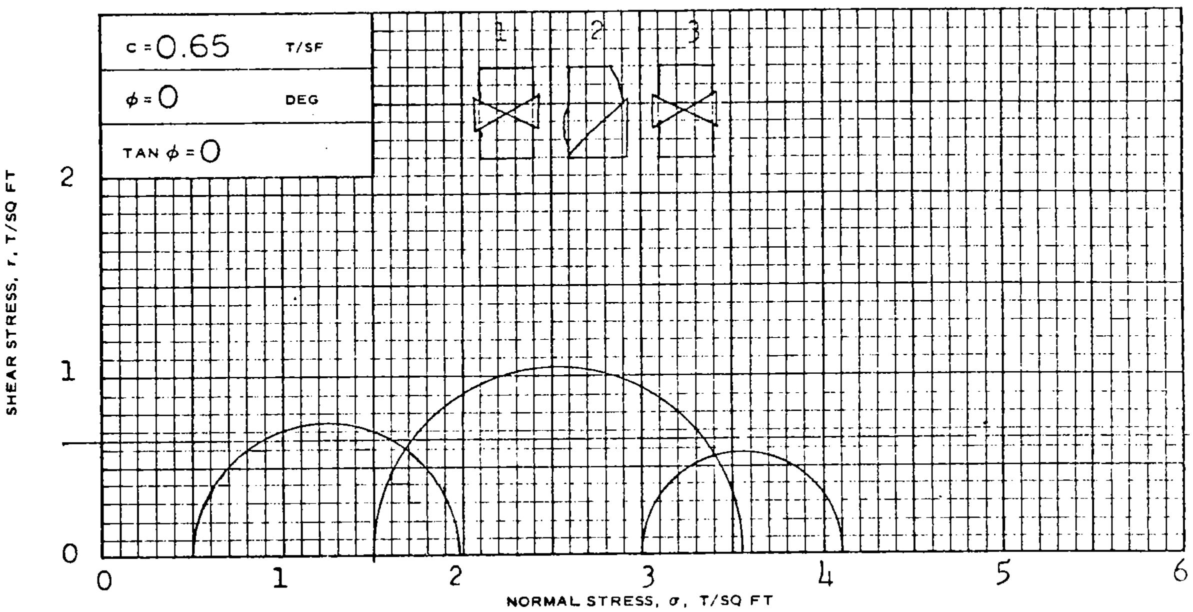
CONTROLLED- Strain TEST

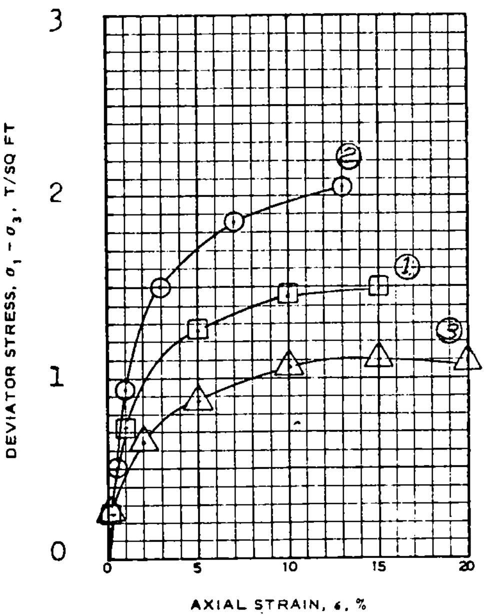
DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray, contains small amounts of organic matter

LL 87	PL 24	PI 63	Gs 2.74	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:	PROJECT LK. PONT., LA. -HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971
	BORING NO. X-13-U — SAMPLE NO. 14-C
	~~DEPTH~~-ELEV -78.9 MSL
	LABORATORY USAEWES — DATE 2 November 1971
	JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.		1	2	3
INITIAL: WATER CONTENT, %	w_o	31.8	30.4	31.8
INITIAL: DRY DENSITY LB/CU FT	γd_o	90.2	90.9	90.6
INITIAL: SATURATION, %	S_o	98.0	95.3	99.0
INITIAL: VOID RATIO	e_o	0.883	0.868	0.874
BEFORE SHEAR: WATER CONTENT, %	w_c			
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c			
BEFORE SHEAR: SATURATION, %	S_c			
BEFORE SHEAR: VOID RATIO	e_c			
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.5	1.5	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	1.49	2.05	1.10
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	75	42	57
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.	D_o	1.40	1.40	1.40
INITIAL HEIGHT, IN.	H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS LEAN CLAY(CL),gray, with sand, and contains seams of plastic clay

LL 40	PL 18	PI 22	Gs 2.72	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

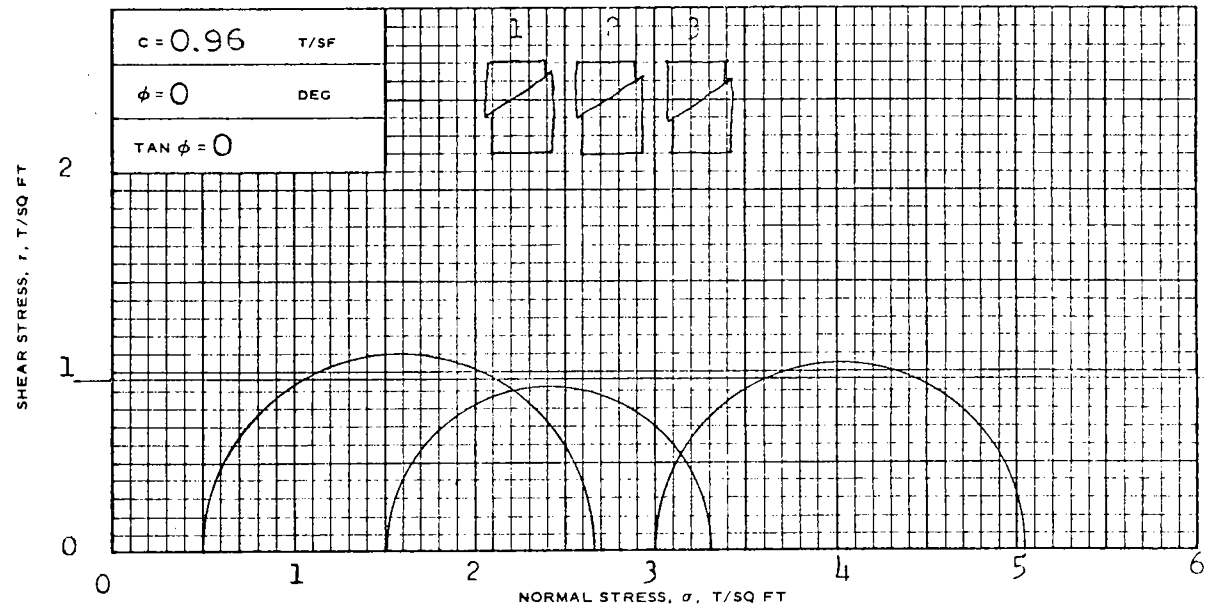
BORING NO. X-13-U SAMPLE NO. 18-D

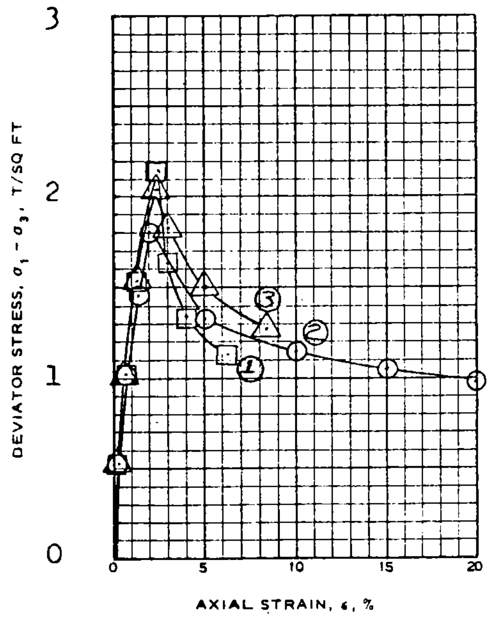
~~DEPTH~~/ELEV -95.6 MSL

LABORATORY USAEWES DATE 2 November 1971

GDA TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	45.4	45.4	45.6	
	DRY DENSITY LB/CU FT	γ_{d_o}	76.1	76.1	76.1	
	SATURATION, %	S_o	99.1	99.1	99.5	
	VOID RATIO	e_o	1.26	1.26	1.26	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.15	1.80	2.04	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	32	27	31	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray

LL 80	PL 24	PI 56	Gs 2.75	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM, 1971

