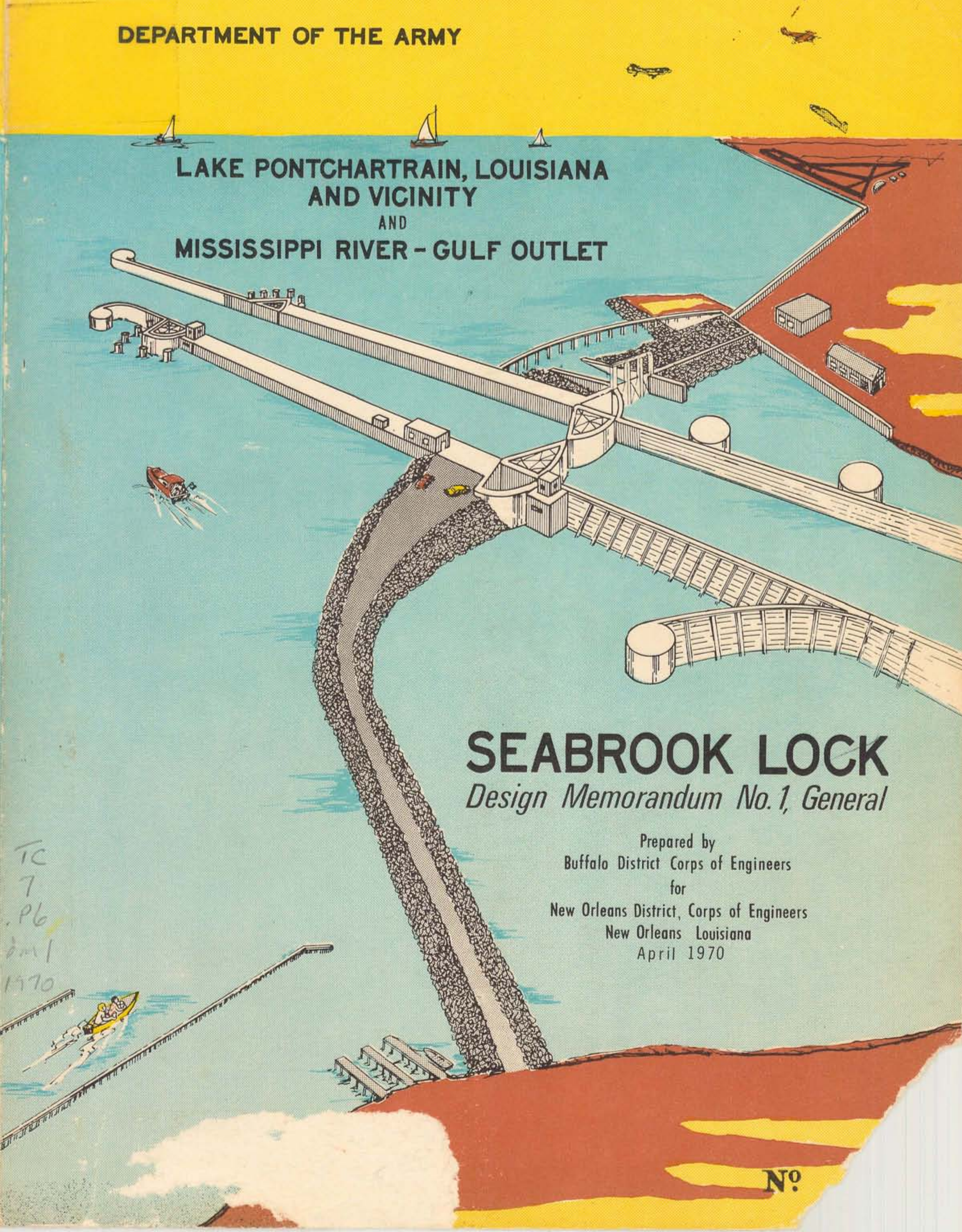
DEPARTMENT OF THE ARMY

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET



SEABROOK LOCK
Design Memorandum No. 1, General

Prepared by
Buffalo District Corps of Engineers
for
New Orleans District, Corps of Engineers
New Orleans Louisiana
April 1970

No.

DEPARTMENT OF DEFENSE
UNITED STATES OF AMERICA

DEPARTMENT OF THE ARMY
NEW ORLEANS DISTRICT, CORPS OF ENGINEERS
P. O. BOX 60267
NEW ORLEANS, LOUISIANA 70160

LMNED-PP

22 April 1970

SUBJECT: Lake Pontchartrain, Louisiana and Vicinity, and Mississippi River-Gulf Outlet, Louisiana, General Design Memorandum No. 1, Seabrook Lock

Division Engineer, Lower Mississippi Valley
ATTN: LMVED-TD

1. The subject general design memorandum, prepared by the Buffalo District, is submitted herewith for review in accordance with the provisions of ER 1110-2-1150 dated 1 July 1969.

2. Approval of this memorandum is recommended.

1 Incl (16 cys) fwd sep
GDM No. 1

HERBERT R. HAAR, JR.
Colonel, CE
District Engineer

LAKE PONTCHARTRAIN, LA. AND VICINITY
AND
MISSISSIPPI RIVER-GULF OUTLET, LOUISIANA
DESIGN MEMORANDUM NO. 1 - GENERAL
SEABROOK LOCK

STATUS OF DESIGN MEMORANDA

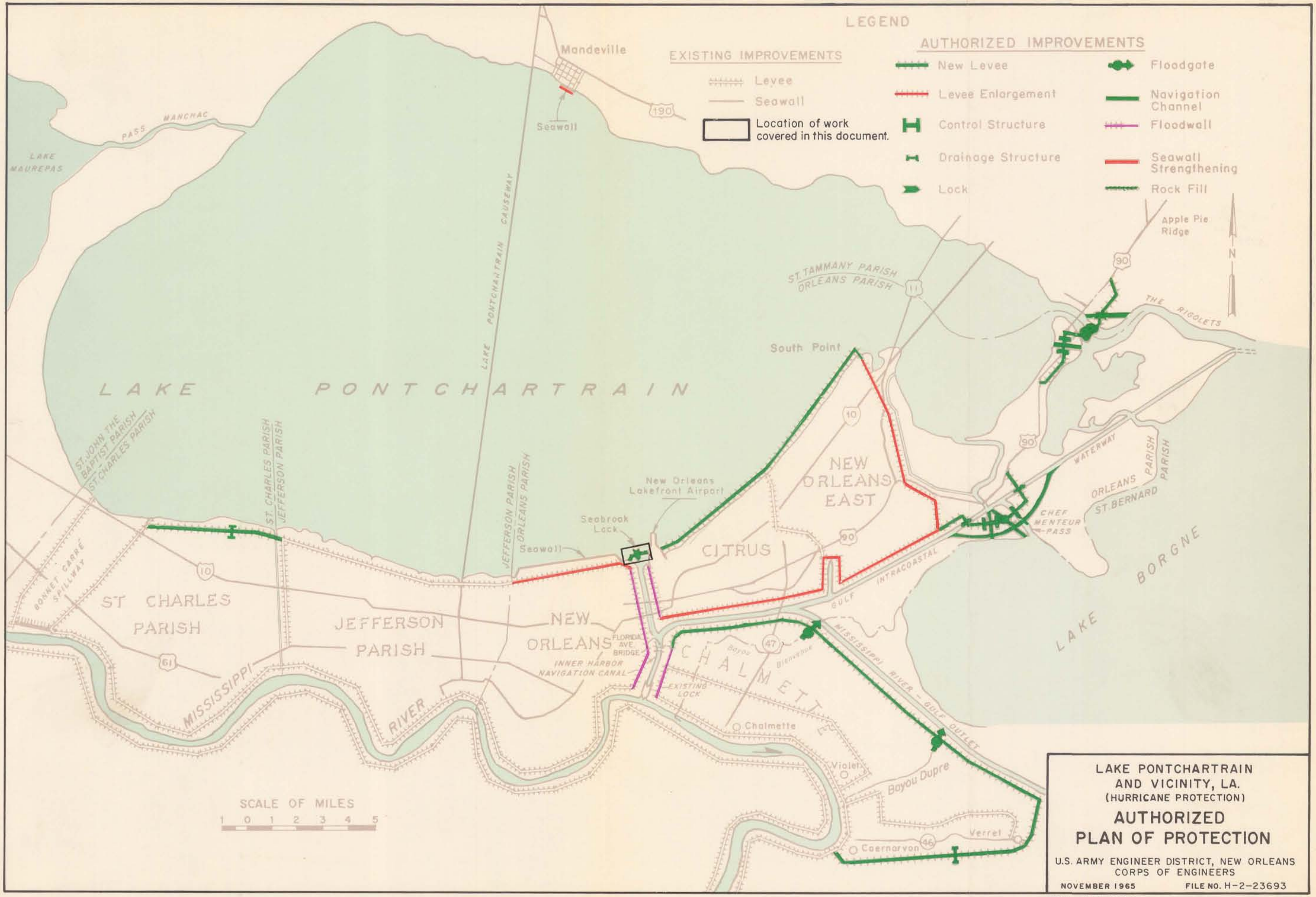
Design Memo No.	Title	Status
1	Hydrology and Hydraulic Analysis	
	Part I - Chalmette	Approved 27 Oct 66
	Part II - Barrier	Approved 18 Oct 67
	Part III - Lakeshore	Approved 6 Mar 69
	Part IV - Chalmette Extension	Approved 1 Dec 67
2	Lake Pontchartrain Barrier Plan, GDM, Advance Supplement, Inner Harbor Navigation Canal Levees	Approved 31 May 67
2	Lake Pontchartrain Barrier Plan, GDM, Citrus Back Levee	Approved 29 Dec 67
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 1, Lake Pontchartrain Barrier, Rigolets Control Structure, Closure Dam, and Adjoining Levees	Scheduled Apr 70
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 2, Lake Pontchartrain Barrier, Rigolets Lock and Adjoining Levees	Submitted 16 Jul 69
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 3, Lake Pontchartrain Barrier, Chef Menteur Pass Complex	Approved 19 Sept 69
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 4, New Orleans East Back Levees	Scheduled Jul 70
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 5, Orleans Parish Lakefront Levees - West of IHNC	Scheduled Jul 72

STATUS OF DESIGN MEMORANDA (cont'd)

Design Memo No.	Title	Status
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 5A, Orleans Parish Lakefront Levee - East of IHNC	Scheduled Aug 71
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 6, St. Charles Parish Lakefront Levees	Submitted 30 Sept 69
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 7, St. Tammany Parish, Mandeville Seawall	Scheduled Jun 71
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 8, IHNC Remaining Levees	Approved 6 Jun 68
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 9, New Orleans East Levee from South Point to GIWW	Scheduled Sept 71
3	Chalmette Area Plan, GDM	Approved 31 Jan 67
3	Chalmette Area Plan, GDM, Supplement No. 1, Chalmette Extension	Approved 12 Aug 69
4	Lake Pontchartrain Barrier Plan, and Chalmette Area Plan, GDM, Florida Avenue Complex, IHNC	Scheduled Jul 71
5	Chalmette Area Plan, DDM, Bayous Bienvenue and Dupre Control Structures	Approved 29 Oct 68
6	Lake Pontchartrain Barrier Plan, DDM, Rigolets Control Structure and Closure	Scheduled Jan 71
7	Lake Pontchartrain Barrier Plan, DDM, Chef Menteur Control Structure and Closure	Scheduled Aug 70

STATUS OF DESIGN MEMORANDA (cont'd)

Design Memo No.	Title	Status
8	Lake Pontchartrain Barrier Plan, DDM, Rigolets Lock	Scheduled Oct 70
9	Lake Pontchartrain Barrier Plan, DDM, Chef Menteur Navigation Structure	Scheduled Aug 70
10	Lake Pontchartrain Barrier Plan, Corrosion Protection	Approved 21 May 69
12	Source of Construction Materials	Approved 30 Aug 66
1	Lake Pontchartrain, La. and Vicinity, and Mississippi River-Gulf Outlet, La., GDM, Seabrook Lock	Submitted 22 Apr 70
2	Lake Pontchartrain, La. and Vicinity, and Mississippi River-Gulf Outlet, La., DDM, Seabrook Lock	Scheduled Oct 70



LAKE PONTCHARTRAIN
AND VICINITY, LA.
(HURRICANE PROTECTION)

**AUTHORIZED
PLAN OF PROTECTION**

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

NOVEMBER 1965 FILE NO. H-2-23693

REV. OCT. 1968

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
DESIGN MEMORANDUM NO. 1 - GENERAL
SEABROOK LOCK

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont.)

LIST OF PLATES

APPENDICES

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

PERTINENT DATA

Components of Seabrook Lock unit of the Lake Pontchartrain Barrier Plan	
Navigation Lock	
Rock and shell dam	
Outlet structure (thru dam)	
Lock dimensions, feet	
Width	84
Length, usable	800
Length, between pintles	860
Length, guide wall (total)	1470
Lock elevations, feet, m.s.l.	
Top of lock walls and gates	13.5
Gate sills	-15.8 (-15.0 m.l.g.)
Lock floor	-16.8 (-16.0 m.l.g.)
Type of gates	Sector
Rock and shell dam, feet, m.s.l.	
Controlling elevation (crest)	7.2
Outlet Structure	
Number of gated openings	3
Type of gates	Vertical lift
Width of each opening, feet	32
Elevations, feet, m.s.l.	
Gate sills	-15.8 (-15.0 m.l.g.)
Top of gate, fully closed	4.2
Bottom of gate, fully open	9.0
Maximum discharge capacity, c.f.s.	31,700
Hydraulic design criteria, feet	
Max. differential head, IHNC to lake	14.9
Max. reverse head, lake to IHNC	4.0
Max. navigation lift, either direction	4.0
Estimate of cost	
Federal	$14,790,000
Non-Federal	2,610,000
Total cost	$17,400,000

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
DESIGN MEMORANDUM NO. 1 - GENERAL
SEABROOK LOCK

PROJECT AUTHORIZATION

1. Authority. The Flood Control Act approved 27 October 1965 (Public Law 89-298) authorized a project for hurricane-flood protection on Lake Pontchartrain, Louisiana, substantially in accordance with the recommendations of the Chief of Engineers in House Document No. 231, 89th Congress, 1st session, except that the recommendations of the Secretary of the Army in that document shall apply with respect to the Seabrook Lock feature of the project.

2. The total project as so authorized comprises two individual plans of improvement:

a. The Lake Pontchartrain Barrier Plan, of which the Seabrook Lock unit is a part; and

b. The Chalmette Area Plan.

3. With specific respect to Seabrook Lock, the authorization provides for construction, operation and maintenance of a dual-purpose (navigation and hurricane protection) structure at the lakeward terminus of the IHNC (Inner Harbor Navigation Canal) in the vicinity of Seabrook Bridge in New Orleans, Louisiana. It contemplates that first costs for the lock be apportioned equally to the two purposes, and shared accordingly between the United States and non-Federal interests. It contemplates that annual costs for operation and maintenance of the lock be borne entirely by the United States.

4. The Secretary of the Army, in his letter dated 28 June 1965, noted that the "...Bureau (of the Budget) also discusses cost sharing for the Seabrook facility, and expresses the opinion that under existing circumstances standard methods of cost sharing are inapplicable; consequently, the viewpoint of the Bureau of the Budget is to allocate the cost of the Seabrook feature equally between navigation and hurricane protection. This allocation of costs would result in the additional cost of $687,000 to the local interests and a corresponding reduction in the cost to the United States for the Seabrook Lock. With the understanding that this apportionment of costs would not unduly delay construction, I concur in the views of the Bureau of the Budget..." As previously pointed out, the project was authorized

with the proviso that "...the recommendation of the Secretary of the Army in (House Document Numbered 231, Eighty-ninth Congress) shall apply with respect to the Seabrook Lock feature of the project..."