BORING NO. X-13-U SAMPLE NO. 23-B

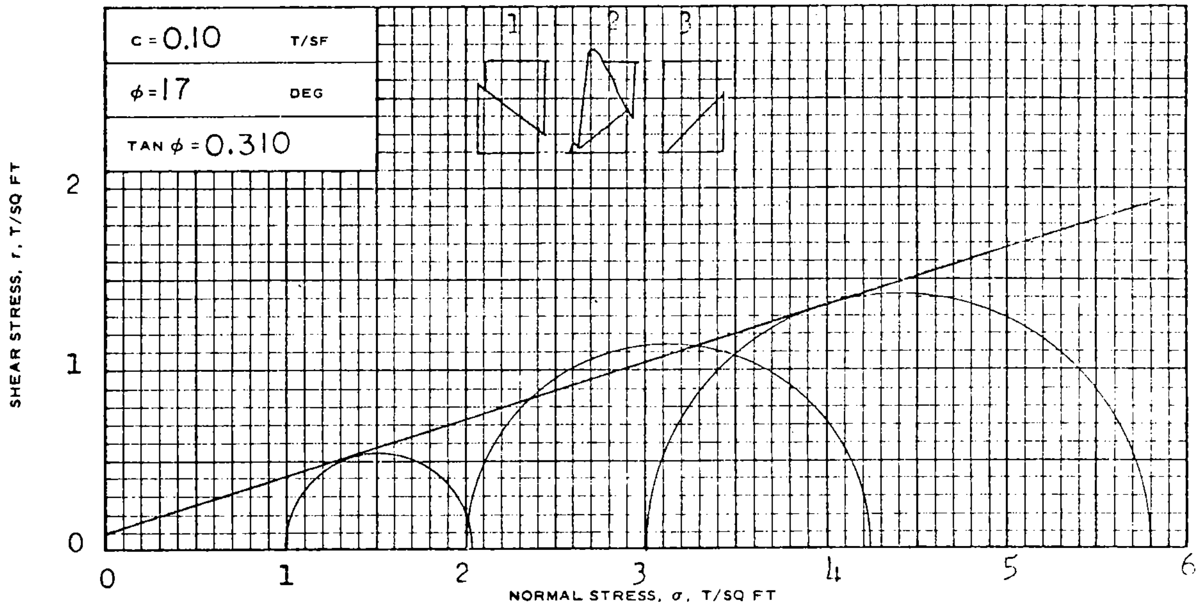
DEPTH/ELEV -113.9 MSL

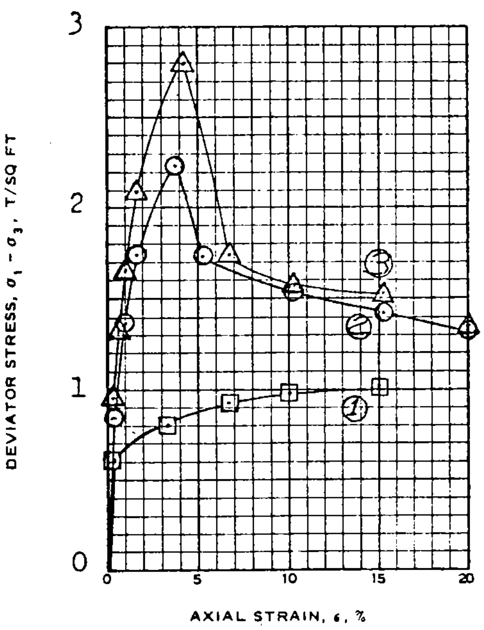
LABORATORY USAEWES DATE 28 October 1971

GDA **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*

Fig. A - 99





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	44.3	43.9	44.2
	DRY DENSITY LB/CU FT	γd_o	77.1	77.4	77.2
	SATURATION, %	S_o	98.8	98.6	98.8
	VOID RATIO	e_o	1.24	1.23	1.24
BEFORE SHEAR	WATER CONTENT, %	w_c	45.7	44.9	44.3
	DRY DENSITY LB/CU FT	γd_c	77.6	79.1	80.2
	SATURATION, %	S_c	100+	100+	100+
	VOID RATIO	e_c	1.23	1.18	1.15
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.84	6.84	6.84
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		1.01	2.24	2.80
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	441	115	120
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), gray, slickensided

LL 71	PL 23	PI 48	Gs 2.77	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS:

PROJECT LK. PONT., LA.& VIC.-HURR. PROT.(1971)
RIGOLETS CONTROL STR.& CLOSURE DAM; DDM#6

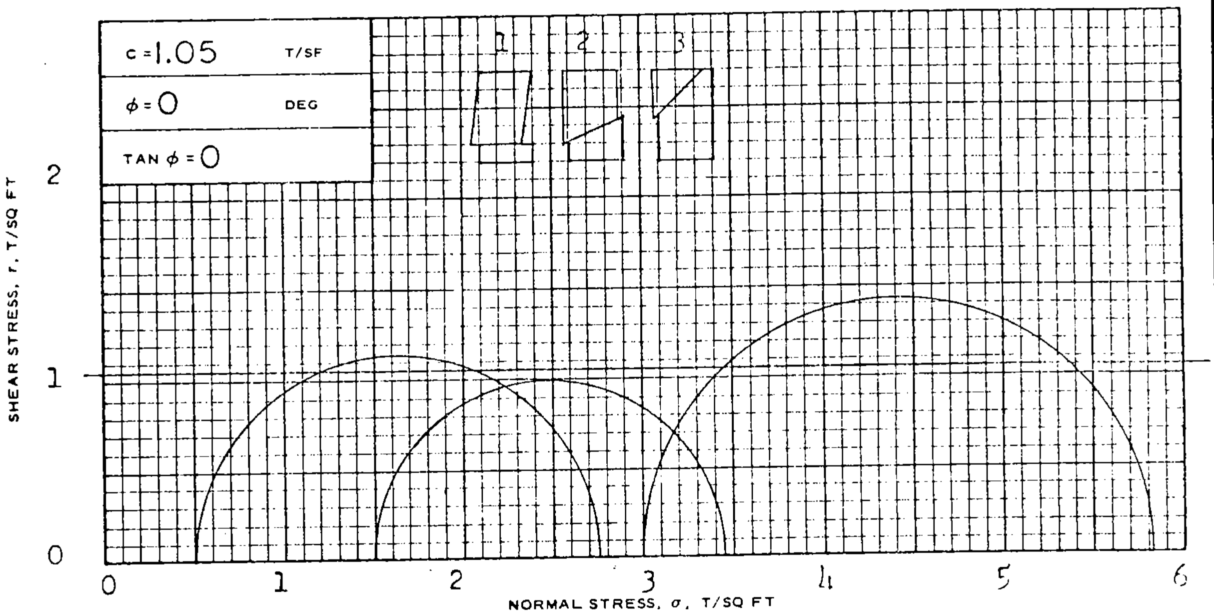
BORING NO. X-13U SAMPLE NO. 30-A

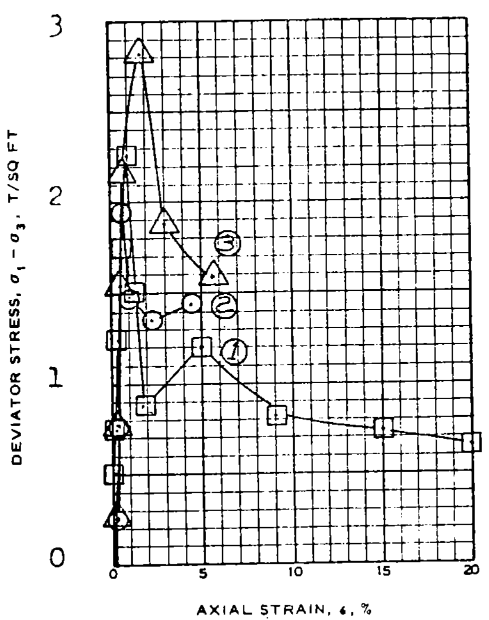
~~DEPTH~~/ELEV -141.0 MSL

LABORATORY USAEWES DATE 16 November 1971

JAL **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	50.3	50.6	50.3	
	DRY DENSITY LB/CU FT	γd_o	71.6	71.4	71.7	
	SATURATION, %	S_o	99.5	99.4	99.5	
	VOID RATIO	e_o	1.38	1.39	1.38	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.26	1.94	2.81	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	71	14	29	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH),gray

LL 85	PL 27	PI 58	Gs 2.73	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.- HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

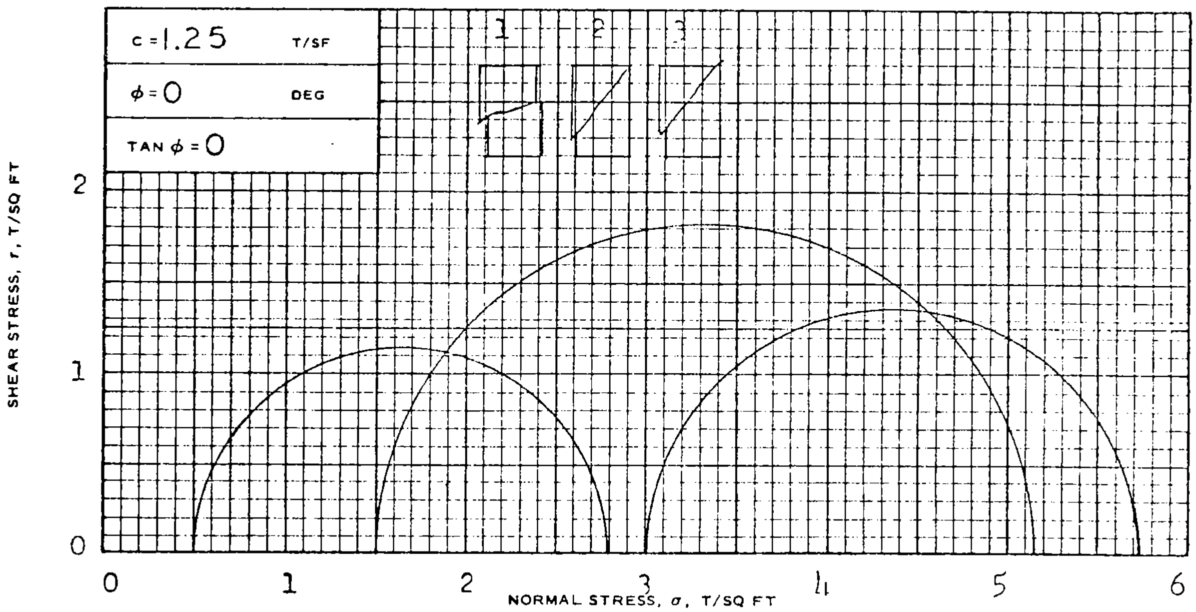
BORING NO. X-13-U SAMPLE NO. 31-C

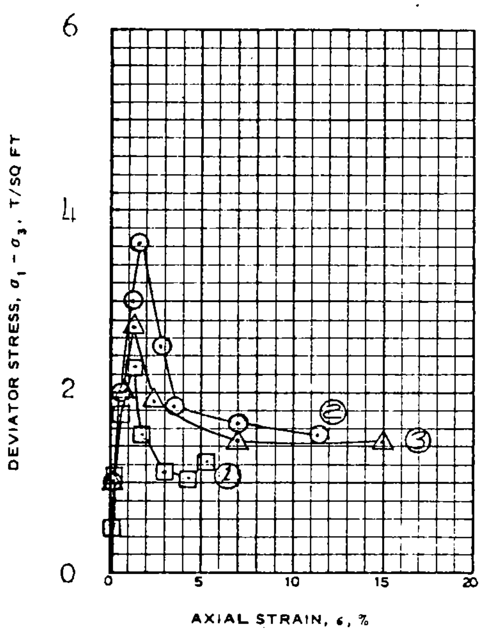
~~DEPTH~~/ELEV -146.7 MSL

LABORATORY USAEWES DATE 1 November 1971

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	42.1	40.9	45.4	
	DRY DENSITY LB/CU FT	γd_o	79.7	81.2	76.3	
	SATURATION, %	S_o	100+	100+	99.4	
	VOID RATIO	e_o	1.16	1.12	1.26	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.28	3.65	2.71	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	16	22	12	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.39	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), gray, fissured