5. Local cooperation. The conditions of local cooperation pertinent to the Lake Pontchartrain Barrier Plan (of which the Seabrook Lock is a feature), 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 assurances 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 to the construction works;

" (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 cash 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 cash 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 eastments or other interest 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..."

6. The conditions of local cooperation pertinent to the navigation project (of which Seabrook Lock is a feature as explained in paragraph 4, above), 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:

" (1) Provide without cost to the United States and upon request of the Chief of Engineers, all lands, easements, and rights-of-way, including borrow and spoil-disposal areas, required for construction, operation, and maintenance of the project; and

" (2) Hold and save the United States free from damages due to the construction works."

INVESTIGATIONS

7. The interim survey report of the District Engineer, New Orleans District, titled "Hurricane Study of Lake Pontchartrain, Louisiana and Vicinity," dated 21 November 1962, and contained in the project document, outlined the basic protection plan recommended for Lake Pontchartrain basin, discussed the relationship between

that plan and the need for a navigation lock at Seabrook, and outlined the dual function of the lock. It contemplated a lock of the same size and in the same location as described herein.

8. Subsequent to completion of the project document studies, a special study was authorized to determine the optimun controlling elevation of the rock and shell dike at Seabrook Lock. The special study recognized the beneficial effects, as demonstrated by Hurricane Betsy in 1965, of allowing waters from the MR-GO (Mississippi River - Gulf Outlet) to flow into Lake Pontchartrain under certain storm conditions. The New Orleans District prepared a report titled, "Report on Controlling Elevation of Seabrook Lock," dated 19 October 1966, and approved by the Chief of Engineers on 12 January 1967 subject to consideration of such modifications as may be indicated by studies on the effects of the MR-GO on hurricane surges and the elevation of wind tides along the south shore of Lake Pontchartrain. Results of the latter studies revealed that modifications to the approved report were not necessary and the results of the report on the controlling elevation are discussed in appendix A.

9. Since initiation of preconstruction planning for the project, the New Orleans District has completed an investigation of the need for an outlet structure through the barrier at Seabrook Lock in mitigation of the adverse effects on riparian users that the barrier would create under certain conditions. The need for an outlet structure has been demonstrated by these studies and the results thereof, together with an analysis of the spectrum of tidal hydraulics as affected by outlet structure operating conditions, is presented in Appendix A. The Buffalo District has completed preliminary design studies directed toward determination of the types of construction to be used and selection of the arrangement and configuration of components of the Seabrook Lock unit. The Mobile District has completed preliminary design studies of cathodic protection. Results of these studies are also presented in this design memorandum.

10. A program of site investigations, including topographic surveys of the area and a series of 46 foundation borings at the lock location and along the alignment of the dam, has been completed. Laboratory analyses of samples obtained from the borings have been made to determine strength, weight, consolidation and permeability characteristics of the foundation materials. Results of these investigations are presented in Appendix D.

LOCAL COOPERATION

11. Requirements. The pertinent conditions of local cooperation as specified by the authorizing law are cited in paragraphs 5 and 6.

12. Status. 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 Rouge, 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

13. 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. The estimated non-Federal contribution applicable to the work presented herein is $2,610,000. The intention and capability of the local sponsor to provide the required non-Federal contribution has been amply demonstrated.

LOCATION OF PROJECT AND TRIBUTARY AREA

14. The MR-GO provides a deep draft navigation route from the Gulf of Mexico to New Orleans. It connects, via the GIWW (Gulf Intracoastal Waterway), with the IHNC. The latter terminates at the Mississippi River to the south and at Lake Pontchartrain to the north. Thus, direct access from the Gulf to the Lake exists, for navigation and tidal effects. The several channels, and their relationship to the Seabrook Lock site at the northerly terminus of the IHNC, are shown on accompanying plate 1.

15. The Seabrook Lock unit of the project will contribute the desired lake salinity control and hurricane barrier capability, consistent with related riparian concerns, and will eliminate excessive current velocity for safe passage between Lake Pontchartrain and the IHNC.

DATUM PLANES

16. All elevations used in this memorandum are in feet and refer to m.s.l. (mean sea level), except those noted as referring to m.l.g. (mean low gulf). Zero datum plane for m.l.g. is 0.78 feet below zero datum plane for m.s.l.

PROJECT PLAN

17. The general plan for the Seabrook Lock unit of the project is shown on accompanying plate 2. There are three basic components: the navigation lock; the rock and shell dam; and the outlet structure through the dam. Four alternate plans for the lock complex were considered. Comparative cost estimates and other considerations as discussed in Appendix C indicated the recommended plan as reflected by Plates 2 and 3 to be most desirable. Descriptions of layouts, exhibits, cost estimates and discussion are included in Appendix C.

18. The lock chamber will be 84 feet wide by 860 feet long between gate pintles. Usable length of the chamber will be about 800 feet. Gates will be of the sector type. Top of lock walls and gates will be at elevation 13.5; gate sills at elevation -15.8 (-15.0 m.l.g.); and lock floor at elevation -16.8 (-16.0 m.l.g.). The lock will be located far enough out into Lake Pontchartrain to permit navigation to bypass the site during construction and to allow construction of the outlet structure without undermining existing shoreline facilities on the east side of lock. Navigation during construction will be accomplished by maintaining a temporary channel between the land-

ward gate bay cofferdam and the existing dolphin and curved guard wall at the northwest approach to Seabrook Bridge. After construction this channel will be closed by a guide wall connecting the lock wall to the existing guard wall.

19. In addition to the three functions originally contemplated - service as a unit of closure in the Lake Pontchartrain hurricane protection barrier; control of salinity introduced to the lake during high tides; and passage of navigation during high and low tides - the lock complex will be capable of a fourth function. It will be designed so that, during hurricane-generated high stages in the MR-GO and the IHNC, the outlet structure may be opened to afford stage-damage relief for the industries along the IHNC.

20. The rock and shell dam, which will connect the lock complex with shore, will have a controlling (crest) elevation of 7.2.

21. The outlet structure, located east of the lock, will have three gated openings, each 32 feet wide. Gate sills will be at elevation -15.8 and the gates will be 20 feet high. The structure will be used to minimize stage damage due to certain hurricane-generated tidal effects, to guarantee adequate flow for riparian use, and control lake salinity.

DEPARTURES FROM PROJECT DOCUMENT PLAN

22. The project plan departs from the project document plan in the following principal repsects:

a. Lowering of the control elevation. Top elevation of the dam has been reduced from elevation 13.2 to 7.2. The basis therefor is covered in Appendix A.

b. Inclusion of the outlet structure. The need for this structure, which had no counterpart in the project document plan, is covered in Appendix A. The structure will be used to minimize stage damage due to certain hurricane-generated tidal effects, assure riparian flow requirements, and control lake salinity.

c. Raising of lock walls. In the project document plan it was contemplated that top of lock walls lakeward of the barrier be at elevation 7.2. Later re-evaluation of the considerations involved led to the conclusion that top of walls should be at least 10 feet above normal high tides to elevation 13.5 (see paragraph 8 of Appendix A).

d. Change in type of lock walls. In the project document plan it was contemplated that lock walls (between gate bays) would be cantilever structures, constructed of prestressed concrete piles. Based on preliminary studies of alternative designs, discussed subsequently in paragraph 39, it is now contemplated that the walls will be gravity structures, of parallel steel sheet pile wall construction. The change is proposed in consideration of cost savings that are expected to be realized, and the functional superiority of the parallel wall structure.

HYDROLOGY AND HYDRAULICS

23. Climatology. The climatology and hydrology for the entire Lake Pontchartrain, Louisiana and Vicinity project area were presented in Design Memorandum No. 1 - Hydrology and Hydraulic Analysis, Part I - Chalmette, approved 27 October 1966.

24. Flow regimen. The Lake Pontchartrain Barrier Plan is based upon limiting the entry of hurricane-driven waters into Lake Pontchartrain and, in order that this may be accomplished, the MR-GO--IHNC link must be controlled. The barrier plan also provides means for controlling flow from Lake Pontchartrain into the IHNC during hurricanes which produce conditions critical to the south shore of Lake Pontchartrain. Hurricane surges cause sufficient inflow into Lake Pontchartrain through the MR-GO and IHNC connection, and the Chef Menteur and Rigolets Passes to raise the lake level 4 to 6 feet, depending on the stage and duration of the surge in Breton Sound and Lake Borgne. Hurricane generated wind tides combine with these increased lake levels to produce high stages at the lock site. With Seabrook Lock and related structures at Chef Menteur and Rigolets Passes in place, hurricane overflow will increase lake levels less than 0.6 foot. Wind tides combining with the 0.6-foot rise, rainfall, and stream runoff will produce a much lower maximum stage in Lake Pontchartrain.

25. Prior to construction of the MR-GO, the salinity regimen in Lake Pontchartrain was largely controlled by the interaction between surface runoff entering it and tidal inflows from Lake Borgne via the Rigolets and Chef Menteur Pass. The 30-foot deep IHNC channel was connected to Lake Borgne by the GIWW through the Rigolets and Chef Menteur Pass, but, because of the relatively small, shallow cross section (12 feet by 125 feet) of the waterway, this connection exerted little influence on salinities in Lake Pontchartrain. Construction of the MR-GO established a large, deep (36 feet by 500 feet) direct connection with the highly saline waters of Breton Sound. Tidal flow in the MR-GO reaches Lake Pontchartrain via the IHNC,

and salinities in the lake and in the marsh adjacent to the MR-GO have increased significantly since its completion. Unless means are provided to restore a favorable salinity regimen, major damage to marine life in the lake and in the marsh traversed by the MR-GO may be anticipated.

26. A related problem deriving from the construction of the MR-GO is the generation of excessive tidal currents in the IHNC. The increased currents produce navigation difficulties and aggravate scour problems at bridges and along harbor developments.

27. An outlet structure was necessitated to assure that the flow regimen, agreed to by riparian users located along the IHNC and the U. S. Fish and Wildlife Service, is attained without interruption (see Appendix A). This structure will be installed on the east flank of the lock along the alignment of the rock dike.

28. Study of various alternatives leads to the conclusion that control of salinity in Lake Pontchartrain, and control of flow from the canal to Lake Pontchartrain during normal and hurricane periods can be best achieved by an outlet structure at Seabrook. Inasmuch as navigation between Lake Pontchartrain and the IHNC must be preserved, a lock is essential.

29. Hydraulics of lock and outlet structure. Development of hydraulic details of Seabrook Lock will be guided by EM 1110-2-1604, "Hydraulic Design, Navigation Locks," and reports on lock model studies. New model tests for the lock are not contemplated; reasonable assumptions for design of this lock can be made based on results of past model studies of locks of similar size and lift. All hydraulic criteria, including wave data, determined pertinent to the design of the lock and outlet structure are presented in Appendix A. Principal considerations are outlined below:

a. Navigation lift variations. When lock chamber velocities are less than three feet per second the gates will be held open. When they reach three feet per second, in either direction, the lock will be placed in operation. Lifts in the normal range will vary from 0 to 4 feet.

b. Design stages. The normal tidal range is about 1.0 foot in the IHNC and 0.5 foot in Lake Pontchartrain, however, south winds raising tides in Breton Sound can produce higher tides in the IHNC or northwest winds can raise the stage on the lake side of the lock. Also, hurricanes on various tracks create greater design heads for certain conditions.

(1) Elevation 13.5 was established for the lock chamber, guide walls, and gates. This is based on navigation requirements for 10 feet of freeboard above-normal high water. Further, the lock walls and gate walkways will be high enough to permit personnel to work thereon under all conditions while the lock is in operation. During brief periods at the storms peak intensity, the lock may be inaccessible by vehicle because the roadway atop the rock dike, may be submerged.

(2) Stages for normal maximum reverse loading. The normal maximum reverse head was determined to be 4 feet which resulted from stages on the Lake Pontchartrain side of 4.0 and on the IHNC side of 0.0. These stages occur under non-hurricane conditions with northwest winds blowing across the lake and will produce the normal maximum reverse loading.

(3) Stages for extreme maximum reverse loading. Studies indicated that the maximum reverse head generated by hurricanes would be 0.3 feet with stages of 7.9 in the lake and 7.6 in the IHNC for the SPH (Standard Project Hurricane) on Track A. This head, when considered in conjunction with the wave loadings that would be associated with SPH, yields a reverse loading that may be more critical for some structures than that described in the preceding paragraph.

(4) Stages for normal maximum direct loading. The proposed procedure for operation of the lock complex just prior to the arrival of a hurricane (refer to more detailed description in Appendix A) calls for the outlet structure to be opened as soon as IHNC stages adjacent to the complex reaches 3.5. This stage may be reached several times a year and may exist concurrently with a lake stage of 0.0 feet. Assuming that opening of the outlet structure gates might be unintentionally delayed for some short period, a normal direct head of 4 feet (stages assumed to be 4.0 in IHNC and 0.0 in lake) was selected as a criteria adequately encompassing normal maximum operating conditions.

(5) Stages for extreme maximum direct loading. Studies show that extreme maximum direct head across the structure will occur as a result of the SPH crossing the project area on a precise track (Track C refer to Appendix A) which produces stages in Lake Borgne of 10.5 and a coincident low stage of -7.9 in Lake Pontchartrain. Assuming the outlet structure to be fully open as soon as IHNC stage reaches 3.5 and with flow occurring through the MR-GO, the IHNC and the outlet works, the maximum concurrent stage on the IHNC side of lock would be 7.0. This produces the extreme maximum direct head of 14.9 feet.

(6) The maximum design water elevation on both sides of the lock is approximately 8.5. This condition occurs with the SPH on Track A (refer to Appendix A) and results in a stage of about 8.5 on the south shore of Lake Pontchartrain and a stage of 8.6 on the IHNC side of the lock.

(7) Maximum water levels along the IHNC will occur with the SPH on Track F (refer to Appendix A) and will vary from elevation 13.00 at Lake Borgne to elevation 8.3 on the IHNC side of lock. This condition produces a slightly lower elevation on the IHNC side of lock than that described in subparagraph (6) above, and produces a considerably lower direct head than that described in subparagraph (5), above.

c. Gates. Since the lock will be subject to reversals of head, and considering the anticipated range of heads, sector gates will be most suitable for this installation - performing the dual functions of service gates and devices for filling and emptying the lock. The rate of gate opening, with attendant acceleration of flows into or out of the lock chamber, will be of considerable importance. Chamber turbulence for various operating conditions will be investigated in detail and reported on in Design Memorandum No. 2.

d. Lock floor and approaches. Velocities in the lock chamber and over the approaches to the lock will be used to determine the size and extent of stone protection in those areas.

e. Outlet structure. Lake bottom materials in the vicinity of the lock are highly erodible. In view of this, and estimated critical exit velocities that could occur during hurricanes, it is contemplated that a stilling basin will be required lakeward of the outlet structure sill. The stilling basin as shown on plate 2 is tentative pending a model study and discussion with WES (Waterways Experiment Station). The final stilling basin configuration will be covered in detail in the detail design memorandum. Studies indicate that a similar facility will not be required on the gulfward side and that stone paving can be designed to provide protection when flow is in that direction. The outlet structure will normally be operated at a partial opening that will create constantly varying cross currents through the southeast guide wall. These currents are estimated to be about 0.1 f.p.s. for the maximum normal daily lift and about 0.2 f.p.s. during the maximum normal design lift of 4.0 feet. Such velocities are not considered excessive enough to affect navigation traversing the channel. After the outlet structure has been fully opened these cross currents could, for the most critical condition, approach 1.8 f.p.s. This is certainly undesirable but as currently planned, lockages will have ceased and the lock will have been shut down before cross current velocities of such magnitude were attained.

FOUNDATION CONDITIONS AND SEEPAGE

30. A foundation exploration program sufficiently comprehensive to support detailed design studies was completed in 1966. It included a series of borings at the project site followed by laboratory analyses and tests of samples to determine soil shear strengths, consolidation characteristics, permeability coefficients, classification and density data. Supplemental borings were taken in 1968 to more precisely define the limits of material layers. Results of these programs are presented in Appendix D.