LL 76	PL 25	PI 51	Gs 2.76	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR. & CLOSURE DAM 1971

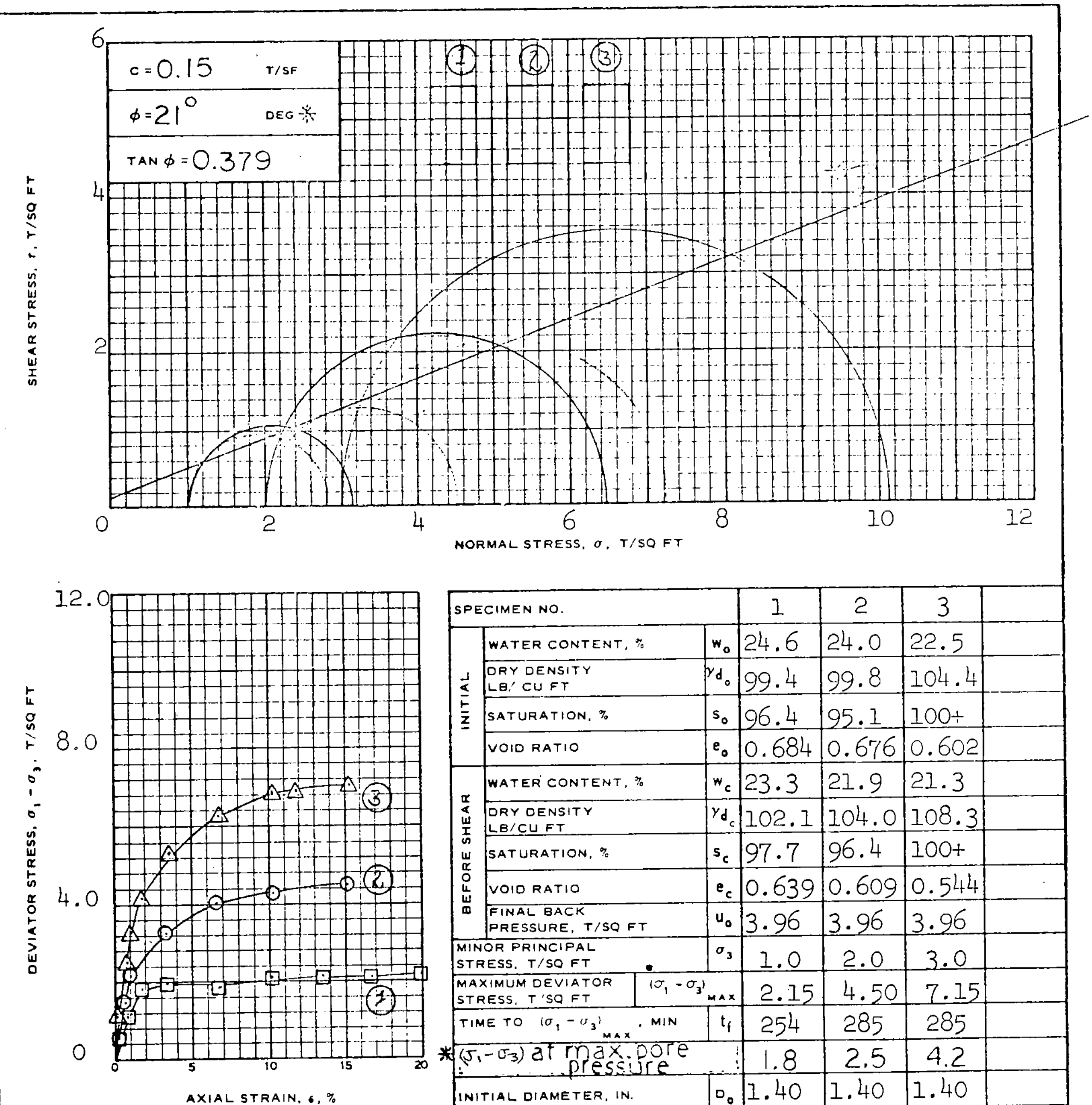
BORING NO. X-13-U | SAMPLE NO. 36-C

~~DEPTH~~/ELEV -166.8 MSL

LABORATORY USAEWES | DATE 2 November 1971

CDA **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089
REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*



SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	24.6	24.0	22.5
	DRY DENSITY LB/CU FT	γ_{d_o}	99.4	99.8	104.4
	SATURATION, %	S_o	96.4	95.1	100+
	VOID RATIO	e_o	0.684	0.676	0.602
BEFORE SHEAR	WATER CONTENT, %	w_c	23.3	21.9	21.3
	DRY DENSITY LB/CU FT	γ_{d_c}	102.1	104.0	108.3
	SATURATION, %	S_c	97.7	96.4	100+
	VOID RATIO	e_c	0.639	0.609	0.544
	FINAL BACK PRESSURE, T/SQ FT	u_o	3.96	3.96	3.96
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.15	4.50	7.15
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	254	285	285
* $(\sigma_1 - \sigma_3)$ at max. pore pressure			1.8	2.5	4.2
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS CLAYEY SAND(SC),gray, contains pockets of plastic clay

LL -	PL -	PI -	Gs 2.68	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS:

PROJECT LK. PONT., LA.-HURR. PROT. RIGOLETS CONT. STR.&CLOSURE DAM, 1971

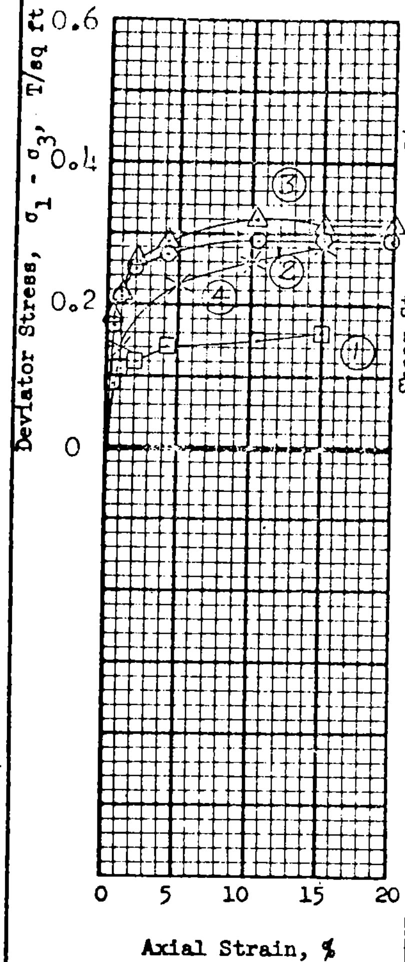
BORING NO. X-13U SAMPLE NO. 39-C

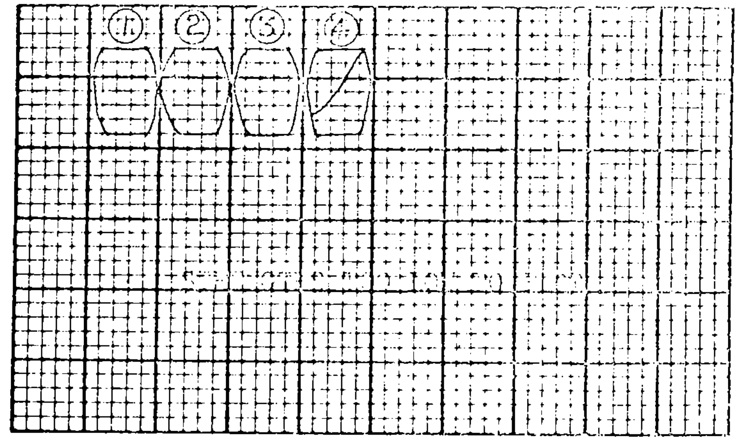
DEPTH/ELEV -178.8 MSL

LABORATORY USAEWES DATE 16 November 1971

JAL **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





Normal Stress, σ, T/sq ft

Test No.			1	2	3	4
Initial	Water content	w_o	64.5 %	52.0 %	51.0 %	51.6%
	Void ratio	e_o	1.75	1.42	1.41	1.42
	Saturation	S_o	100+ %	100+%	99.1 %	99.6%
	Dry density, lb/cu ft	γ_d	62.1	70.8	71.0	70.6
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	0.5
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.16	0.29	0.32	0.28
Time to failure, min		t_f	96	67	67	31
Rate of strain, percent/min			0.155	0.155	0.155	0.488
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.41	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.15 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),gray, contains root fragments up to 1/4" in*

LL 82 | PL 21 | PI 61 | G_s 2.74

Remarks *diameter

Project LK.PONT.LA.,& VIC.-HURR.PROT. - RIGOLETS CONTROL STRUCT. & CLOSURE DAM

Area

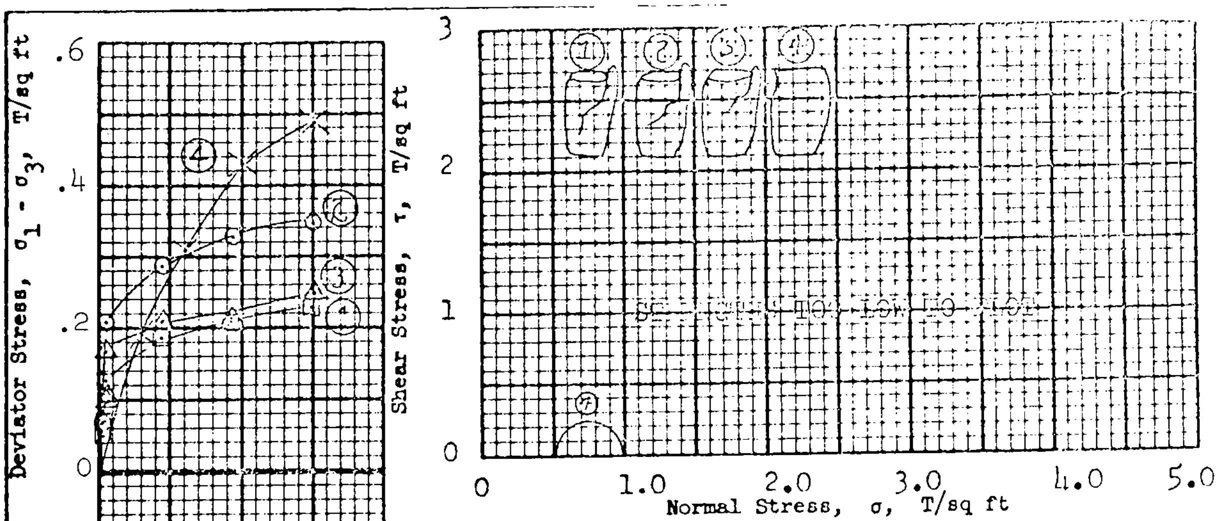
Boring No. X 14 - U | Sample No. 1-C

~~Depth~~ El - 2.8 MSL | Date 15 June 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT



Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.13 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	4
Initial	Water content	w_o	48.9 %	46.2 %	50.4 %	34.3%
	Void ratio	e_o	1.27	1.23	1.23	0.938
	Saturation	S_o	100+ %	100+ %	100+ %	92.5%
	Dry density, lb/cu ft	γ_d	74.5	76.2	72.8	87.6
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	0.5
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.22	0.34	0.25	0.49
Time to failure, min		t_f	109	109	109	31
Rate of strain, percent/min			0.138	0.138	0.138	0.480
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.38	1.39	1.38	1.40
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),gray, contains 1/8" in diameter concretions

LL 56 | PL 20 | PI 36 | G_s 2.72

Remarks ______

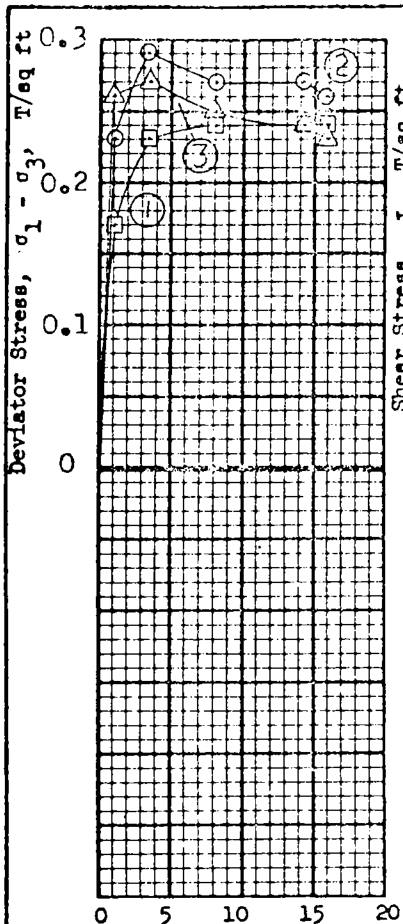
Project LK. PONT.LA.&VIC.-HURR. PROT.-
RIGOLETS CONTROL STR.& CLOSURE DAM, DDM NO.6

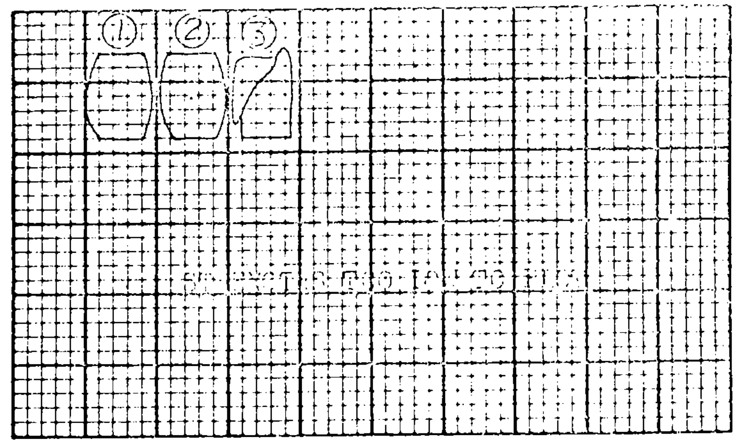
Area 1971

Boring No. X14-U | Sample No. 2-D

~~Depth~~ El -7.6 MSL | Date 17 June 1971

TES TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.13 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	37.5 %	35.5 %	39.8 %	%
	Void ratio	e_o	0.994	0.948	1.05	
	Saturation	S_o	100+ %	100+ %	100+ %	%
	Dry density, lb/cu ft	γ_d	84.2	86.2	81.9	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.24	0.29	0.27	
Time to failure, min		t_f	56	24	24	
Rate of strain, percent/min			0.142	0.142	0.142	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification SANDY CLAY(CL),gray

LL 42 | PL 14 | PI 28 | G_s 2.69

Remarks ______

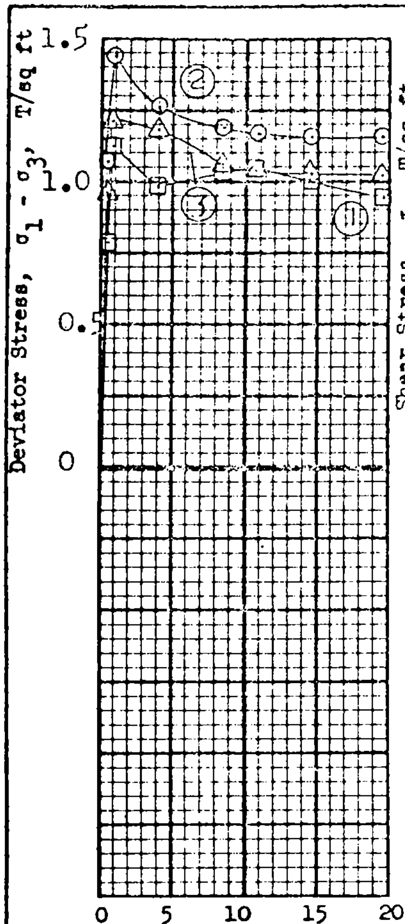
Project LK.PONT.LA.,& VIC.,- HURR.PROT.- RIGOLETS CONTROL STRUCT. & CLOSURE DAM

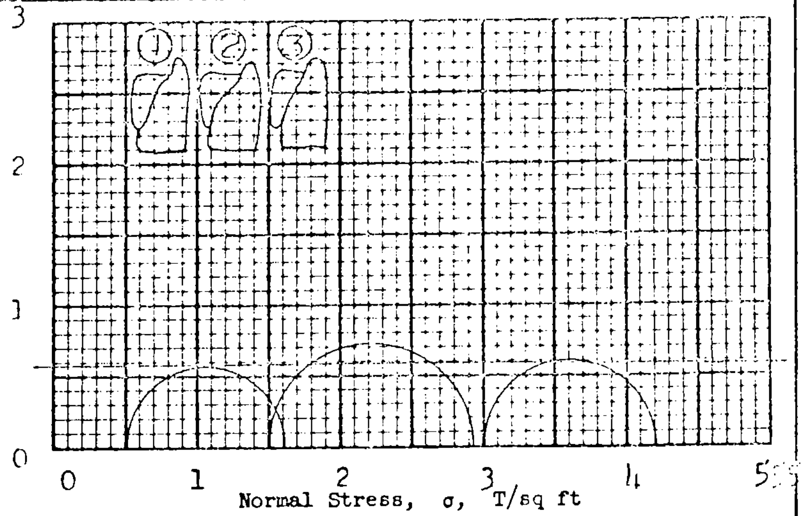
Area

Boring No. X 14 - U | Sample No. 4-C

~~Depth~~ El - 15.1 MSL | Date 17 June 1971

TES TRIAXIAL COMPRESSION TEST REPORT





Test No.			1	2	3	
Initial	Water content	w_o	40.3 %	38.3 %	39.8 %	%
	Void ratio	e_o	1.11	1.05	1.09	
	Saturation	S_o	98.4 %	98.8 %	99.0 %	%
	Dry density, lb/cu ft	γ_d	80.3	82.5	81.1	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.12	1.44	1.21	
Time to failure, min		t_f	0.69	0.69	0.69	
Rate of strain, percent/min			0.130	0.130	0.130	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.40	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.58 T/sq ft

Method of saturation ______

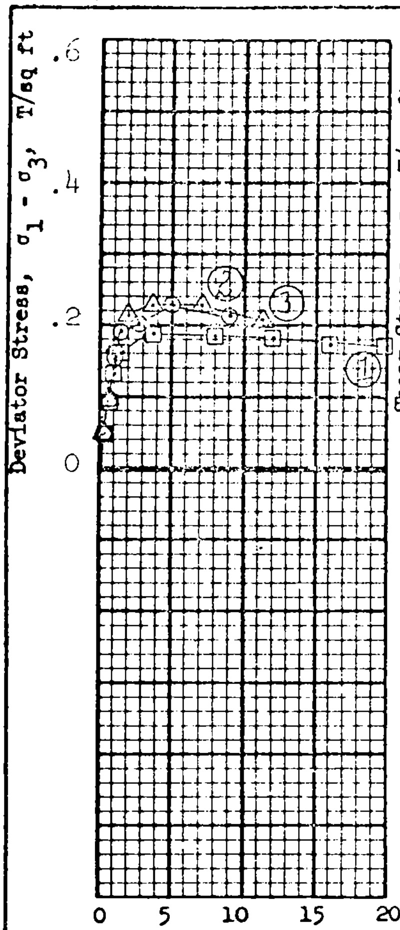
☐ Controlled stress

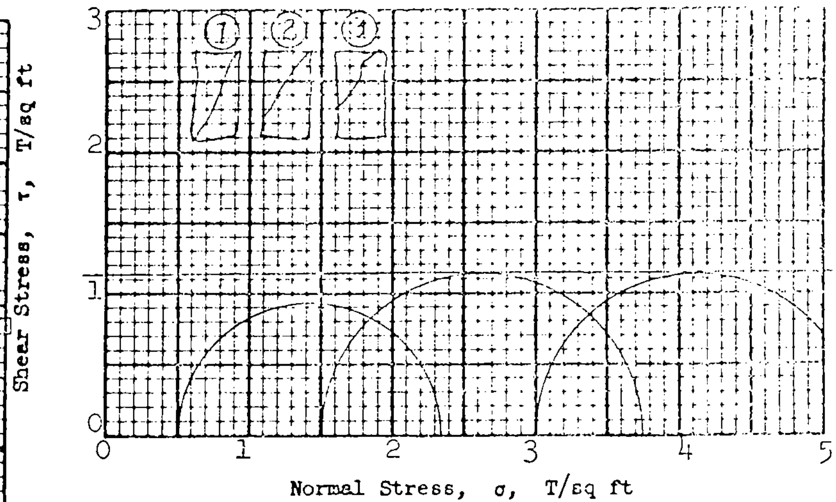
☒ Controlled strain

Type of test Q	**Type of specimen** UNDISTURBED		
Classification PLASTIC CLAY(CH),gray, contains pockets of fine sand up to*			
LL 56	**PL** 17	**PI** 39	G_s 2.71