31. The foundation at and near the lock site generally consists of a sandy layer of Recent Bay Sound deposits overlain in some places by variable Lacustrine deposits and underlain by Recent Nearshore Gulf Deposits and the uppermost Pleistocene Prairie Formation strata, both of which are predominantly clayey. The next layer beneath them, another stratum of the Pleistocene Prairie Formation, is dense sand. All identifiable layers penetrated by borings are widely variable in thickness.

32. Investigations have been made to determine the feasibility of placing reinforced concrete gate bays directly on the lake bottom or, where excessive lake bottom depths prevail, on sand or shell fill. Although this type of foundation would be most economical it does not appear to be practicable in view of the foundation conditions and estimated loadings.

a. Characteristically, the Bay Sound deposit sands are of a loose nature (standard penetration test - less than 10 blows per foot) and do not meet requirements for use in supporting a raft structure. Compaction of this submerged material so as to obtain a significant increase in density is practically impossible.

b. Differential settlement could be expected in the underlying clays with detrimental effects on the gate bays. This would be further aggravated by the influence of unequal loading caused by the rock and shell dam adjacent to the landward gate bay. Consequently, it is planned to support the gate bays and the outlet structure on piling. Because of the inherent susceptibility of the underlying clays to consolidation under load it is contemplated that the piling will extend through the clays and the long-term pile loadings distributed entirely to the lower level sand strata within the Pleistocene Prairie Formation. To insure that the foundation piles will reach and transmit to the Pleistocene Prairie sand the design long-term loadings, a series of pile tests designed to meet the foundation conditions peculiar to this site will be conducted early in the construction phase. A typical test will consist of driving a metal pipe pile through overlying materials to Zone 2 of the Pleistocene Prairie Formation; washing all material out of the pile so that an empty casing is obtained; driving the steel test pile through the casing and into the Zone 2 sands; and load testing the pile by conventional jacking methods. A test of this kind will provide reliable data concerning the load carrying capability of the stratum to which the long-term loads are intended to be transmitted. The lengths of permanent service piles will be based on information obtained from these tests.

33. The Lacustrine deposits which lie on the lake bottom (soft to very soft silt and clay) are considered inadequate for use as foundation material due to their extremely high water contents and significant quantities of decaying organic matter. They will be removed wherever they occur beneath a gravity structure.

34. The upper strata of foundation materials are sufficiently permeable to require that, where hydraulic head differentials will some day exist, cutoff walls must be placed through them and extended to the underlying impervious clayey layers. During detailed design of the lock the permeability data already obtained in the laboratory will be utilized to develop a construction dewatering system for each gate bay area. Computations indicate that a permanent pressure relief system will not be required under the gate bays or under the outlet structure.

OPERATION OF PROPOSED STRUCTURES

35. Operation of lock and outlet structure. The proposed operating procedure for the Seabrook Lock and Outlet Structure is essentially as follows:

a. The lock will normally be left open with no lockages for current velocities up to 3 feet per second in the lock chamber. Navigation through the lock during this period will be controlled by the lockmaster because of obvious limitations in the channel width.

b. When velocities through the lock chamber fall within the range of 3 feet per second to 6 feet per second, the lock gates will be left open unless a vessel approaches - in which case the gates will be closed and a normal lockage operation accomplished. As described in Appendix A, this condition will exist for about seven hours on an average day.

c. When velocities through the lock chamber reach 6 feet per second (this will occur when there is about a 1 foot differential between the IHNC and lake), the lock gates will be closed and operated only as required for lockage operations.

d. Because of the flow interruption that will be experienced by closing the lock gates as described in preceding paragraphs b and c (refer to Appendix A for detailed discussion of this subject) the outlet structure will be continuously operated at a programmed partial opening that will assure flow interchanges in accordance with the control regimen agreed to with local interests.

e. If a hurricane enters the Gulf of Mexico the outlet structure and the lock will be closed to prevent storm water intrusion into Lake Pontchartrain for the few days preceding arrival of the storm. During this period stages in the IHNC can be expected to gradually rise so that, if these structures were not closed, continuous storm water intrusion would raise lake levels significantly. When IHNC stage reaches 3.5, the outlet structure will be fully opened. With the outlet structure open, flow from IHNC to the lake will increase and stages along the IHNC will be relieved. Refer to Appendix A for more detailed discussion of proposed operating procedures during hurricanes and for projected water surface profiles under various hurricane conditions.

36. Design of the lock, dam and outlet structure will be based on hydraulic conditions that would prevail with proper operation of structures under the anticipated conditions outlined in paragraph 29, "Hydraulics of lock and outlet structure." Should incorrect operation occur, the following consequences, as they affect safety of the structures involved, are considered to be possible:

a. If lock gates are not closed when velocity reaches 3 feet per second (a possibility that will arise daily): Problems to navigation might result but the structures would not endangered. Lock floor and approaches will be protected against erosion from the normal maximum daily velocity of 6 feet per second that would occur if there were no occasion to close the gate (i.e. if no vessels were to approach the lock).

b. If the normal amount of opening in the outlet structure is incorrect: Structures would in no way be endangered.

c. If the lock gates are not closed when IHNC stage reaches 3.5 feet or when velocity through the lock would exceed 6 feet per second (possibilities that will predictably arise several times each year): Depending on the ultimate severity of storm, erosion of the lock floor and approaches, undermining of chamber walls and gate bays, and undermining of guide walls could be expected. If the storm were to approach SPH intensity a hydraulic jump in or near the lock might occur and lock chamber walls might be lost.

d. If outlet structure gates are not opened when IHNC stage reaches 3.5 feet, (a possibility that may arise several times a year) consequences depend on storm severity. If SPH intensity were reached the IHNC stage would raise to about the profiles indicated on plates in Appendix A and head on the structures would be severely increased. Total failure due to instability of structures would not be expected but lock gates would be overstressed and outlet structure gates would rupture. Stage damage along the IHNC would occur prior to failure of the outlet gates.

DESIGN CRITERIA

37. Hydraulic criteria for structure design. Hydraulic conditions for which the structures will be designed are based on the stages described in paragraph 29, "Hydraulics of lock and outlet structures." These are:

a. Extreme maximum direct loading due to a 14.9 foot direct head with IHNC at elevation 7.0, lake elevation -7.9, lock gates closed, outlet structure gates open and unrestricted flow through the outlet structure (refer to paragraph 29b (5)). Above-normal design stresses will be used. Lock gates will not be operated under this loading.

b. Normal maximum direct loading is assumed to be a head of 4 feet. Stages of 4.0 in the IHNC and 0.0 in the lake would produce this head (refer to paragraph 29b (4)). Normal design stresses will be used except that above-normal stresses will be permitted when this hydraulic loading is combined with assumed boat-impact loads.

c. Extreme maximum reverse loading is with IHNC at elevation 7.6, lake at elevation 7.9 and waves acting on lakeward exposures (Refer to paragraph 29b (3)). Above-normal design stresses will be used. Lock gates will not be operated under this loading.

d. Normal maximum reverse loading is with IHNC at elevation 0.0 and lake at elevation 4.0 (Refer to paragraph 29b (2)). Normal design stresses will be used except that above-normal stresses will be permitted when this hydraulic loading is combined with boat-impact loads.

e. For purposes of design of needle girders to be infrequently used for maintenance dewatering a water elevation of +5.0 will be used. Normal design stresses will be used.

f. For purposes of design of the construction cofferdam a lake stage and wave loading produced by the 25 year frequency storm is assumed. Lake stage for this condition is 7.4 and significant wave height is 6.8. Above-normal design stresses will be used. Cofferdams will be constructed to a top elevation of 8.0.

g. For purposes of design of the lock chamber walls a partial unwatering of the chamber, to elevation -4.0 for possible repainting of the splash zone along the chamber wall face, will be assumed. Concurrent water elevation outside the lock will be assumed at elevation 5.0 as described in "e" above. Above-normal stresses will be used.

38. Structures will be designed for hawser pull or boat impact, as appropriate, in accordance with the following schedule:

a. Guide and guard walls. 1,000 pounds per foot of wall applied in either direction.

b. Sector gates. 120,000 pound boat impact applied as a concentrated load, except that skin plate and intercoastals will not be designed for such load.

c. Lock chamber walls. 1,000 pounds per foot of wall due to vessel inside the chamber and applied in either direction, or 3,000 pounds per foot of wall due to vessel outside the chamber and applied as an impact loading under storm conditions. The value of 3,000 pounds per foot was arbitrarily selected on the basis that boat load on outside of wall would, in all probability, be caused by an angle of vessel incidence much greater than would be anticipated from a vessel within the chamber.

DESCRIPTION OF PROPOSED STRUCTURES AND IMPROVEMENTS

39. Lock walls. Preliminary designs and estimates were made for four alternative lock chamber walls which could, in each case, be constructed in the wet. The four types of wall considered were:

a. Cantilever type constructed of prestressed concrete cylinder piles with concrete filler between piles;

b. A cantilever type similar to that described in the preceding subparagraph except that a reinforced, cast-in-place, concrete super-structure would be used in lieu of the portions of the cylinder piles that would project above water surface;

c. Gravity type, filled, parallel steel sheet pile walls with tie rods and wales;

d. Gravity type, filled, cellular steel sheet pile walls.

Because of the corrosive nature of the water at this site all walls in which steel would comprise an exposed structural element were assumed to require sand blasting followed by a zinc-rich primer and coal tar epoxy coatings plus an independent impressed current type cathodic protection system.

40. Comparative estimates indicate that the parallel steel sheet pile wall would be the most economical by a narrow margin. Descriptions, work sketches and cost estimates for each type of wall are included in Appendix C.

41. General features of the proposed wall are shown on plate 3. It will consist of two rows of sheet piling tied together with a wale and tie rod system. It will be filled with clam shells and will be fendered on both sides. Riprap fill was considered because of its inherently greater ability to withstand vessel impact. It was not selected because the foundation material does not have adequate strength to withstand the heavier loads riprap would impose. All steel components will be protected from corrosion. Shell fill, protected from scour by concrete or riprap, will be placed on both sides of the wall to enhance its stability. The sheet pile rows will extend down through the more pervious sandy layers into the underlying clays to form a cutoff wall to provide more substantial toe anchorage, and to more fully protect against possible undermining by erosion. The walls will not be designed for stability in the totally unwatered condition as unwatering of the chamber after construction is not anticipated. Instrumentation for the chamber walls will not be provided.

42. Gate bays and gate bay construction methods. The gate bays will be conventionally shaped reinforced concrete frames with tops at elevation 13.5 and sills at elevation -15.8 (-15.0 m.l.g.) Each gate bay will be designed as a monolithic unit, supported by piles, with pile load intensity determined by analyzing the relationship of each pile to the pile group and to the resultant of all forces acting thereon using the method of elastic center as given in Andersen's "Substructure Analysis and Design," second

Edition, 1948. As discussed in paragraph 32, the selection of piles as the supporting medium is based on the proximity of underlying clay layers and on the impracticability of preparing, under water, a suitable raft type soil foundation. Appendix C contains a brief description of four types of bearing piles that have been investigated, and comparative estimates of cost therefor. Based on the investigations made, steel piles have been selected for this project. The 14 BP 73 piles will be designed for 86 and 114 ton capacities under the normal and above-normal stress conditions, respectively, described in paragraph 37.

43. Each gate bay will be approximately eighty-six feet long and about one hundred ninety feet wide. Recesses will be provided for needle beams and girder on each end for use in future unwatering work. Provision will be made at top of wall for gate machinery and control houses. Instrumentation will consist of piezometers for measurement of uplift beneath the base slab and reference monuments for measurement of vertical and horizontal movement of the concrete.

44. Two methods of constructing and placing the gate bays have been considered. The first is based on construction of the lower portion of each gate bay at a dewaterable shoreline site, placing needle girders and beams to form a floatable chamber, then towing the floating gate bays to the lock site and sinking them to their precise final positions much as has been done in the closing of tidal estuaries in the Netherlands. Top portions of the bays would then be constructed and installation of gates and machinery accomplished. The second construction method considered was conventional on-site construction within unwatered cofferdams. The first method, though of great interest because of possible potential savings, was found to be expensive because the only available site - about twenty miles away - would require that a channel of about 20 feet draft be dredged completely across Lake Pontchartrain. Further, as studies of the two alternatives progressed it became increasingly evident that methods of securely connecting a precast gate bay to the tops of bearing piles, particularly in an underwater environment, could not be considered functionally equal to a cast-in-place connection and, in fact, would be of doubtful dependability. For these reasons it was decided to proceed on the basis that a conventional construction plan within cofferdams would be pursued.

45. Guide and guard walls. Guide walls will be constructed of timber pile bents connected with a wooden walkway and fendered on the channel side. They will form straight extensions of both ends of the easterly lock wall and the south end of the westerly

lock wall. A curved guard wall at the northerly end of the west lock wall will assist vessel alignment prior to entry into the chamber. The guide and guard walls at the north end of the lock will terminate at steel sheet pile dolphins. Tops of the dolphins and the guide and guard wall walkways will be at elevation 13.5.

46. Dewatering system. Each end of each gate bay will be provided with recesses to receive a needle support girder and vertical needle beams. The support girders will be designed for this lock; the required needles will, when needed, be drawn from storage at other locks in the New Orleans District. Only two girders will be furnished; thus, only one gate bay may be dewatered at a time. The dewatering will be accomplished with portable pumps brought to the site when an occasion for dewatering arises. The needle support girders will be stored on pads installed on top of the easterly chamber wall.

47. A similar dewatering system is planned for the sill area of the outlet structure discussed in paragraph 53. It, too, will have two girders so that only one gate area can be dewatered at a time using portable pumps. Because the sill elevation of -15.8 (-15.0 m.l.g.) is the same as for the sector gate bays it will be possible to use the same needle beams as described in preceding paragraphs. The needle support girders will be stored on pads installed on top of the easterly lock chamber wall.

48. Sector gates. Radial gates of the sector type will be used. Each leaf will be framed from curved skin plate and curved horizontal ribs supported by equally spaced vertical girders which, in turn, will be supported by a truss system that conveys all loads to the hinge and pintle. Because of problems recently encountered in the opening of similarly framed gates under reverse head conditions, consideration was given to possible modifications to skin plate nosing at the miter end and/or modification of the framing itself. However, because of the relatively low reverse head on this project (4.0 feet) it was concluded that the most suitable solution would be to retain the planned framing and to design the gate and its operating machinery for the loads predicted by interpolating results recently obtained from model tests at WES. Gates will have a top elevation of 13.5 to match adjacent walls and can be removed and replaced by floating plant without dewatering. Gates will be designed for combinations of dead load, boat load and the water loads described in paragraph 37. The sector gates will be painted with a cold tar epoxy coating. Cathodic protection will be provided as described in paragraph 65 following.

49. Sector gate machinery. The gate machinery will consist of an electric motor driven hydraulic pump unit, oil reservoir, valves and accessories, piping, hydraulic motor, electrically-operated brake, speed reducer, angle drive unit, idler gear, rack and limit switch. The rack will be mounted on the sector gate and the angle drive unit will be mounted in an open recess in the sector gate bay so that the idler gear will engage the rack to operate the sector gate. All other equipment will be located in an adjacent watertight recess. A floating shaft will extend from the angle drive unit through a watertight bulkhead to the speed reducer. The speed reducer, brake and hydraulic motor will be mounted on a common machinery base and the electric motor hydraulic pumping unit, valves and accessories will be mounted on the oil reservoir near the speed reducer assembly. The hydraulic pumping unit will consist of a positive-displacement, variable-delivery main pump with an integral positive-displacement, constant-delivery auxiliary pump. The main pump will be provided with electric remote operated control for selecting four preset adjustable volumes. Selection of the preset adjustable volumes will be accomplished through operation of four solenoid-operated pilot valves or four relays operating potentiometer units which control the operation of the servo-system. The auxiliary pump will provide a positive source of pressure for operation of the servo-system. The limit switch will be driven by a speed changer which will be connected to the reducer output shaft. The pumping unit will drive the hydraulic motor at a fast speed to operate the gate at approximately 20 feet per minute for a 3 minute cycle and at a slow speed of approximately 4 feet per minute for slow operations at the beginning and end of a cycle or when otherwise required or desired. Two intermediate speeds also will be provided to give additional flexibility of speed control for filling and emptying of the lock. As alluded to in the preceeding paragraph, machinery design will take into account reverse head loadings from recent model tests conducted by WES.

50. Control houses. One story control houses to shelter control desks and the control panels will be constructed on the westerly side of each gate bay. Floor elevations will be coincident with top of lock walls, 13.5, which is above maximum high water. Both sector leaves of both gates will be controllable from each house.

51. Approach channels. Existing depths and channel widths are adequate for navigation needs during and after construction. No approach channel excavation will be required. As described in paragraph 29.e, stone protection for the approach channels will be designed to resist erosion due to the highest anticipated velocities.

52. Rock and shell dam. Top elevation of the dam, which is the controlling elevation of the lock unit, has been established at 7.2 as discussed in Appendix A. The dam will provide access to the lock from the west shore. It will be constructed of a clam shell core with riprap and derrick stone protection designed in accordance with the methods outlined in the U. S. Army Coastal Engineering Research Center's Technical Report No. 4 titled "Shore Protection Planning and Design," to withstand wave forces. A steel sheet pile cutoff wall, extending from top of dam to the underlying clayey Nearshore Gulf Deposits, will connect from shore to shore and will be continuous through portions of the dam and under the outlet structure and the gate bay. Concrete sheet piles with plastic interlocks were considered for this application but were not considered suitable because of the anticipated difficulty in driving them through the deep layer of shell fill. A shell-surfaced roadway access ramp to the west lock wall and a parking area adjacent to the gate bay will be provided. Location of these features and the expected cross section of the dam are shown on plates 2 and 3. Piezometric instrumentation of the dam is not planned.

53. Outlet structure. Appendix A establishes the need for an outlet structure and describes the planned operational guidelines that will apply under normal as well as hurricane conditions. The structure will have three gated openings each 32 feet wide with sill at elevation -15.8. Piers will separate the gates, support the gate operating machinery, and support a light concrete footbridge on which the machinery line shaft will rest. The bridge will be designed for dead load plus an assumed live load of 100 p.s.f., and will be checked for 150 m.p.h. wind without concurrent live load. As described in paragraph 29.e, a stilling basin will comprise the lakeward end of structure for control of the hydraulic jump that could occur during high discharges. Two steel needle girders will be provided so that, by obtaining concrete needle beams from other locks in the District, future emergency unwatering of a gate sill area would be feasible. Stairs will be installed on the westerly end pier and the easterly side of the landward gate bay to permit personnel access from top of lock, elevation 13.5, down to top of dam, elevation 7.2 and to top of pier and footbridge, elevation 33.0. No means of access will be provided between the easterly end pier and the east shore. Instrumentation will consist of piezometers in the sill and reference monuments on the pier portion of the structure. None will be provided in the stilling basin.

54. Outlet structure gates will be the vertical lift type, wheeled, with horizontal girders and vertical diaphragms supporting a flat skin plate. Seals will not be used because leakage of water in either direction is of little consequence. Cathodic protection will be provided.

55. The lift gates will be raised by means of individual, geared, electric motor driven wire rope, hoists. Each hoist will have a helically grooved cylindrical hoist drum mounted on the pier at each end of a gate. Operation of each gate will be accomplished by the motor on the adjacent right pier. Power will be transmitted across the spillway bay between piers with a medium speed solid line shaft supported on bearing pillow blocks. Wire rope will be of the general purpose type, 6 x 19, improved plow steel, with IWRC. Hoist machinery will be designed to raise or lower the gates at a normal speed of about one foot per minute. The drive end unit of the hoists will be provided with a brake to hold the load in any position with the power off, and a traveling nut or rotating-cam limit switch to control gate position. Dogging devices will be provided to hold the gates in an open position when the machinery is disconnected for removal of the gates.

56. Operations and maintenance buildings. Operations and maintenance buildings will consist of a 6' x 8' oil and paint storage building and a 20' x 60' main building, both located on the west lock wall. The main building will be partitioned for radio room, lockmaster's office, toilet, storeroom, entry, locker room, work room and an equipment room housing the standby generator. Approximate locations of buildings are shown on plate 2.

57. Protective structures. Exposure of the lock chamber walls and lakeward gate bay to the open lake renders them vulnerable to damage from drifting or uncontrolled vessels particularly during storms. The lock chamber walls will be designed to resist resulting impact loads, assumed to be 3,000 pounds per linear foot in intensity during storm attack, but the lakeward gate bay cannot practicably be designed to resist such forces. Consequently, protective dolphin clusters consisting of a 3-unit king pile with 9 peripheral battered piles will be placed as shown on plate 2 to intercept any potentially damaging craft. Other dolphins of steel sheet pile construction will be placed at the end of the lakeward guide and guard walls, as shown on plate 2, so as to protect those structures from collision damage if approaching tows are improperly positioned.

58. Fire Protection. A 200 gpm electric-motor-driven fire pump with a minimum pressure rating of 100 psi will be provided. Hose outlets and hose reels will be provided in accordance with EM 1110-2-2608.

59. Potable water. Potable water is available from the west side of the lock. Connection to the existing city water main will be made and potable water will be piped across the dam to the operation and maintenance buildings on the west lock wall.

60. Sewage Treatment. Sewage will be disposed of through a direct connection to the city sewer system. The new sewer will be run from the toilet room in the main operations and maintenance building, down the access ramp, along the westerly portion of the dam and then to the nearest point in the city lines.

61. Natural Gas System. A natural gas system will be provided for supplying gas fuel to the engine generator set and space heaters in the main operations and maintenance building. Natural gas is available from the west side of the lock. Connection to the existing utility gas main will be made and gas will be piped across the dam to the operations and maintenance building on the west lock wall.

62. Electrical. The lock will be electrically operated from the two control houses, one at each end of the lock (see paragraph 50). The houses will be electrically heated. Power will be obtained at the voltage supplied by the local utility, and transformed as required for the proper utilization voltages. Switchboards, of the motor-control center type, containing circuit breakers, control items for the sector gates, lights, and other circuits will be installed at the most practical locations, probably in each control house. Lock gates will be controlled through manually operated switches located on consoles in each control house. Each gate will be controllable from each of the two consoles. To aid in trouble shooting, each gate circuit will contain switches to completely isolate the several control loops from the balance of the circuit.

63. Vertical lift gates in the outlet structure will be electrically operated and controlled. Each of the three hoist units will be operable from a control panel in the landward lock control house. When IHNC stage reaches 3.5 the lift gates can be operated to open fully without requiring an operator to mount the piers and travel the footbridge. A control-transfer switch will be provided at each hoist so that control can be moved from the central panel. Local control of any one hoist will be used for try-out, testing, and for normal minor variation of partial gate opening where it is desirable to have a direct visual indication of gate position.

64. Incandescent lighting will be used throughout the lock and outlet structure to minimize illumination loss due to momentary outages. A gas engine driven generator, sized to operate resistance heaters and lights simultaneously with either the sector gate machinery, or the lift gate machinery, will be installed for emergency use. For use in maintenance and testing, a sound

powered, common-talking, non-ringing test telephone circuit, with at least two plug-in hand telephones, will be available with jack boxes located wherever control and/or power conductors terminate. Permanent mounted, sound powered telephones shall be installed in both control houses and in the O&M building for normal communication purposes.

65. Corrosion control system. Water quality data obtained at the Lake Pontchartrain connection with the IHNC shows chloride concentrations varying between approximate limits of 290 and 12,000 parts per million. The maximum corrosion rate for submerged ferrous structures occurs in an environment having a chloride concentration of approximately 18,000 parts per million. On this basis, local-corrosion-cell action will proceed with great speed, and severe corrosion damage to an exposed, non-protected steel structure would result. To prevent corrosion of the steel lock gate structures, the sheet-piling walls, the wales, tie rods, tie-back walls, stay bolts and stay plates, an impressed current cathodic protection system will be installed. Since the vertical lift gates associated with the outlet structure can be lifted free of the water for periodic maintenance, protection against corrosion will be provided by the installation of a system of sacrificial anodes.

a. Lock Gates. Protection for the lock gates will be provided by a system of cable-supported Durichlor-51 Type "G" 2 inch x 9 inch anodes suspended from insulated clevises. Anode strings on the exterior and interior of the skin plate will be suspended within an anode guide consisting of 4-inch diameter, Schedule 80, rigid-plastic pipe fastened to the skin plate or ribs by means of formed, flat-iron brackets, welded to the structure. Cutouts, (or windows), will be provided in the anode guide at each anode to permit exit of the protective current. Anode strings will be capable of being removed from the top of the gate structure for inspection or replacement. Four strings of 5 anodes each will be provided on the exterior and interior of the skin plate. Protection of the main gate framing and the fender supports will be provided by 8 free-handing strings of four anodes each. Strings will be supported from insulated clevises, and insulated guide brackets will be provided at intermediate frame members. An eyebolt will be installed at the bottom frame member, and a nylon-rope stay line will extend from the bottom anode, through the eye-bolt and up to a snap fastener at the clevis. This stay line will preclude any movement of the free-hanging anode string during water turbulence and will assist in removal and replacement of the anode string for inspection and replacement. A terminal and resistor cabinet will be provided on the top of the gate, in a location accessible to the walkway, for connection of the anode strings to the header cable, and to provide a means of readily inserting control resistors, if necessary. Each anode string cable will be continuous, without splices, from the anodes to the terminal and resistor cabinet. Anode leads on the top of the gate structure will be installed in Type 40 rigid plastic conduit.

b. Lock Walls. Sheet piles will be bonded together, electrically, after driving by means of a No. 6 plain steel reinforcing rod welded to the pile sheets at the top. Bonds will be provided between pile sheets, wale channels, tie rods and stay bolts, to assure electrical conductivity. Protection for the exterior faces of the walls exposed to the water of Lake Pontchartrain, and to the water in the lock chamber, will be provided by a system of suspended anode strings installed in Schedule 80 plastic pipe guides, similar to those used on the sector gate skin plate. Approximately forty strings of 4 - 2" x 9" anodes will be installed at intervals of approximately 20' on each exterior face of each sheet-pile wall. Anode strings may be removed from the top of the lock wall, for inspection or repairs. Because they will be suspended within the configuration of the "Z" pile sheets the anode strings and guides will be protected from mechanical damage. Protection for the internal faces of the pile sheets, the wale channels, tie rods, stay bolts, and interior tie-back walls which will be exposed to water-saturated-shell fill, will be provided by forty 3" x 60" Durichlor-51 Type "E" anodes placed on the undisturbed lake bottom, midway between the pile sheets. Individual leads will be brought to the top of the wall in 1/2-inch plastic conduit for mechanical protection. Connection to the header cable will be made in conduit fittings suitable for insertion of balancing resistors, if required, between individual anode lead and the header cable.

c. Rectifiers. Four dual rectifiers, each having two separately adjustable d.c. outputs, and suitable for outdoor installation will be provided. Each rectifier will serve one gate leaf, and one-half of one chamber wall. Rectifiers have been sized on the basis of supplying 0.5 milliampere per square foot of painted surface. Ample allowance has been made for increased current requirements, as the paint deteriorates or may become damaged.

d. Adjustment of system. Prior to placing the impressed current cathodic protection system in operation, a complete preprotection survey will be made of the lock gates and lock walls. Structure-to-reference cell potentials will be measured at selected locations on each gate and at intervals along the outside and inside of the lock walls. Perforated plastic drain-pipe test wells will be provided in the clamshell ballast to enable measurements to be made of the lock wall interior faces. Measurements of potential will be made using a copper-copper sulphate reference cell and a corrosion voltmeter having an internal resistance of at least 200,000 ohms per volt, or a potentiometer voltmeter. After the pre-protection survey has been completed, the entire system will be placed in operation and the system voltages adjusted to give a maximum structure-to-reference cell potential over the entire area of approximately -1.00 volts, with protective current "on." After the system has operated for a period of several weeks, to permit initial polarization of the structure, the installation will be resurveyed and measurements of potential taken with current "on and "off." The system voltages will be adjusted to provide potential readings, with current "off," of 0.22 to 0.30 volts more negative than the pre-protection potentials. Periodic adjustive surveys will be made until stable operating conditions are obtained. Reports of the surveys will be forwarded, through channels, to the Office, Chief of Engineers, in accordance with requirements of EM 1110-2-3701 dated 15 May 1962.

e. Outlet Structure Gates. A sacrificial anode system was chosen for these gates. Zinc was selected as the anode material, due to the low solution potential of the metal. A zinc anode system will not disturb the coal tar epoxy coating system by the evolution of hydrogen at holidays in the coating. A system of 24 pound zinc anodes will be installed for protection of the skin plate of each gate. Each bay of the open side of each gate will be protected by means of one or two, 24 pound anodes installed within the physical boundaries of each bay (see plate 6). Ends of the gates, wheels, and guides will be protected by anodes located on the end beams.

66. Access roads. Since the tie-ins for the rock dike abut existing public roads, no access roads will have to be constructed. Local interests will bear the responsibility of providing permanent easements for all public roads used for access both during and after construction.

CONSTRUCTION MATERIALS

67. Sources. An investigation of sources of construction materials has been completed and results thereof are presented in "Lake Pontchartrain, Louisiana and Vicinity, Design Memorandum No. 12, Sources of Construction Materials" approved 30 August 1966.

REAL ESTATE REQUIREMENTS

68. The Seabrook Complex is essentially located in the lake. Lands necessary for permanent rights-of-way are limited to a small area at west tie-in of the rock dike with the existing shoreline. Since

land in this area is very expensive, its value is appraised on a square-foot basis. The lands required are estimated as follows:

5,640 sq. ft. unpaved	@ $1.25/sq.ft. =	$ 7,050
1,020 sq. ft. paved	@ $2.25 sq. ft. =	2,295
Subtotal		$ 9,345
Contingencies 20%		1,855
Total		$11,200

Because access roads already exist in the project area, permanent easements needed for access purposes will be acquired by usage permits issued by local interests.

ENVIRONMENTAL QUALITY

69. Since the project area has essentially no land, it does not lend itself to planting for screening or baffling effects. The low design elevation and general location of the structures will reduce their intrusion on the general esthetic values of the area. Structural components and buildings on the lock wall and gate bays will be painted to blend with each other and the surrounding environment. Special design of the entrance gates to the lock site and other small features will be incorporated into the final design to add to the scenic value of the area. A boat-launching facility constructed by the Orleans Levee District and located adjacent to and on the west side of the Complex contributes to the recreational quality of the area.

COST ESTIMATES

70. Estimate of first costs. The estimated cost for construction of the Seabrook Lock complex in accordance with the project plan presented in this design memorandum is given in table 3, following. The estimate is based on July 1969 price levels. For division of the estimated costs between Federal and non-Federal interests, see table 4 in paragraph 71.