Remarks *1/2" in diameter

Project LK.PONT.LA.,&VIC.-HURR.PROT.- RIGOLETS CONTROL STRUCT. & CLOSURE DAM	
Area	
Boring No. X 14- U	**Sample No.** 15-D
Depth El - 60.1 MSL	**Date** 18 June 1971
TFS	TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.13 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	28.8 %	26.8 %	27.9 %	%
Initial	Void ratio	e_o	0.822	0.759	0.800	
Initial	Saturation	S_o	96.4 %	97.1%	95.9 %	%
Initial	Dry density, lb/cu ft	γ_d	94.2	97.6	95.4	
Before Shear	Water content	w_c	%	%	%	%
Before Shear	Void ratio	e_c				
Before Shear	Saturation	S_c	%	%	%	%
Before Shear	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
Final	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.85	2.25	2.26	
Time to failure, min		t_f	18	28	24	
Rate of strain, percent/min			0.206	0.176	0.153	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.42	1.42	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),gray

LL 55 | PL 18 | PI 37 | G_s 2.75

Remarks ________

Project LK. PONT.LA.& VIC.-HURR. PROT.-
RIGOLETS CONTROL STR. & CLOSURE DAM, DDM # 6

Area (1971)

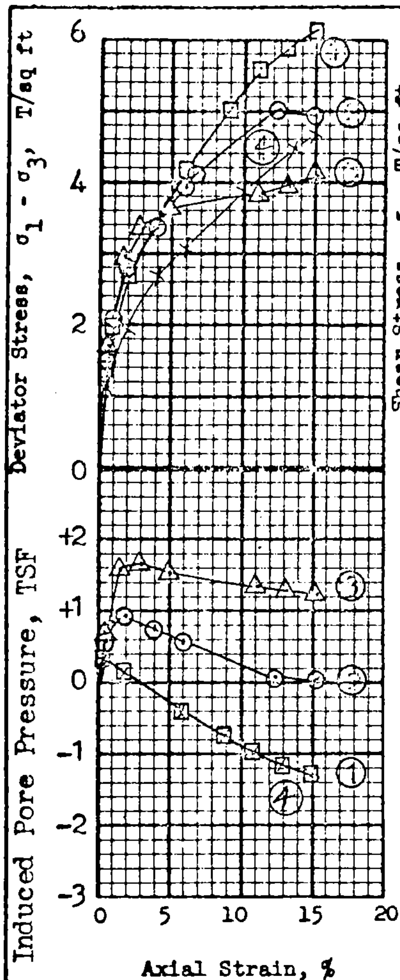
Boring No. X14-U | Sample No. 17-D

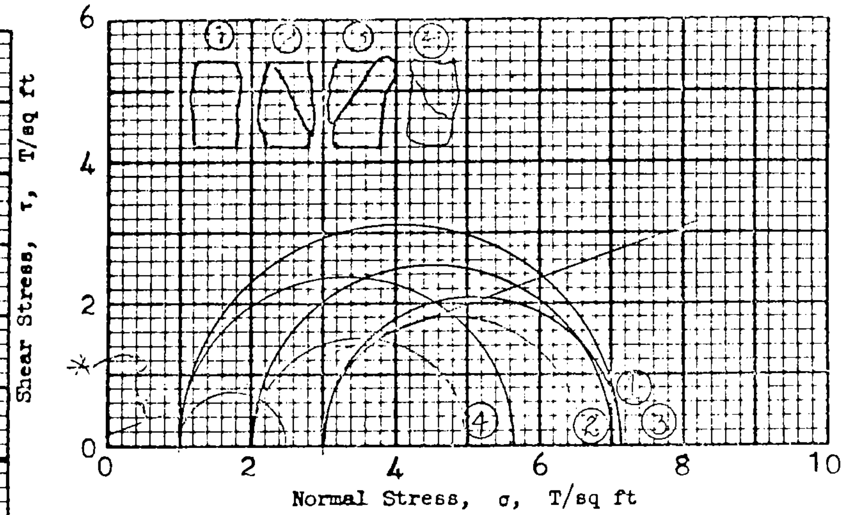
Depth El -68.0 MSL | Date 18 June 1971

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS





Shear Strength Parameters

* φ = 20°

tan φ = 0.366

c = 0.15 T/sq ft

Method of saturation BP

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	4
Initial	Water content	w_o	25.9 %	27.5 %	27.6 %	26.6 %
	Void ratio	e_o	0.716	0.750	0.750	0.740
	Saturation	S_o	97.7 %	99.0 %	99.4 %	97.0 %
	Dry density, lb/cu ft	γ_d	98.2	96.3	96.3	96.9
Before Shear	Water content	w_c	25.8 %	26.6 %	26.6 %	26.4 %
	Void ratio	e_c	0.708	0.722	0.724	0.720
	Saturation	S_c	98.4 %	99.5 %	99.2 %	99.0 %
	~~Final back pressure~~, PSI	u_o	70	70	70	75
	Dry Density Lbs/cu.ft.	γ_d	98.7	97.9	97.8	98.0
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	1.0	2.0	3.0	1.0
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	6.14	5.01	4.14	4.66
Time to failure, min		t_f	114	94	114	120
Rate of strain, percent/min			0.132	0.132	0.131	0.120
*$(\sigma_1-\sigma_3)$ at max pore pressure			1.5	3.0	3.6	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.39	1.39	1.40
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Type of test R	Type of specimen UNDISTURBED			
Classification SANDY SILT(ML), gray, contains 1/8" lenses of plastic clay				
LL 25	PL 22	PI 3		G_s 2.70

Remarks See attached plot for effective values

Portion of sample allowed to drain before trimming

Sheet 1 of 2

Insufficient material for check test.

Project LK. PONT., LA.& VIC.-HURR. PROT.

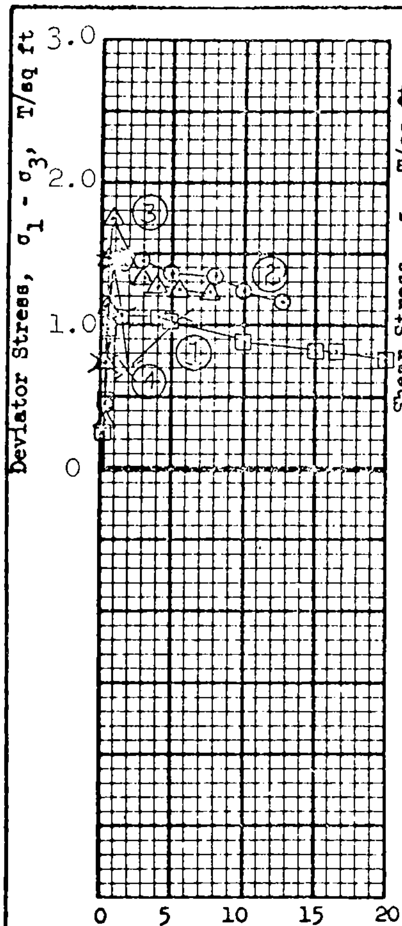
RIGOLETS CONTROL STR.&CLOSURE DAM, DDM#6(1971)

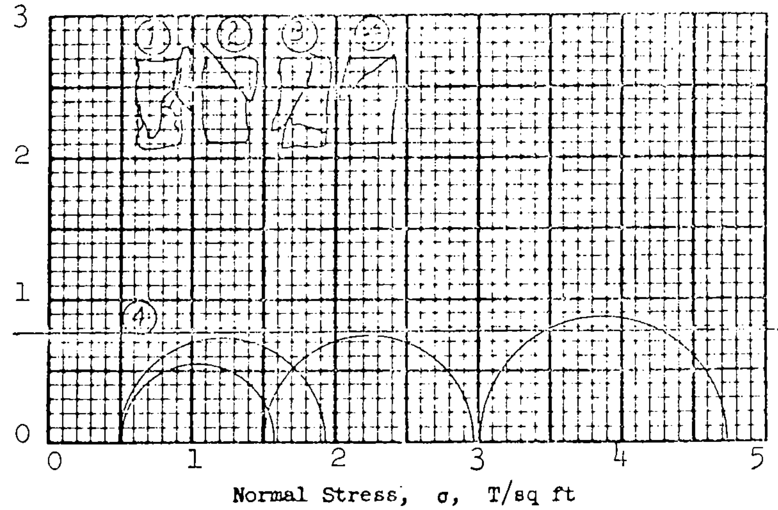
Area

Boring No. X-14-U | Sample No. 18-D

~~Depth~~ El -71.9 MSL | Date 21 June, 1971

PJR TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.78 T/sq ft

Method of saturation ________

[ ] Controlled stress

[X] Controlled strain

Test No.			1	2	3	4
Initial	Water content	w_o	54.3 %	49.4 %	54.2%	50.3%
	Void ratio	e_o	1.52	1.38	1.51	1.40
	Saturation	S_o	98.2 %	98.4 %	98.7 %	98.8%
	Dry density, lb/cu ft	γ_d	68.2	72.2	68.4	71.4
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	0.5
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.07	1.48	1.74	1.44
Time to failure, min		t_f	71	12	27	8
Rate of strain, percent/min			0.210	0.081	0.030	0.100
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.41	1.40	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH),gray