TABLE 3
ESTIMATE OF FIRST COSTS

Line Item	Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
					$	$
01		LANDS AND DAMAGES				
	1.	Land Cost				
		(a) Unpaved	5,640	S.F.	$ 1.25	$ 7,050
		(b) Paved	1,020	S.F.	2.25	2,295
05		LOCK				
	2.	Excavation-Lacustrine Deposits	210,000	C.Y.	1.25	262,500
	3.	Shell Fill	277,000	C.Y.	3.45	955,650
	4.	Cofferdams				
		(a) Steel Sheet Piling S-28 (24 Mo. Rental)	5,696	Ton	176.00	1,002,496
		(b) Set, Drive & Pull	590,070	S.F.	3.10	1,829,217
		(c) Shell Cell Fill	123,650	C.Y.	4.60	568,790
	5.	Dewatering		L.S.		530,000
	6.	Excavations - Structure	31,150	C.Y.	2.10	65,415
	7.	Riprap Stone - in Wet	43,100	Ton	10.50	452,550
	8.	Derrick Stone				
		(a) Stone in Dry	19,100	Ton	13.65	260,715
		(b) Stone in Wet	33,200	Ton	10.50	348,600
	9.	Rock Spalls	22,000	Ton	8.95	196,900
	10.	Lock Chamber Walls				
		(a) Plain Steel Sheet Piling Z-27	212,200	S.F.	5.35	1,135,270
		(b) Fabricated Piling	2,100	S.F.	7.95	16,695
		(c) Metal Work	264,000	Lb.	0.37	97,680
		(d) Shell Fill	55,500	C.Y.	3.90	216,450
		(e) Timber Fenders	12,360	L.F.	10.00	123,600
	11.	Guide and Guard Walls				
		(a) Treated Timber "Marine Piling"	50,200	L.F.	4.00	200,800
		(b) Treated Timber "Framing & Planking"	81	MBFM	840.00	68,040
		(c) Timber Fenders	5,600	L.F.	10.00	56,000
	12.	Mooring Dolphins				
		(a) Plain Steel Sheet Piling S-28	10,100	S.F.	5.50	55,550
		(b) Shell Cell Fill	2,100	C.Y.	3.90	8,190
		(c) Timber Pile Clusters "Marine Piling"	12	Ea.	3,000.00	36,000
	13.	Steel Bearing Piles				
		(a) 14BP73	28,200	L.F.	9.50	267,900
		(b) 12BP53	1,800	L.F.	7.45	13,410
	14.	Pile Loading Test	4	Ea.	3,045.00	12,180

TABLE 3 (Cont.)

Line Item	Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
					$	$
	15.	Concrete				
		(a) Gate Bays				
		(1) Slab	5,180	C.Y.	$ 45.00	$ 233,100
		(2) Walls	3,600	C.Y.	75.00	270,000
		(b) Outlet Structure & Stilling Basin				
		(1) Slab	3,100	C.Y.	45.00	139,500
		(2) Walls	2,100	C.Y.	75.00	157,500
		(c) Retaining Walls	140	C.Y.	75.00	10,500
	16.	Portland Cement	19,400	Bbl.	5.50	106,700
	17.	Steel Reinforcement	2,111,000	Lb.	0.16	337,760
	18.	Cut Off Walls-Steel Sheet Piling MA-22	69,350	S.F.	4.75	329,412
	19.	Sector Gates 788,000 Lbs. - Steel (Including Painting & Embedded Metals)		L.S.		604,000
	20.	Vertical Lift Gates (123,000 Lbs. Gate (Including Painting & Embedded Metals)		L.S.		93,000
	21.	Needle Girders	64,000	Lbs.	0.45	28,800
	22.	Miscellaneous Metal Work		L.S.		63,000
	23.	Cathodic Protection System		L.S.		107,200
	24.	Sandblasting & Coaltar-Epoxy Painting Steel Surface	810,360	S.F.	0.65	526,734
	25.	Control Houses	2	Ea.	3,700.00	7,400
	26.	Operating Machinery		L.S.		310,000
	27.	Engine Generator Set	1	Ea.	12,000.00	12,000
	28.	Electrical Work		L.S.		282,000
	29.	Water Distribution & Fire Protection System		L.S.		32,000
	30.	Sewage System		L.S.		20,000
	31.	Natural Gas System		L.S.		24,000
	32.	Foot Bridges	3	Span	1,800.00	5,400
	33.	Operations Building		L.S.		32,000
	34.	Paint Buildings		L.S.		1,600
		SUBTOTAL				12,523,549
		Contingencies @ 20% ±				2,476,451
		Total Estimated Contractors Earnings Plus Contingencies				15,000,000
30		ENGINEERING & DESIGN 8% ±				1,200,000
31		SUPERVISION & ADMINISTRATION 8% ±				1,200,000
		TOTAL ESTIMATED FIRST COST				$17,400,000

71. Comparison of estimates. A comparison between the estimate of cost presented in this design memorandum and previous estimates for the Seabrook Lock unit is provided in table 4, following (price levels of the several estimates are shown in parenthesis). In each case, pursuant to the requirements of local cooperation, a non-Federal contribution is shown amounting to 30 percent of the costs apportioned to hurricane protection (half of the estimated construction costs).

TABLE 4
COMPARISON OF ESTIMATES

Item	Project document estimate (1) (Dec. 1961)	Latest approved estimate (2) (July 1969)	Design memorandum estimate (July 1969)
	$	$	$
FEDERAL			
Locks	4,727,000	7,510,000	15,000,000
Engineering and design	265,000	900,000	1,200,000
Supervision and administration	388,000	530,000	1,200,000
Subtotal	5,380,000	8,940,000	17,400,000
Less non-Federal contribution	- 807,000	-1,341,000	-2,610,000
Net Federal cost	4,573,000	7,599,000	14,790,000
NON-FEDERAL (Cash contributions)			
For Seabrook lock hurricane protection purpose	807,000	1,341,000	2,610,000
TOTAL	5,380,000	8,940,000	17,400,000

(1) House Document No. 231, 89th Congress, 1st session.

(2) Project Cost Estimate (PB-3) dated 1 July 1969.

72. The total difference of + $3,560,000 between the project document estimate and the latest approved estimate is due primarily to price level increases, December 1961 to July 1969.

73. The total difference of + $8,460,000 between the latest approved estimate and the estimate presented in this design memorandum is due to the following:

Changes in lock foundation design: excavation of lacustrine deposits and replacement with shell fill (+ $1,184,000); and addition of bearing piles for gate bay support (+ $254,000)	+	$1,438,000
Changes in cofferdam design and other provisions for construction dewatering including those due to deficiencies of lacustrine foundation material and enlargement of area to be dewatered due to addition of outlet structure	+	3,051,000
Addition of outlet structure	+	934,000
Addition of timber pile clusters	+	36,000
Change in type of lock wall	-	19,000
Addition of land costs	+	11,000
Addition of 550 L.F. of guide wall	+	196,000
Addition of Operation and Maintenance facilities	+	146,000
Addition of sandblasting and coal tar epoxy painting steel surfaces	+	632,000
Addition of cathodic protection	+	129,000
Reanalyses of requirements for engineering and design and supervision and administration	+	970,000
Raising of lock walls, lowering of dam crest and net of other changes in plan resulting from these and foregoing modifications also more detailed planning and refinement of estimates	+	936,000

74. <u>Estimate of annual charges</u>. The estimated total investment costs and annual costs for the Seabrook Lock unit are shown below. Investment costs include interest during construction at an interest rate of 3-1/8 percent for one-half of an assumed 3-year construction period. Annual charges are based on the same interest rate and an assumed 50-year project life. The estimated annual maintenance and operations cost is based on a current analysis of require-

ments for the lock structure and appurtenant facilities proposed herein. Like estimated first costs, it reflects July 1969 price levels.

Estimated total first costs	$17,400,000
Interest during construction	816,000
Total investment costs	18,216,000
Annual costs:	
Interest	$ 569,000
Amortization	156,000
Maintenance and operations	180,000
Total annual costs	905,000

PROJECT JUSTIFICATION

75. General. Completion of the Lake Pontchartrain, Louisiana and Vicinity, project will return benefits of very considerable magnitude from reduction of hurricane-induced flood damages to existing and future developments. Estimates of these benefits easily exceed estimated project costs. The Seabrook Lock unit, as an integral feature of the Lake Pontchartrain barrier plan, is justified on the basis thereof, and also on the basis of its requirement for mitigation of adverse salt water intrusion to the lake, adverse flow conditions to riparian users, and increased current velocities in the IHNC that are detrimental to facilities therein, all attributable to the MR-GO navigation project.

76. Project formulation and evaluation. The Seabrook Complex is not a separable unit of the Lake Pontchartrain Barrier Plan, therefore, an incremental justification and individual economic analysis is not practicable.

COORDINATION WITH OTHER AGENCIES

77. General. As mentioned in paragraph 12, the State of Louisiana, Department of Public Works, was appointed project coordinator for the State by Governor McKeithen. This agency has functioned to coordinate the needs, desires, and interests of state agencies and the Corps of Engineers. The Orleans Levee District, which will provide the local cooperation for all features of the project other than those located in St. Bernard Parish, actively assisted in coordinating the project planning. The project plan presented herein is acceptable to both of the above agencies.

78. U. S. Fish and Wildlife Service. Extensive coordination with the U. S. Department of the Interior Fish and Wildlife Service was accomplished relative to the Seabrook Lock feature of the "Lake Pontchartrain, La. and Vicinity," project. By letter dated 4 November 1966, the Regional Director, U. S. Fish and Wildlife Service, Atlanta, Georgia, was informed that detailed planning for Seabrook Lock was underway, provided a tentative operating plan for the lock, and requested to furnish views and comments thereon. Subsequent to a meeting held relative to the salinity control in Lake Pontchartrain and in response to LMNED-PP letter dated 18 January 1967, the Regional Director, in a letter dated 26 April 1967, stated "...it appears at this time that operation of the lock at full discharge capacity throughout the full tidal cycle can be tolerated insofar as fish and wildlife resources are concerned." In addition, by letter dated 17 April 1967, the Regional Director was informed that preparation of a general design memorandum for the Seabrook Lock was underway, apprised of significant design modifications in the authorized lock, and requested to furnish views and comments on the modified plan. In a letter dated 7 June 1967, the Regional Director stated "...lowering the controlling elevation of the rock dike to elevation 7.2 feet will have no effect on fish and wildlife resources. On the other hand, the auxiliary water control structure should provide a more flexible system for salinity control in Lake Pontchartrain." The Regional Director also suggested that "...a salinity surveillance system be located in Lake Pontchartrain after the Seabrook structure is in place." Relative to the Regional Director's suggestion, upon completion of the lock, an adequate evaluation of the effects of lock operation on the salinity regimen and a determination as to the extent that the lock operation is producing the salinity regimen indicated by model data will be provided. Copies of the above correspondence are contained in Appendix B.

79. Federal Water Pollution Control Administration. By letter dated 19 April 1967, the Regional Director, Federal Water Pollution Control Administration of the U. S. Department of the Interior, was informed that preparation of a general design memorandum for the Seabrook Lock was underway, apprised of the departures from the project document plan, and requested to furnish views and comments on the modified plan. The Regional Director requested, in his letter of response dated 23 June 1967, that consideration be given to the following:

a. Minimizing water quality degradation during construction.

b. Constructing and operating the control structures so as to insure that ecological conditions remain unchanged.

c. Precluding mosquito breeding problems caused by increasing the Lake Pontchartrain water level, as a result of the hurricane protection project, thus flooding the lowlands bordering the lake.

d. Minimizing the accidental spillage of petroleum products or other harmful materials and maintenance of sanitary facilities to adequately treat domestic wastes.

Provisions relative to water quality degradation during construction, control of accidental spillages, and maintenance of adequate sanitary facilities by construction contractors will be incorporated into the construction plans and specifications. The Seabrook Lock will be operated to provide a desirable salinity regimen in Lake Pontchartrain to the end that deleterious alterations in lake ecology will be avoided. The Regional Director has been advised of the action to be taken in connection with his comments. Copies of correspondence with the Regional Director are included in Appendix B. With respect to the concern relative to mosquito breeding problems, in the event that the average level of Lake Pontchartrain is raised, it is noted that the Lake Pontchartrain Barrier Plan will not result in any material increase in the average lake level, but will serve only to prevent uncontrolled increases in lake levels during hurricanes.

80. Louisiana Wild Life and Fisheries Commission. By letter dated 17 April 1967, the Director, Louisiana Wild Life and Fisheries Commission, was informed that preparation of a general design memorandum for the Seabrook Lock was underway, apprised of significant design modifications in the authorized lock, and requested to furnish views and comments on the modified plan. In a letter dated 2 May 1967, the Director recommended that provisions be installed in the Seabrook Lock to allow salinities in Lake Pontchartrain to be adjusted as may be necessary for the maintenance of fish and wildlife resources. Relative to the above recommendation, upon completion of the lock, an adequate evaluation of the effects of lock operation on the salinity regimen and a determination as to the extent that the lock operation is producing the salinity regimen indicated by the model data will be provided. Copies of the above correspondence are contained in Appendix B.

81. New Orleans Public Service Inc. Subsequent to project authorization, extensive coordination covering operation of the Seabrook Lock for riparian needs was accomplished. NOPSI (New Orleans Public Service, Inc.), the electric utility for the city of New Orleans, operates steam electric generating stations on the IHNC and the MR-GO and is the controlling riparian use; therefore, its satisfaction will insure that all riparian needs are adequately provided

for. Based on careful engineering studies relative to adverse effects of the Seabrook Lock on NOPSI cooling water requirements, Mr. L. J. Cucullu, Vice-President and Chief Engineer of New Orleans Public Service, Inc., in a letter dated 10 February 1967 stated that his agency is in agreement with the proposal presented in LMNED-PP letter dated 1 February 1967 to operate Seabrook Lock at full discharge capacity on a continuous basis. Copies of the above correspondence are contained in Appendix B.

SCHEDULE FOR DESIGN AND CONSTRUCTION

82. It is planned that the entire Seabrook Lock complex will be constructed under a single contract. The proposed schedule for completion of design and construction is as follows:

Submission of Design Memorandum No. 2, Detailed Design	October 1970
Submission of plans and specifications	February 1972
Advertise for bids	July 1972
Award contract	September 1972
Start construction	October 1972
Complete construction	May 1975

83. The portion of the above schedule concerning construction is based on the following assumed sequence of contract operations:

Phase I. October 1972 - December 1973. Predredge lacustrine deposits and place shell fill in entire lock area. Construct lakeward cofferdam, unwater, and construct lakeward gate bay and sector gate.

Phase II. September 1973 - December 1974. Divert navigation to the west of lock site and start landward cofferdam. Remove lakeward cofferdam and reuse the piling and shell fill to complete landward cofferdam. Construct landward gate bay, outlet structure, retaining walls and install sector and lift gates. Construct lakeward guide wall, guard wall and dolphins and place stone protection around lakeward gate bay.

Phase III. July 1974 - January 1975. Start construction of lock chamber walls. Predredge lacustrine deposits in area to be occupied by dam. Remove landward cofferdam, using the cell fill in dam core. Place stone protection around lock chamber walls, on floor of lock and around landward gate bay and outlet structure.

Construct section of dam east of outlet structure and complete southeast guide wall. Complete the mechanical and electrical installation and testing.

Phase IV. December 1974 - May 1975. Divert navigation through lock chamber. Construct remainder of rock dam and southwest landward guide wall.