LL 99 | PL 29 | PI 70 | G_s 2.75

Remarks ________

Project LK. PONT.LA.&VIC.-HURR. PROT.-
RIGOLETS CONTROL STR.& CLOSURE DAM, DDM #6

Area

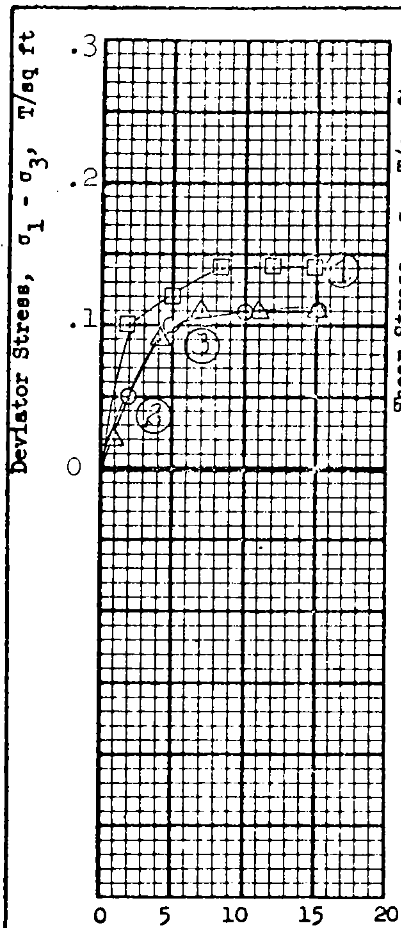
Boring No. X 14-U | Sample No. 21-C

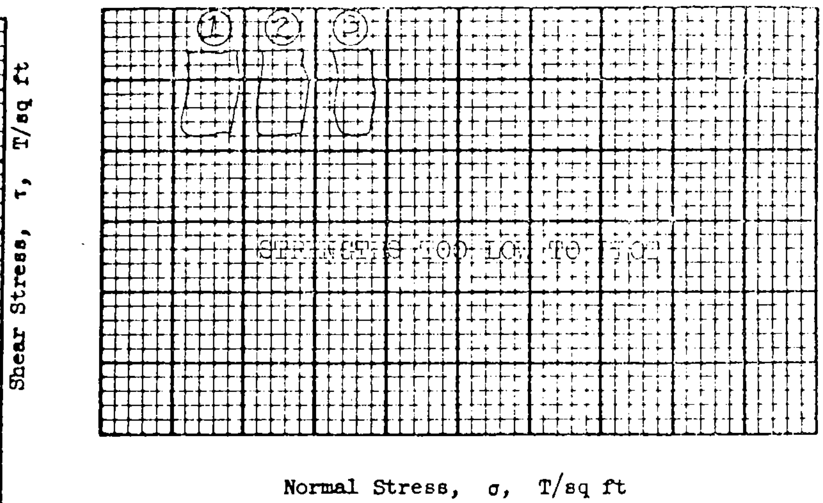
~~Depth~~ El -83.0 MSL | Date 21 June 1971

FAM TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOI





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.06 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	308.7%	327.0%	320.5%	%
	Void ratio	e_o	7.14	7.64	7.34	
	Saturation	S_o	98.1 %	97.2 %	99.1 %	%
	Dry density, lb/cu ft	γ_d	17.4	16.4	17.0	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.14	0.11	0.11	
Time to failure, min		t_f	17	35	8	
Rate of strain, percent/min			7.14	7.64	7.34	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q	Type of specimen UNDISTURBED			
Classification ORGANIC CLAY(OH), dark brown				
LL 315	PL 137	PI 178		G_s 2.27

Remarks ________

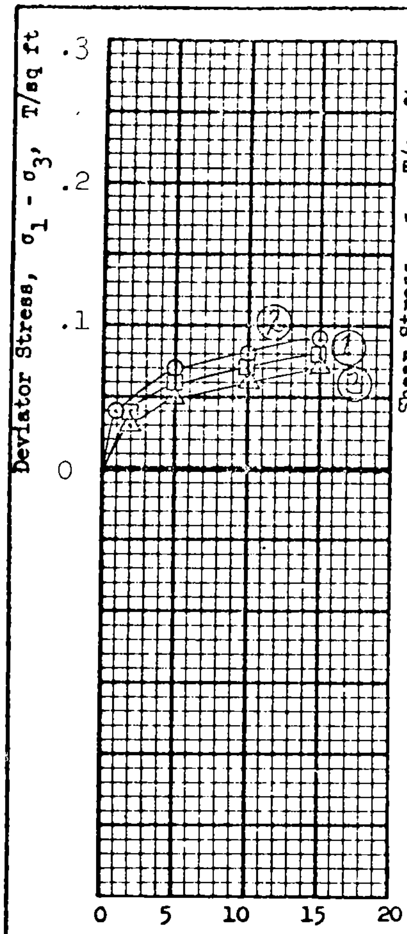
Project LK. PONT. LA. & VIC. - HURR. PROT. - RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,

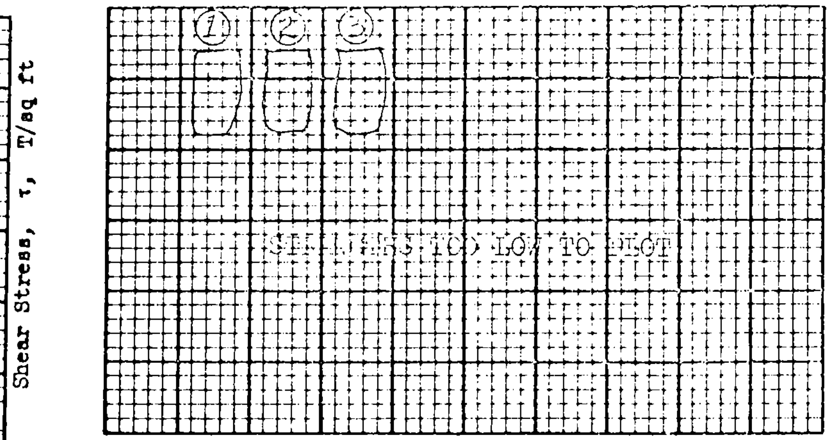
Area DDM NO. 6

Boring No. X-15-U | Sample No. 2-B

Depth El -2.5 MSL | Date 21 July 1971

JMS TRIAXIAL COMPRESSION TEST REPORT





Normal Stress, σ, T/sq ft

Test No.			1	2	3	
Initial	Water content	w_o	60.1 %	55.2 %	58.6 %	%
	Void ratio	e_o	1.67	1.49	1.64	
	Saturation	S_o	97.2 %	100 %	96.5 %	%
	Dry density, lb/cu ft	γ_d	63.1	67.6	63.9	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.08	0.09	0.07	
Time to failure, min		t_f	74	70	42	
Rate of strain, percent/min			0.203	0.214	0.360	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.41	1.41	1.41	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.04 T/sq ft

Method of saturation ________

☐ Controlled stress

☒ Controlled strain

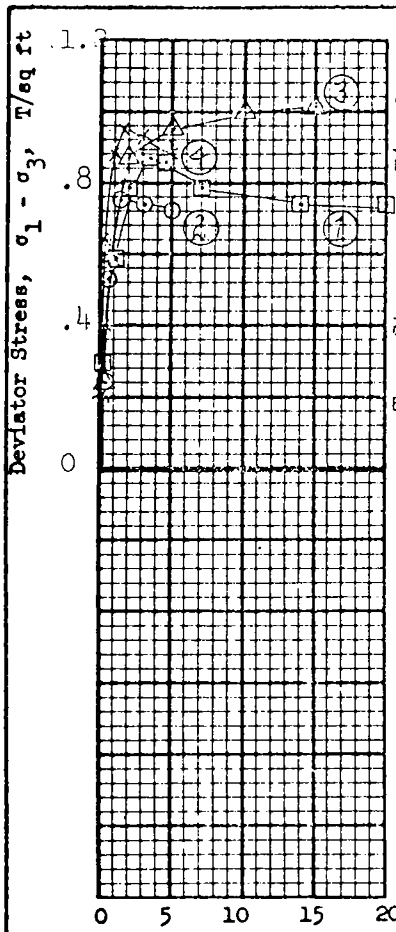
Type of test Q	Type of specimen UNDISTURBED

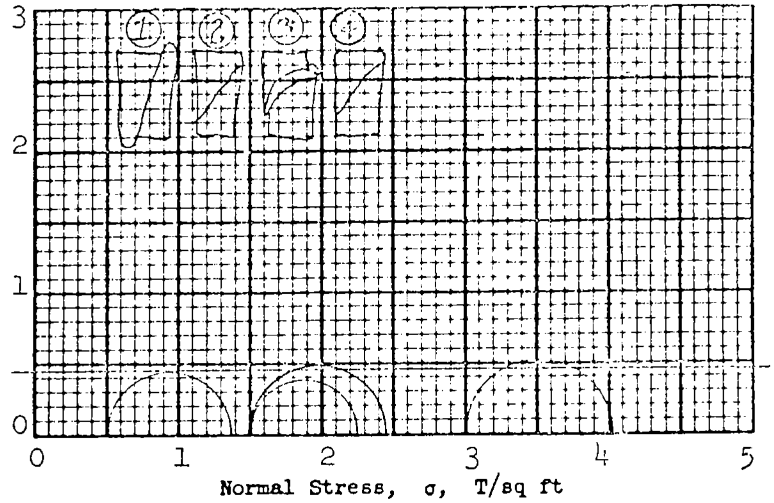
Classification PLASTIC CLAY(CH), gray

LL 55	PL 19	PI 36		G_s 2.70

Remarks ________

Project LK. PONT., LA. & VIC. -HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,	
Area DDM NO. 6	
Boring No. X 15-U	Sample No. 3-B
Depth El -6.0 MSL	Date 21 July 1971
JMS TRIAXIAL COMPRESSION TEST REPORT	





Shear Strength Parameters

φ = 0°

tan φ = 0

c = 0.45 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	4
Initial	Water content	w_o	31.0 %	43.6 %	36.0 %	38.8 %
	Void ratio	e_o	0.989	1.20	1.03	1.14
	Saturation	S_o	86.2 %	99.9 %	96.1 %	93.6 %
	Dry density, lb/cu ft	γ_d	86.3	77.9	84.5	80.3
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	1.5
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.87	0.75	1.02	0.95
Time to failure, min		t_f	17	10	197	18
Rate of strain, percent/min			0.208	0.139	0.076	0.091
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.41	1.42	1.42	1.41
Initial height, in.		H_o	3.00	3.00	3.00	3.00