84. To maintain the design and construction schedule, the following allocation of funds, by fiscal year, will be required:

Thru FY 1969	$ 537,000
FY 1970	340,000
FY 1971	340,000
FY 1972	149,000
FY 1973	5,200,000
FY 1974	7,900,000
FY 1975	2,934,000

OPERATION AND MAINTENANCE

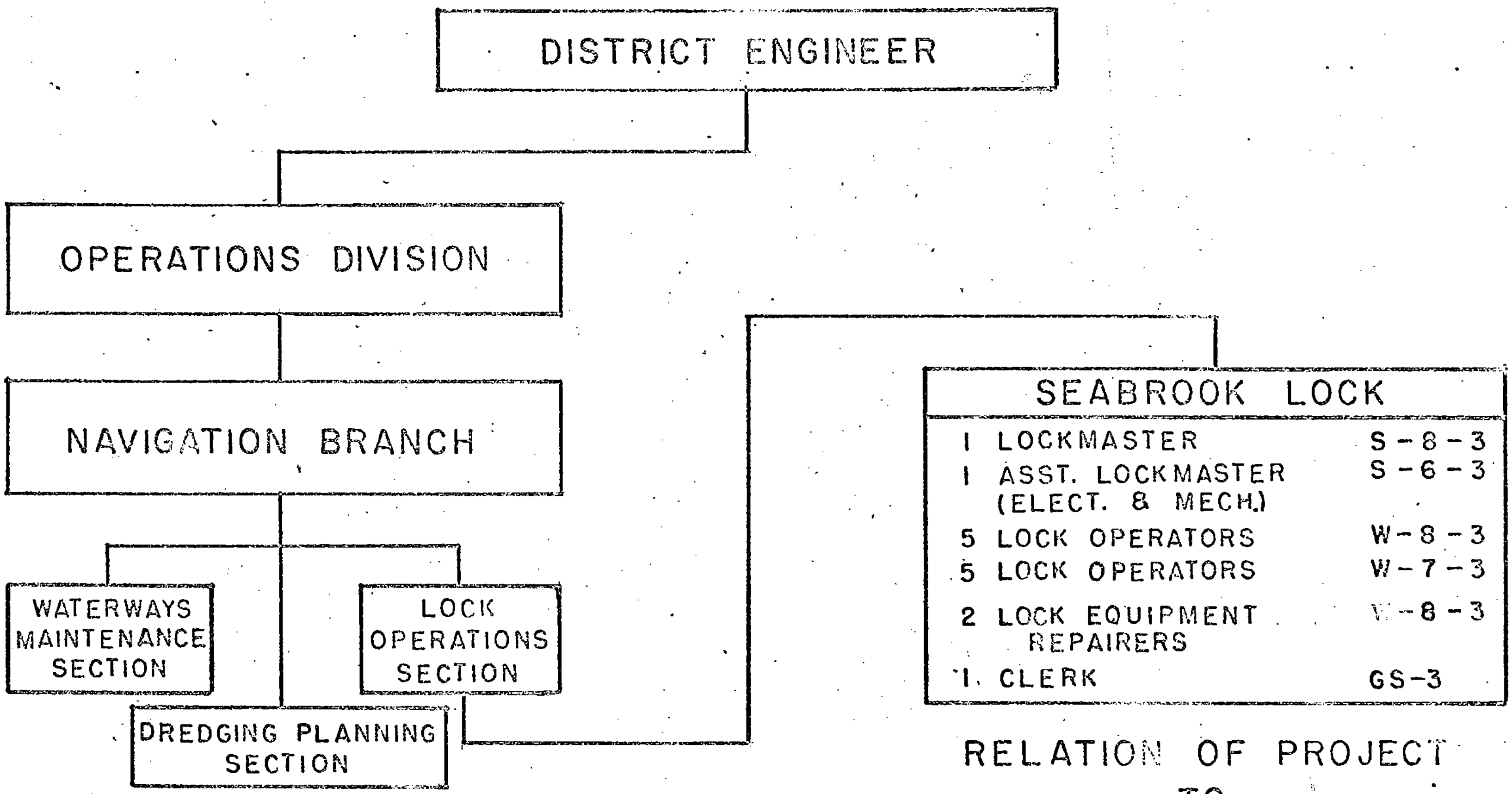
85. Maintenance will consist generally of repairs to lock components and maintenance of cover stone layers on the dam. Operation of the lock will be in accordance with standard operating procedures modified as described in Appendix A because of the dual operational function peculiar to this lock. Operation and maintenance will be under the supervision of the Operations Division, New Orleans District, Corps of Engineers, as shown on figure 1, following. The force required to operate and maintain the lock is estimated as follows:

No.	Position	Grade
1	Lockmaster	S-8
1	Asst. Lockmaster	S-6
5	Lock operators	W-8
5	Lock operators	W-7
2	Lock equipment repairers	W-8
1	Clerk	GS-3

RECOMMENDATION

86. It is recommended that the project plan for Seabrook Lock presented in this design memorandum be approved, and that further detailed design related to this unit of the Lake Pontchartrain, Louisiana and Vicinity hurricane protection project proceed on the basis thereof.

CORPS OF ENGINEERS U.S. ARMY

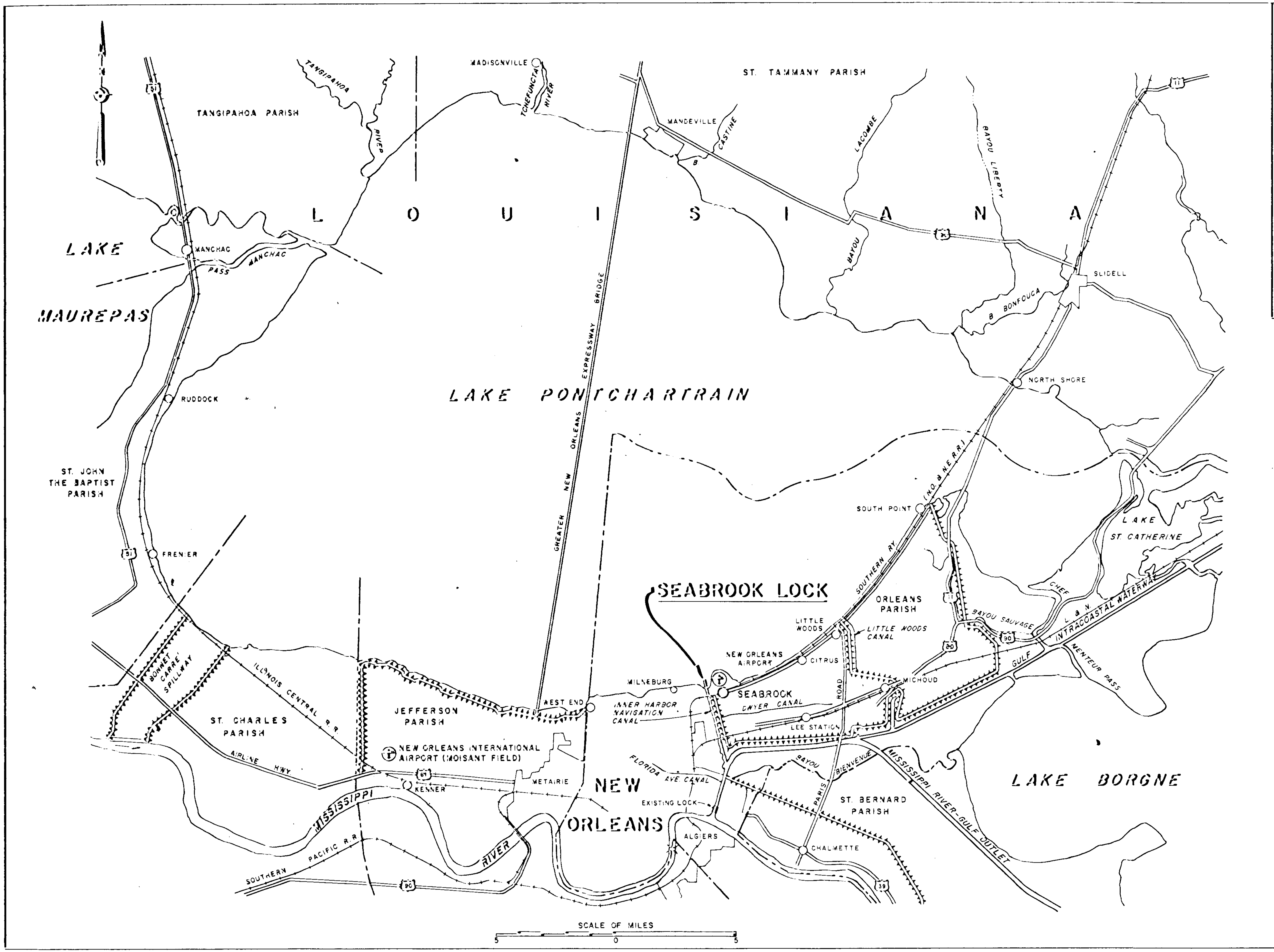


1	LOCKMASTER	S-8-3
1	ASST. LOCKMASTER (ELECT. & MECH.)	S-6-3
5	LOCK OPERATORS	W-8-3
5	LOCK OPERATORS	W-7-3
2	LOCK EQUIPMENT REPAIRERS	W-8-3
1.	CLERK	GS-3

RELATION OF PROJECT
TO
NEW ORLEANS DISTRICT OFFICE
FOR
OPERATION AND MAINTENANCE

FIG. I

FIG. I



LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

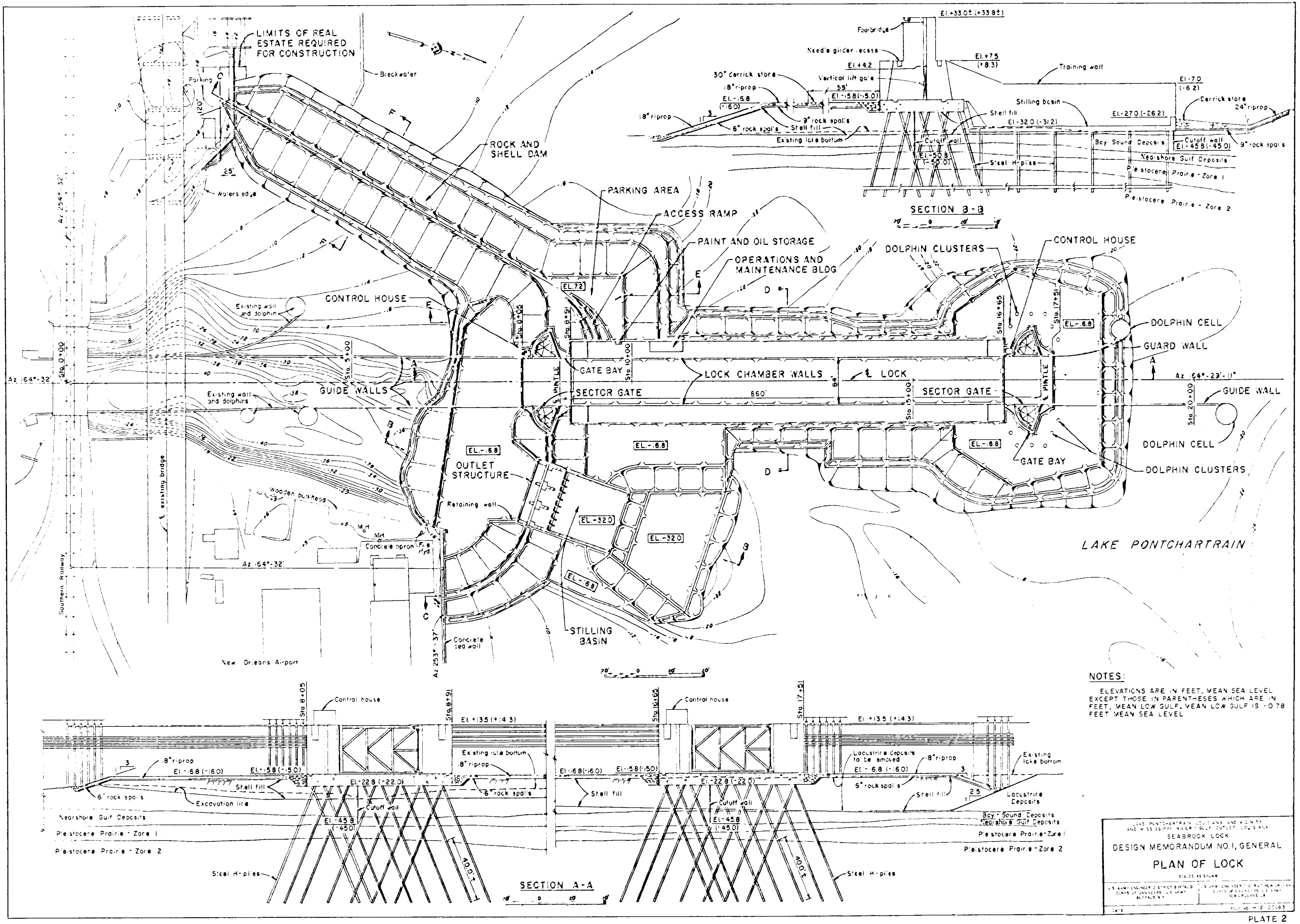
SEABROOK LOCK

DESIGN MEMORANDUM NO. I, GENERAL
LOCATION PLAN
AND VICINITY MAP

SCALES AS SHOWN

U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N Y	U S ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO H-2-23683

PLATE I



NOTES:

ELEVATIONS ARE IN FEET, MEAN SEA LEVEL EXCEPT THOSE IN PARENTHESES WHICH ARE IN FEET, MEAN LOW GULF. MEAN LOW GULF IS -0.78 FEET MEAN SEA LEVEL

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

PLAN OF LOCK

SCALES AS SHOWN

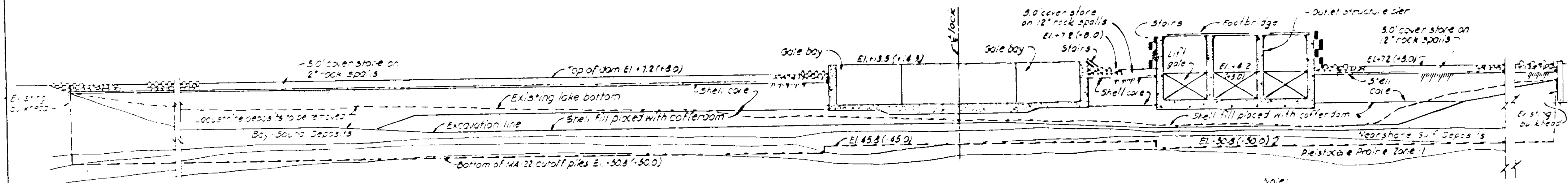
U.S. ARMY ENGINEER DISTRICT, BUFFALO
CORPS OF ENGINEERS, U.S. ARMY
BUFFALO, N.Y.

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

DATE

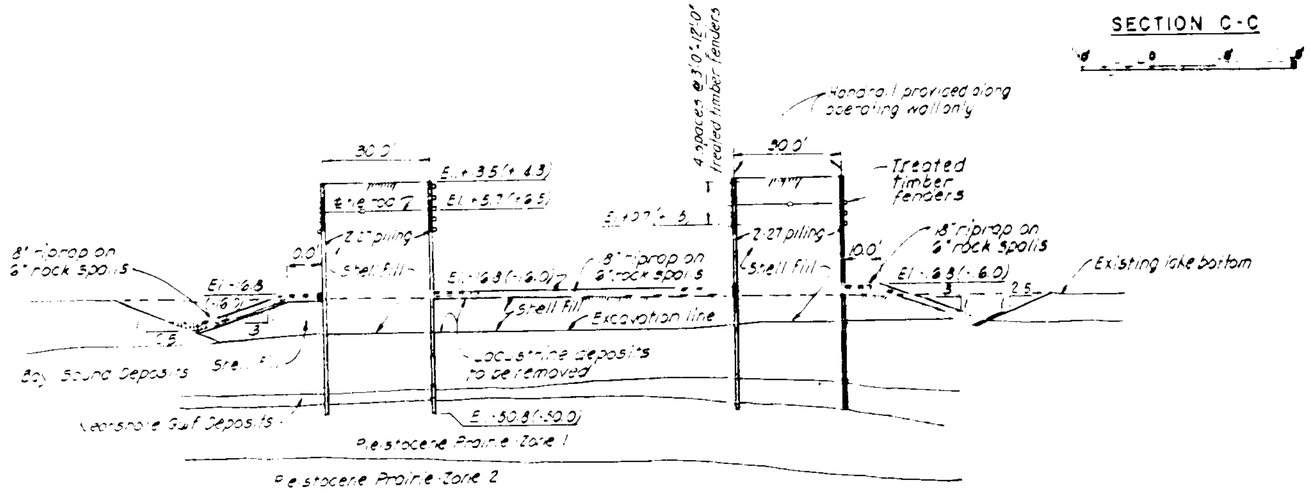
FILE NO. H-2-23083

PLATE 2

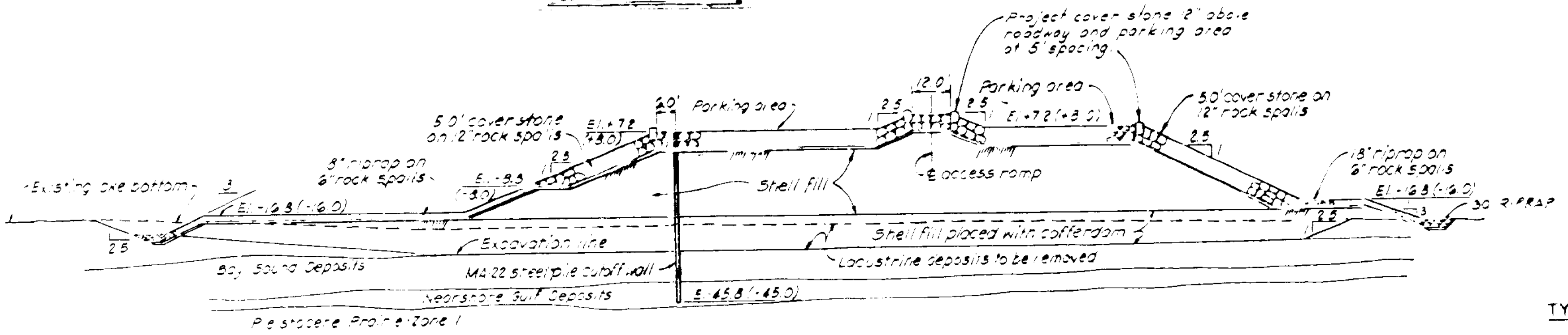


Note:
Foundation piles not shown

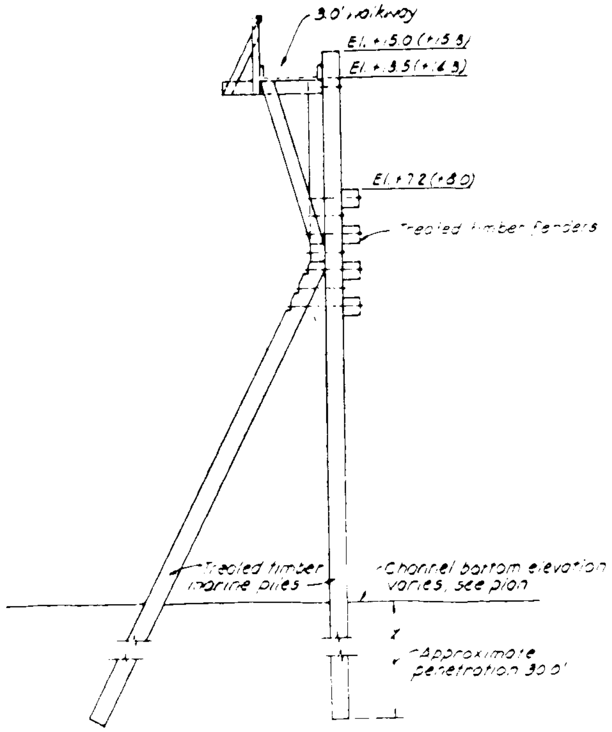
SECTION C-C



SECTION D-D



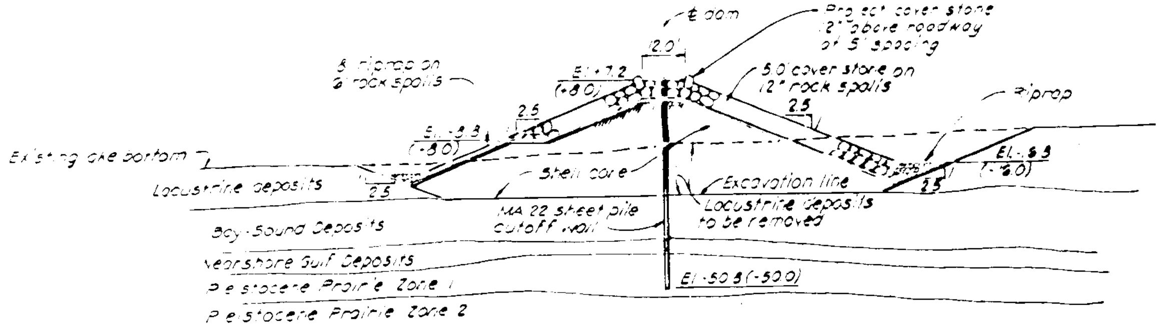
SECTION E-E



TYPICAL SECTION THRU GUIDE AND GUARD WALLS

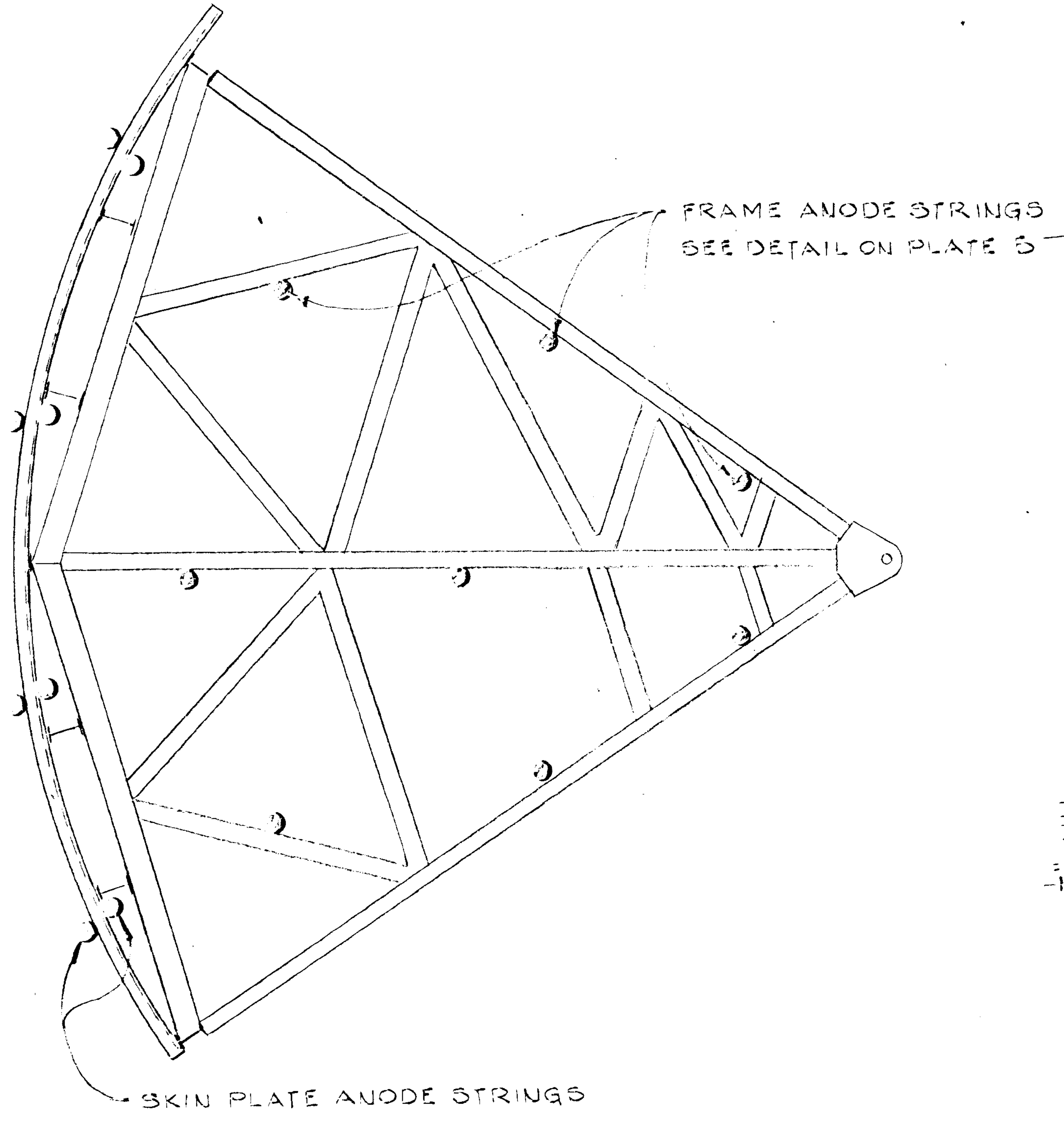
NOTES:

For location of sections C-C, D-D, E-E and F-F see Plate 2
Elevations are in feet Mean Sea Level except those shown in parentheses which are in feet Mean Low Gulf

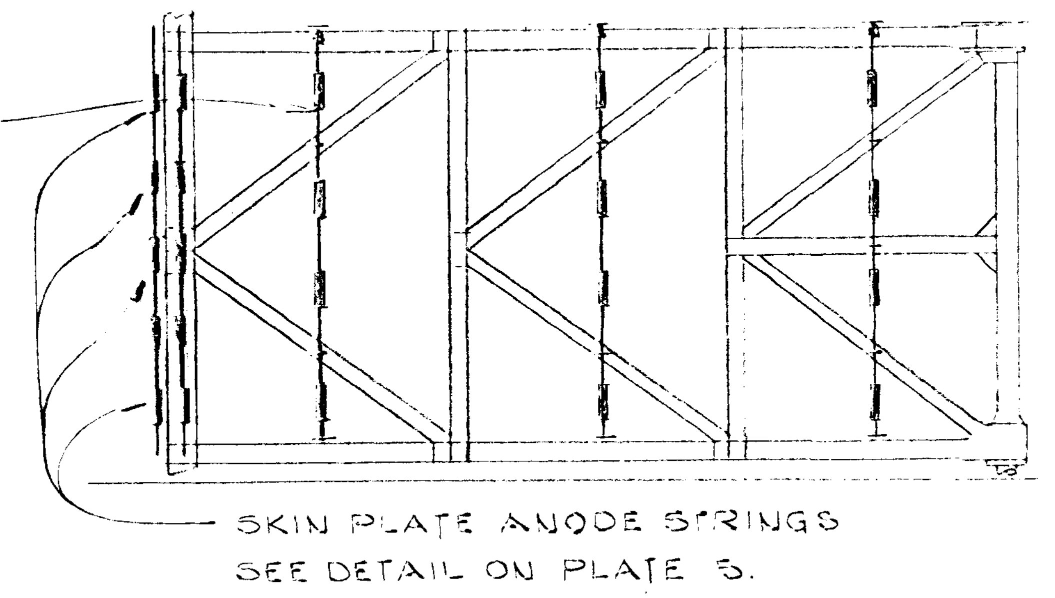


SECTION F-F

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO I, GENERAL
SECTIONS
SCALES AS SHOWN
U.S. ARMY ENGINEER DISTRICT, BUFFALO / CORPS OF ENGINEERS, U.S. ARMY / BUFFALO, N.Y.
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS / CORPS OF ENGINEERS, U.S. ARMY / NEW ORLEANS, LA