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH), bluish gray, contains pockets of fine sand

LL 86 | PL 23 | PI 63 | | G_s 2.75

Remarks ______

Project LK. PONT., LA.& VIC.-HURR. PROT.- RIGOLETS CONTROL STRUCTURE AND CLOSURE DAM,

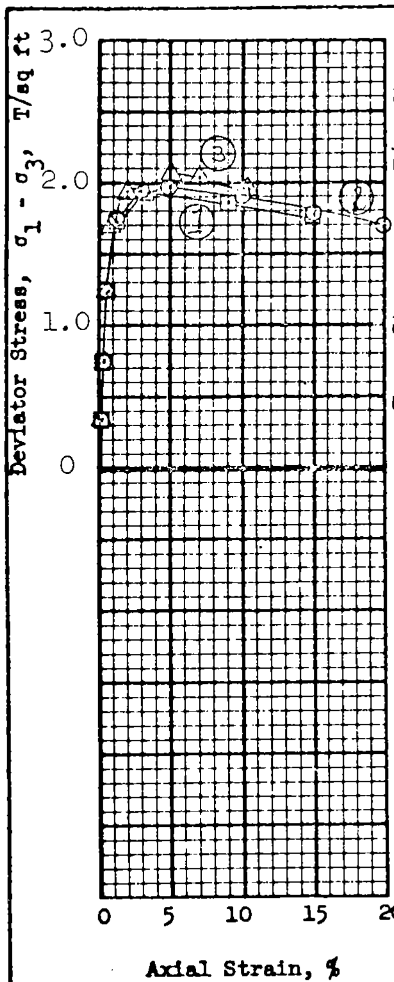
Area DDM NO. 6

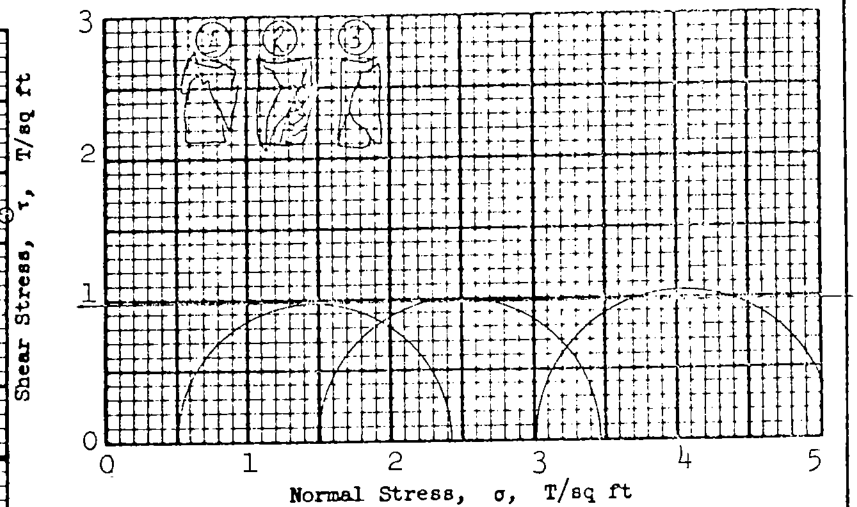
Boring No. X 15-U | Sample No. 6-B

Depth El -18.3 MSL | Date 22 July 1971

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE TRANSLUCENT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.98 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

				1	2	3	
Test No.				1	2	3	
Initial	Water content		w_o	29.0 %	33.4 %	31.9 %	%
	Void ratio		e_o	0.823	0.934	0.902	
	Saturation		S_o	97.2 %	98.7 %	97.6 %	%
	Dry density, lb/cu ft		γ_d	94.5	89.1	90.6	
Before Shear	Water content		w_c	%	%	%	%
	Void ratio		e_c				
	Saturation		S_c	%	%	%	%
	Final back pressure, T/sq ft		u_o				
Final	Water content		w_f	%	%	%	%
	Void ratio		e_f				
Minor principal stress, T/sq ft			σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft			$(\sigma_1-\sigma_3)_{max}$	1.93	1.98	2.06	
Time to failure, min			t_f	12	24	27	
Rate of strain, percent/min				0.283	0.205	0.185	
Ult deviator stress, T/sq ft			$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.			D_o	1.41	1.40	1.41	
Initial height, in.			H_o	3.00	3.00	3.00	

Type of test Q	Type of specimen UNDISTURBED			
Classification PLASTIC CLAY(CH),gray, contains scattered concretions				
LL 56	PL 20	PI 36		G_s 2.76

Remarks ______

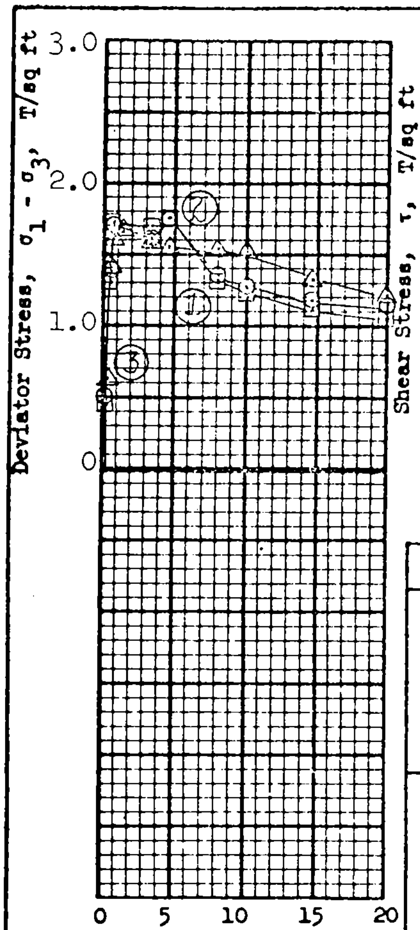
Project LK. PONT.,LA.& VIC.-HURR. PROT.-
RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,

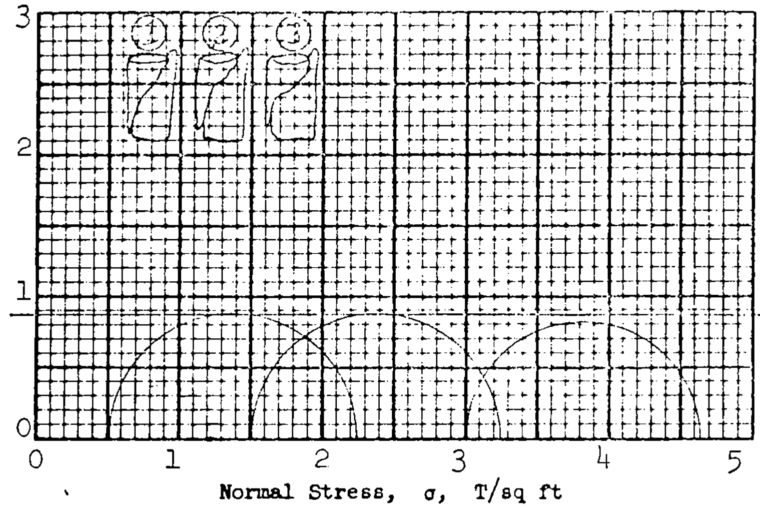
Area DDM NO. 6

Boring No. X 15-U | Sample No. 19-D

Depth El -71.9 MSL | Date 23 July 1971

FAM TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 0.875 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	47.2 %	47.4 %	48.8 %	%
	Void ratio	e_o	1.30	1.32	1.35	
	Saturation	S_o	99.5 %	98.4 %	99.0 %	%
	Dry density, lb/cu ft	γ_d	74.4	73.8	72.8	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	1.75	1.75	1.64	
Time to failure, min		t_f	46	46	13	
Rate of strain, percent/min			0.101	0.101	0.101	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q **Type of specimen** UNDISTURBED

Classification PLASTIC CLAY(CH), gray, contains seams and pockets of silt

LL 93 **PL** 24 **PI** 69 **G_s** 2.74

Remarks ______

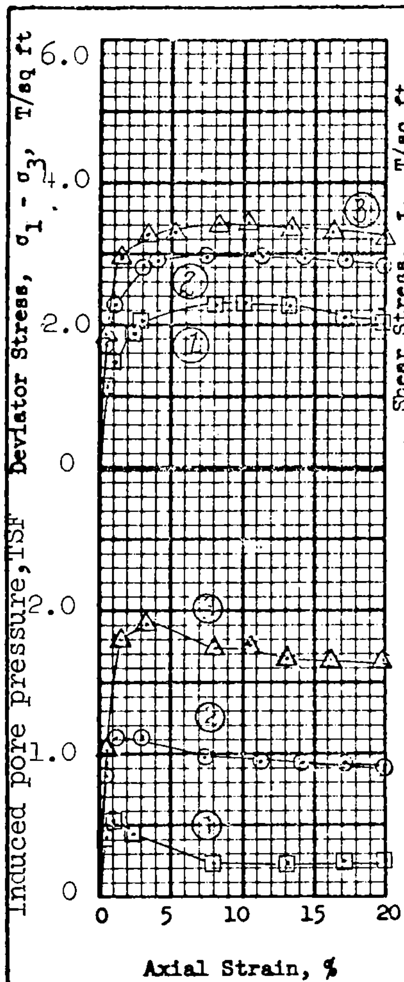
Project LK. PONT., LA.& VIC.-HURR. PROT.-RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,

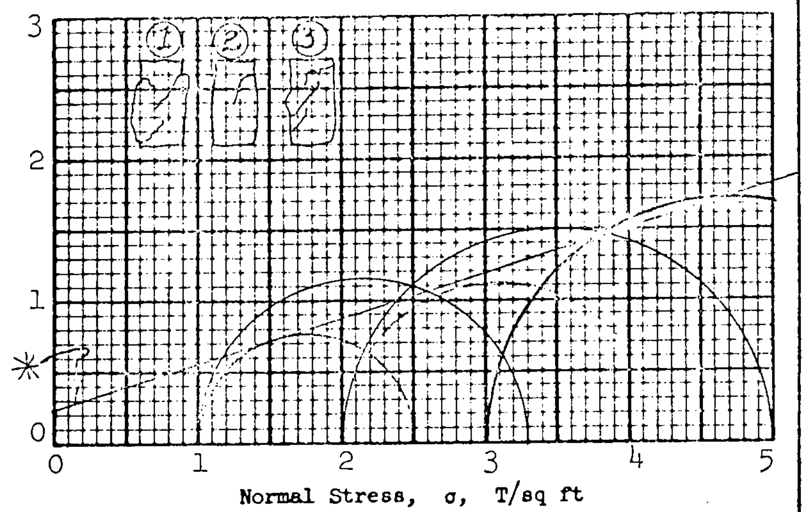
Area DDM NO.6, (1971)

Boring No. X-15-U **Sample No.** 22-D

Depth El -84.0 MSL **Date** 22 July 1971

TES TRIAXIAL COMPRESSION TEST REPORT





Shear Strength Parameters

φ = 18°

tan φ = 0.32

c = 0.22 T/sq ft

Method of saturation ______

[ ] Controlled stress

[x] Controlled strain

Test No.			1	2	3	
Initial	Water content	w_o	29.6 %	29.4 %	29.2 %	%
	Void ratio	e_o	0.838	0.814	0.801	
	Saturation	S_o	95.4 %	97.5 %	98.4 %	%
	Dry density, lb/cu ft	γ_d	91.7	92.9	93.6	
Before Shear	Water content	w_c	30.3 %	29.6 %	29.1 %	%
	Void ratio	e_c	0.797	0.750	0.718	
	Saturation	S_c	100+ %	100+ %	100+ %	%
	Final back pressure, T/sq ft	u_o	70	70	70	
	Dry Density Lbs./cu.ft.	γ_d	93.8 %	96.3 %	98.1 %	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	1.0	2.0	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.30	2.99	3.41	
Time to failure, min		t_f	74	55	60	
Rate of strain, percent/min			0.135	0.135	0.135	
* $(\sigma_1 - \sigma_3)$ at max. pore pressure			1.5	2.2	3.3	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.40	1.40	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test R	Type of specimen UNDISTURBED			
Classification SILTY CLAY(CL), gray, contains trace of fine sand				
LL 36	PL 17	PI 19		G_s 2.70

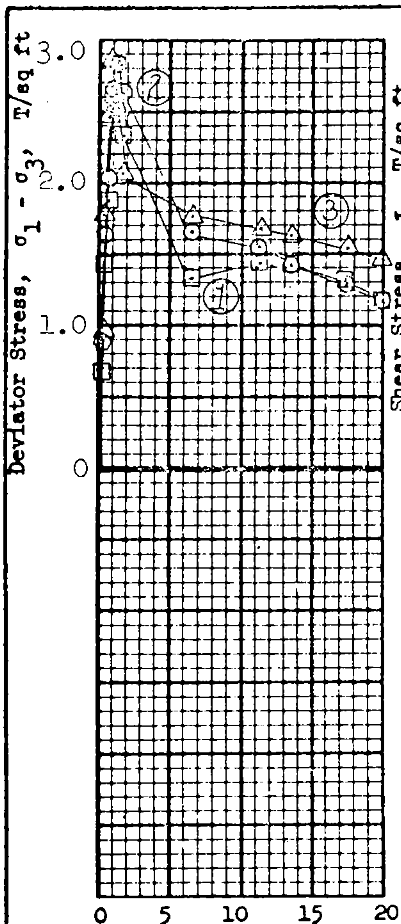
Remarks See attached plot for effective values.

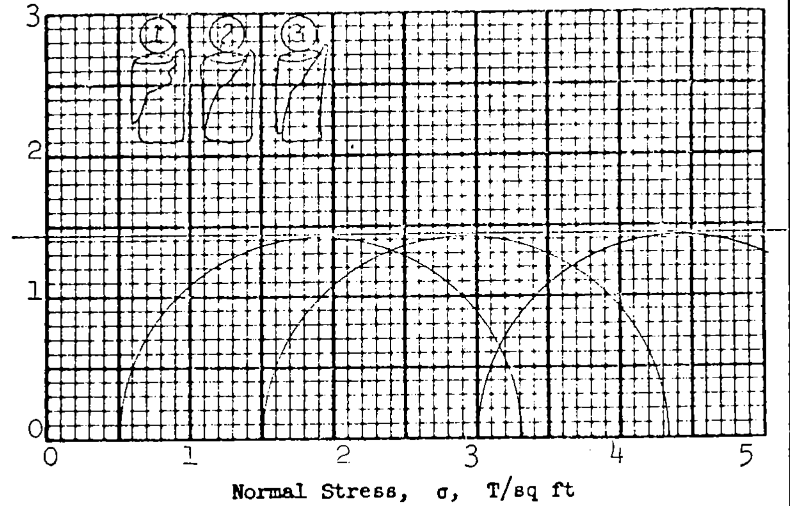
Sheet 1 of 2

Project LK. PONT., LA.&VIC.-HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,	
Area DDM NO. 6	
Boring No. X 15-U	Sample No. 25-B
Depth El -94.1 MSL	Date 20 July 1971
PJR & JAL TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT





Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 1.42 T/sq ft

Method of saturation ______

☐ Controlled stress

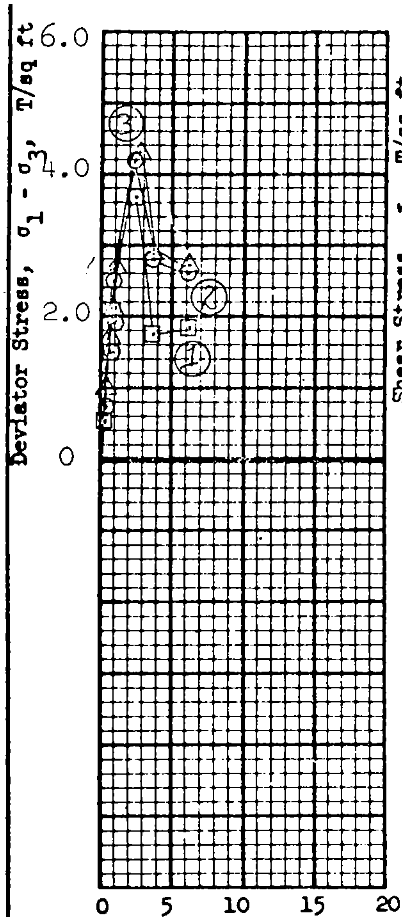
☒ Controlled strain

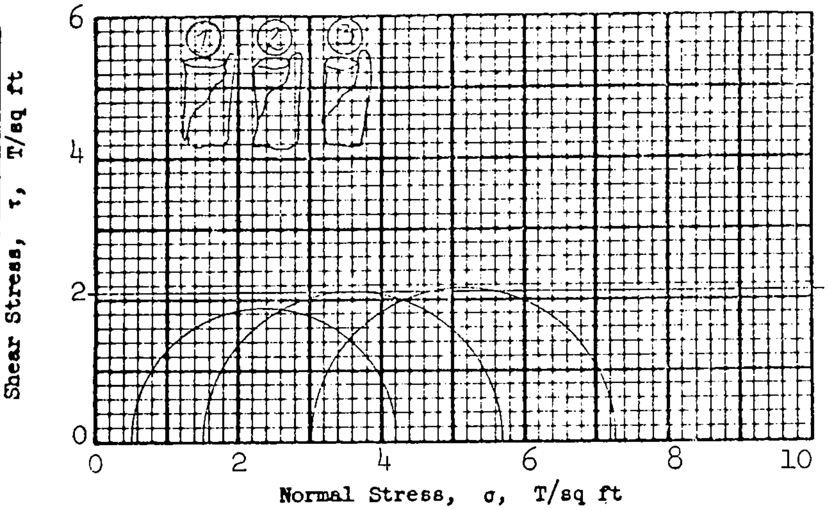
	Test No.		1	2	3	
Initial	Water content	w_o	42.2 %	42.6 %	41.9 %	%
	Void ratio	e_o	1.15	1.16	1.14	
	Saturation	S_o	100+ %	100+ %	100+ %	%
	Dry density, lb/cu ft	γ_d	79.2	78.8	79.5	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	2.81	2.85	2.86	
Time to failure, min		t_f	13	14	11	
Rate of strain, percent/min			0.094	0.094	0.094	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Type of test Q	Type of specimen UNDISTURBED		
Classification PLASTIC CLAY(CH),gray			
LL 74	PL 25	PI 49	G_s 2.73

Remarks ______

Project LK. PONT.,LA.& VIC.-HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,	
Area DDM NO. 6	
Boring No. X 15-U	Sample No. 34-B
Depth El -130.1 MSL	Date 22 July 1971
TES	TRIAXIAL COMPRESSION TEST REPORT





Test No.			1	2	3	
Initial	Water content	w_o	60.7 %	62.2 %	60.6 %	%
	Void ratio	e_o	1.63	1.67	1.62	
	Saturation	S_o	97.6 %	97.6 %	98.0 %	%
	Dry density, lb/cu ft	γ_d	62.1	61.3	62.5	
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft		σ_3	0.5	1.5	3.0	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	3.70	4.20	4.27	
Time to failure, min		t_f	29	28	31	
Rate of strain, percent/min			0.087	0.087	0.087	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o	1.39	1.39	1.39	
Initial height, in.		H_o	3.00	3.00	3.00	

Shear Strength Parameters

ϕ = 0°

tan ϕ = 0

c = 2.12 T/sq ft

Method of saturation ______

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen UNDISTURBED

Classification PLASTIC CLAY(CH), dark gray, contains finely divided organic*

LL 114 | PL 37 | PI 77 | | G_s 2.62

Remarks *matter

Project LK. PONT., LA. & VIC.-HURR. PROT.- RIGOLETS CONTROL STRUCTURE & CLOSURE DAM,

Area DDM NO. 6

Boring No. X-15-U | Sample No. 45-B

Depth El -174.2 MSL | Date 22 July 1971

TES TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 *(EM 1110-2-1902)* PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT USAEWES SOILS Fig. A - 118