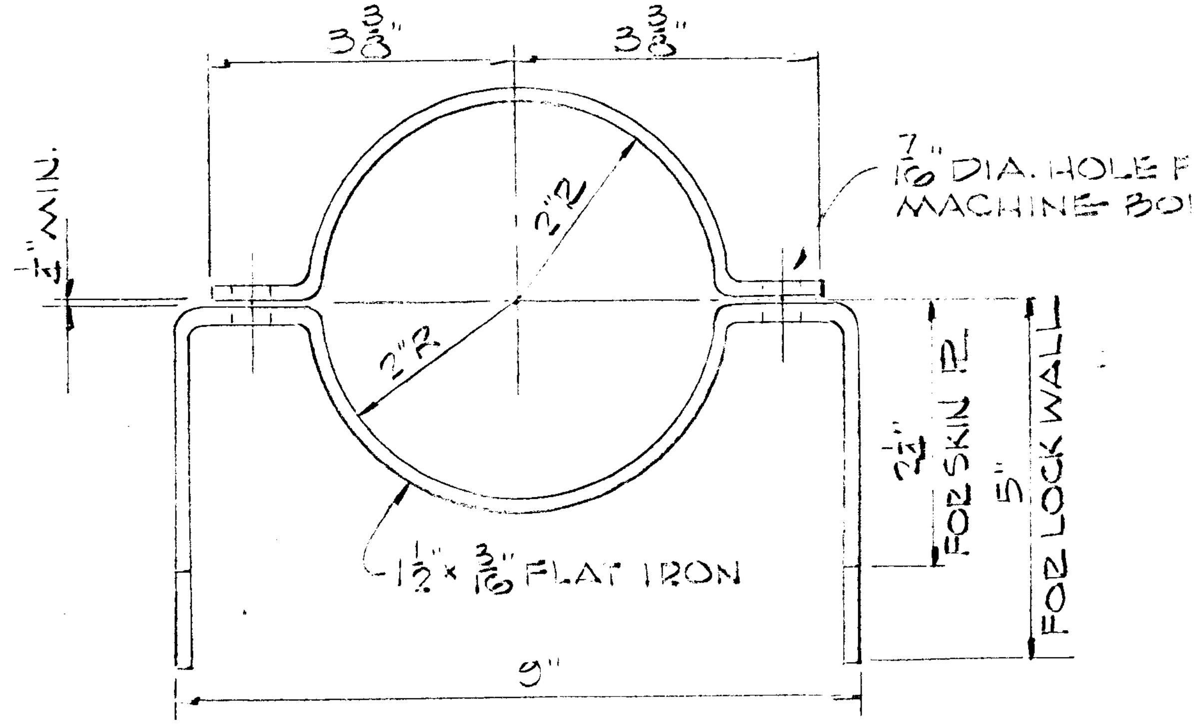
PLAN



SECTIONAL ELEVATION

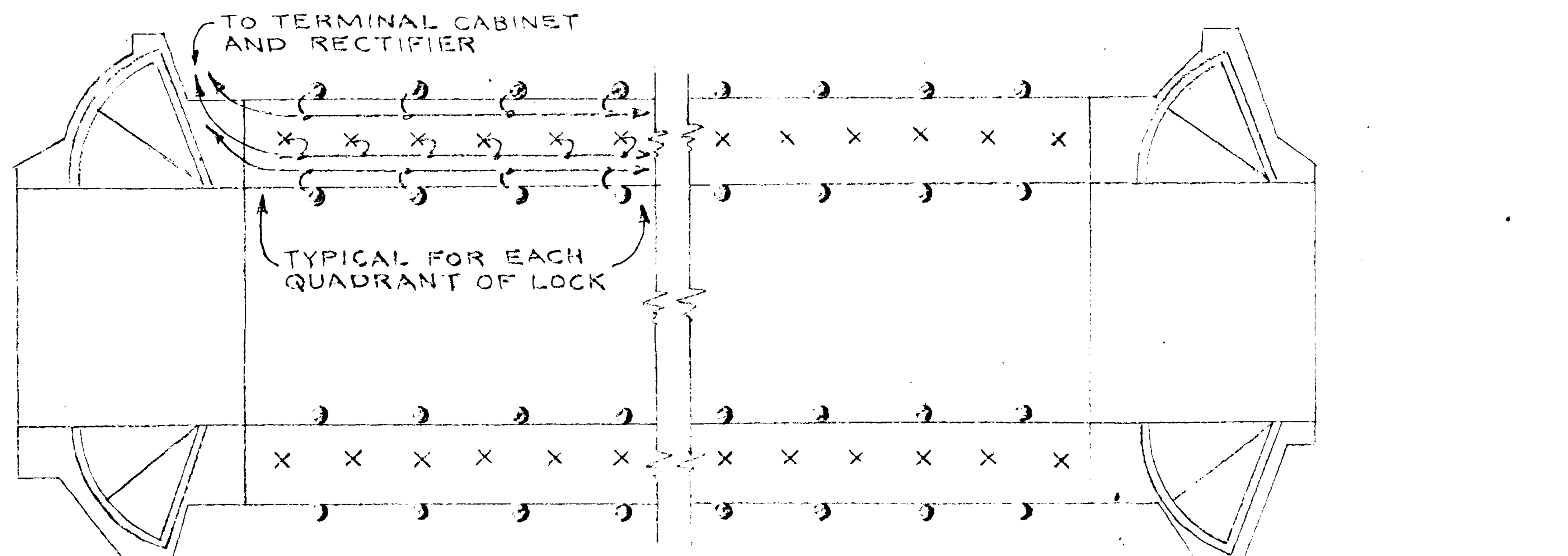
TYPICAL SECTOR GATE

NO SCALE



ANODE SUSPENSION BRACKET

NO SCALE

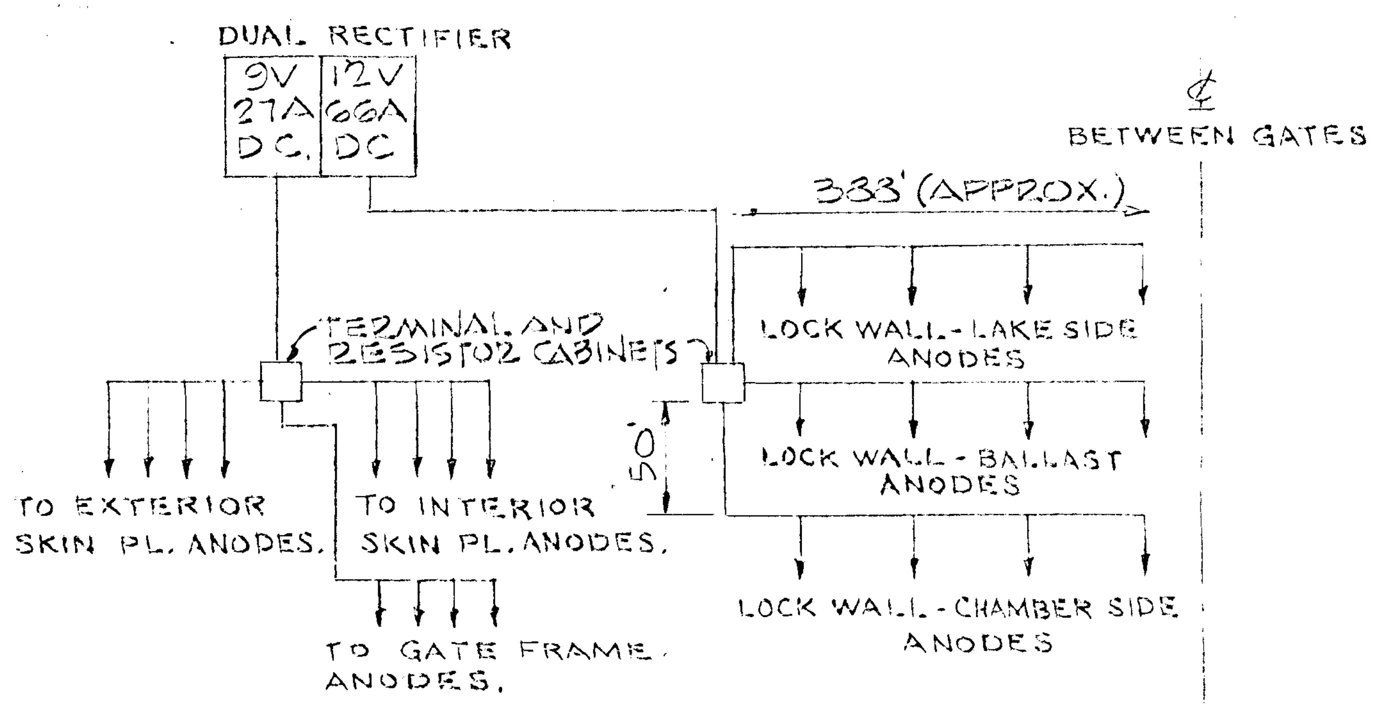


X ANODES INSTALLED IN UNDISTURBED SOIL BETWEEN PILE WALLS.

) ANODES IN SCHEDULE 80 PLASTIC PIPE GUIDES ON PILE WALLS.

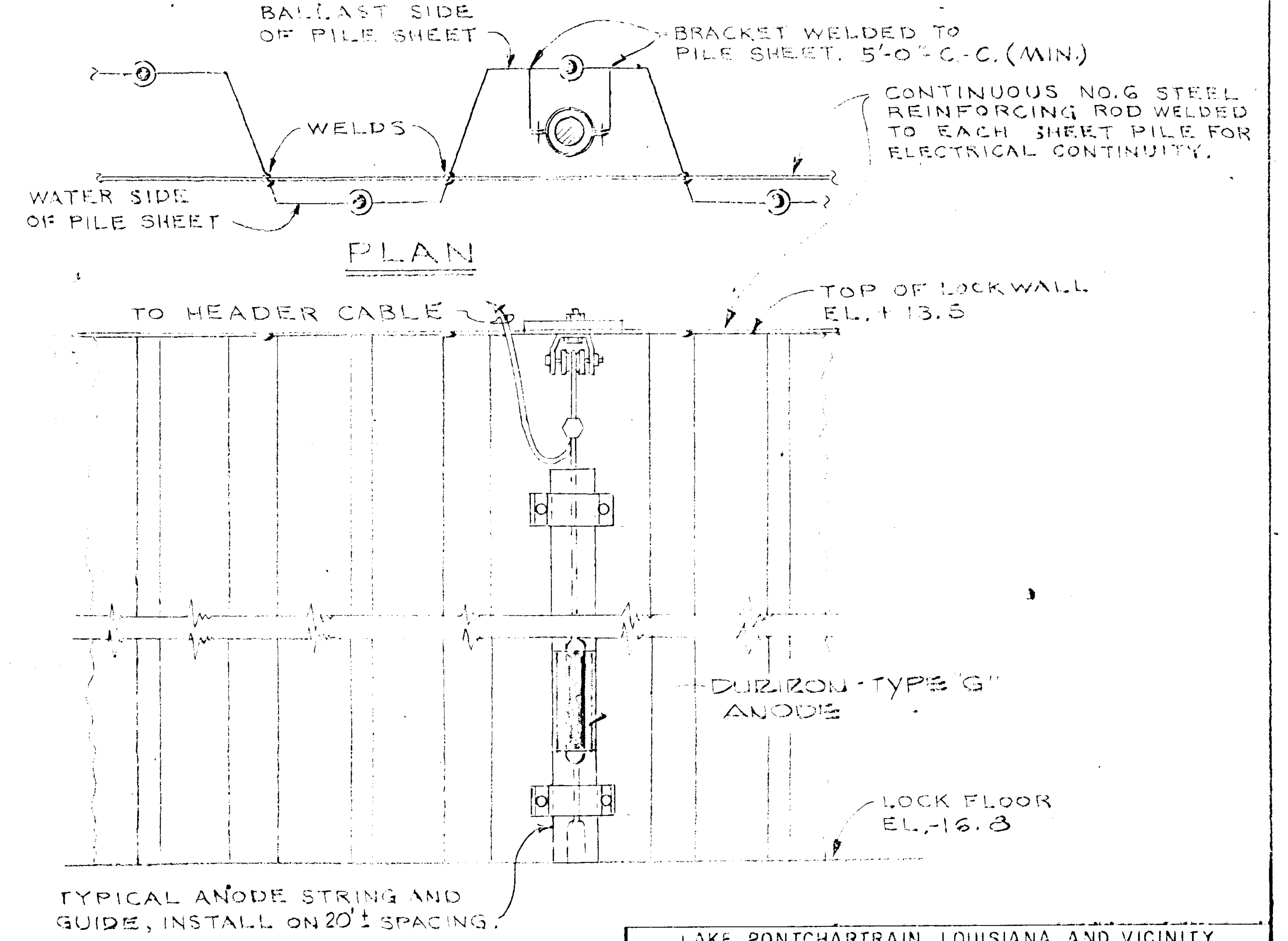
LOCK PLAN

NO SCALE



WIRING DIAGRAM

NO SCALE



ELEVATION

TYPICAL ANODE INSTALLATION ON LOCK WALL

NO SCALE

NOTE:
Elevations are in feet Mean Sea Level.

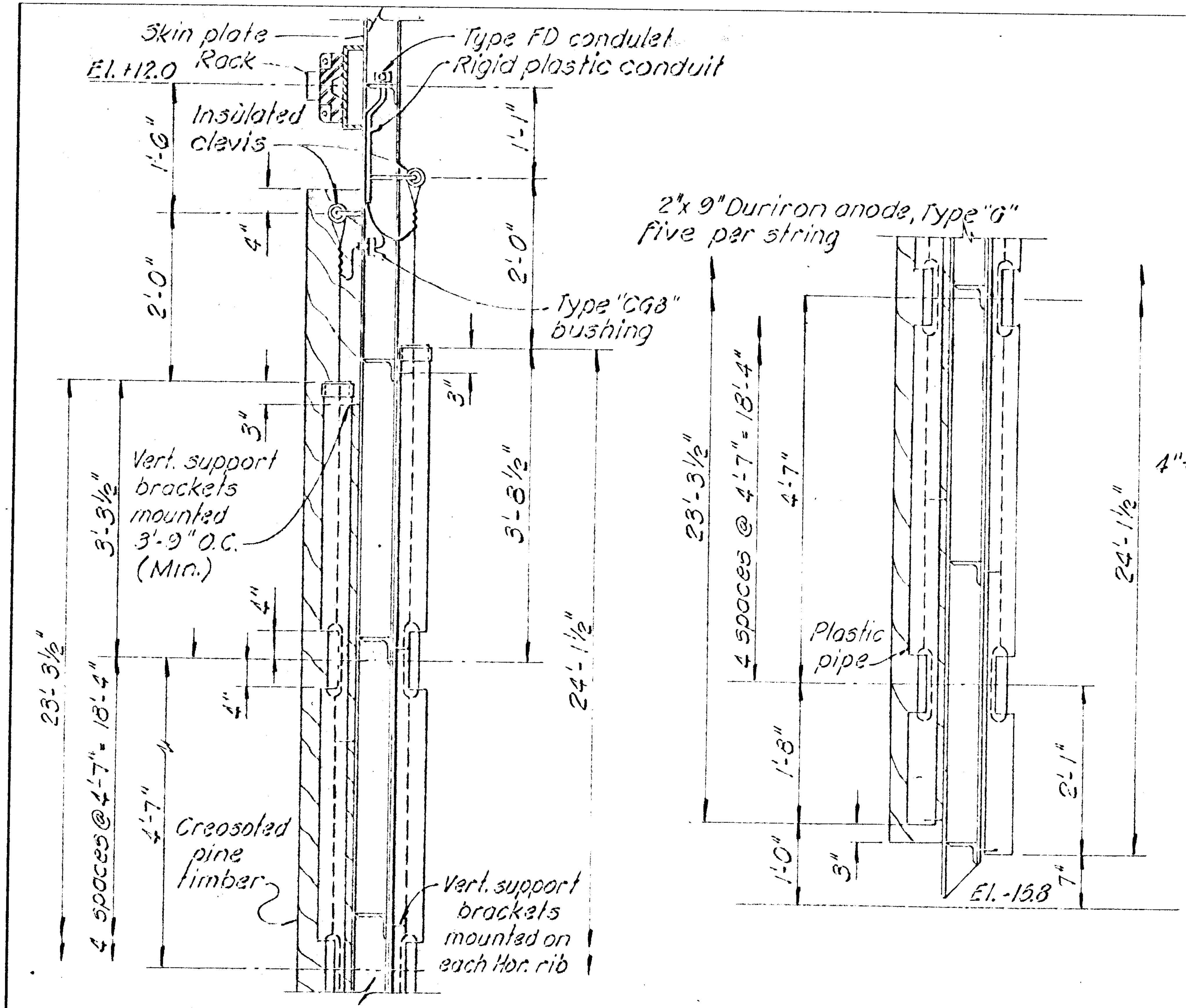
LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

CATHODIC PROTECTION

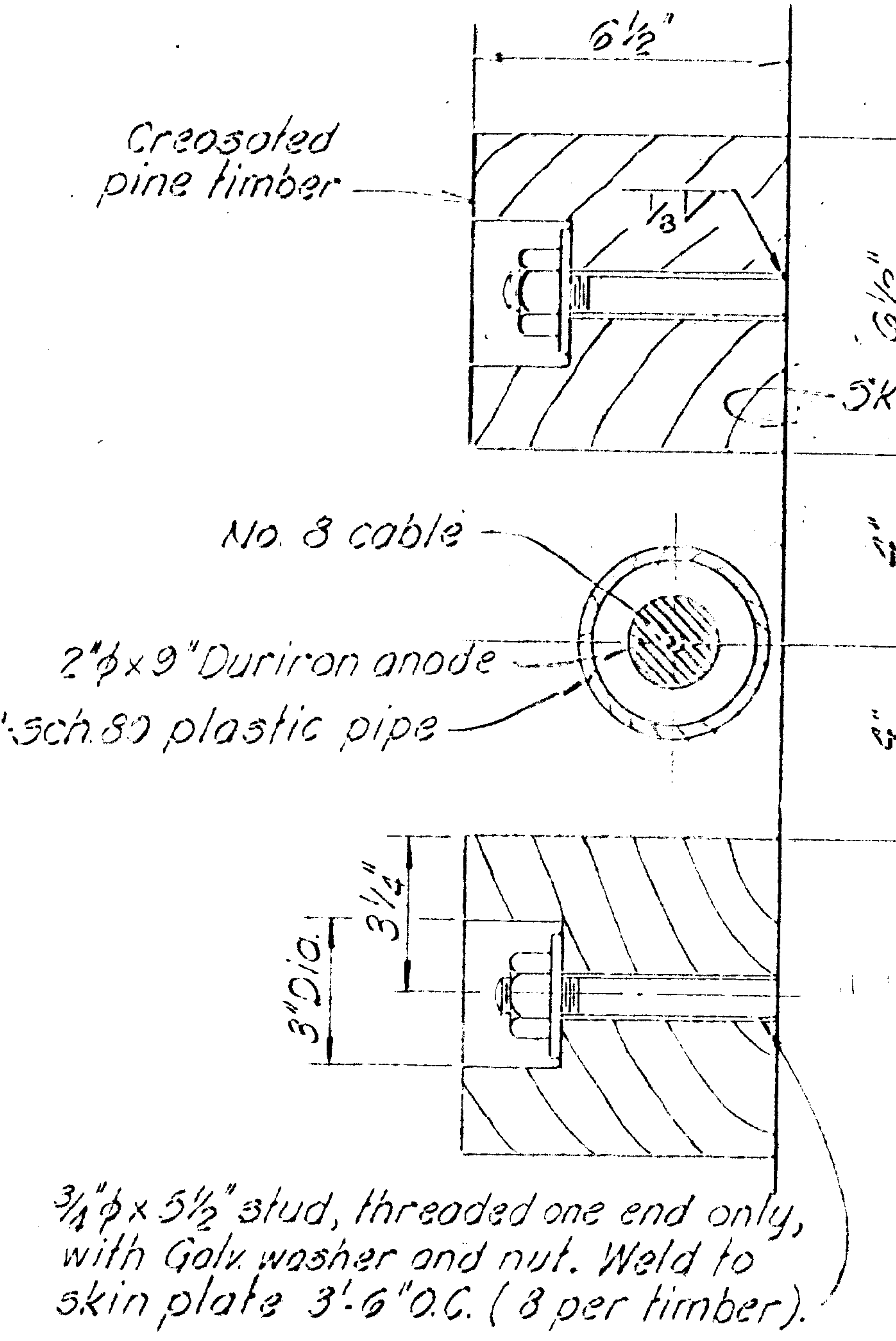
U.S. ARMY ENG. DIST. MOBILE, ALA.	U.S. ARMY ENG. DIST. NEW ORLEANS, LA.
DATE:	FILE NO. H-2-23883

PLATE 4



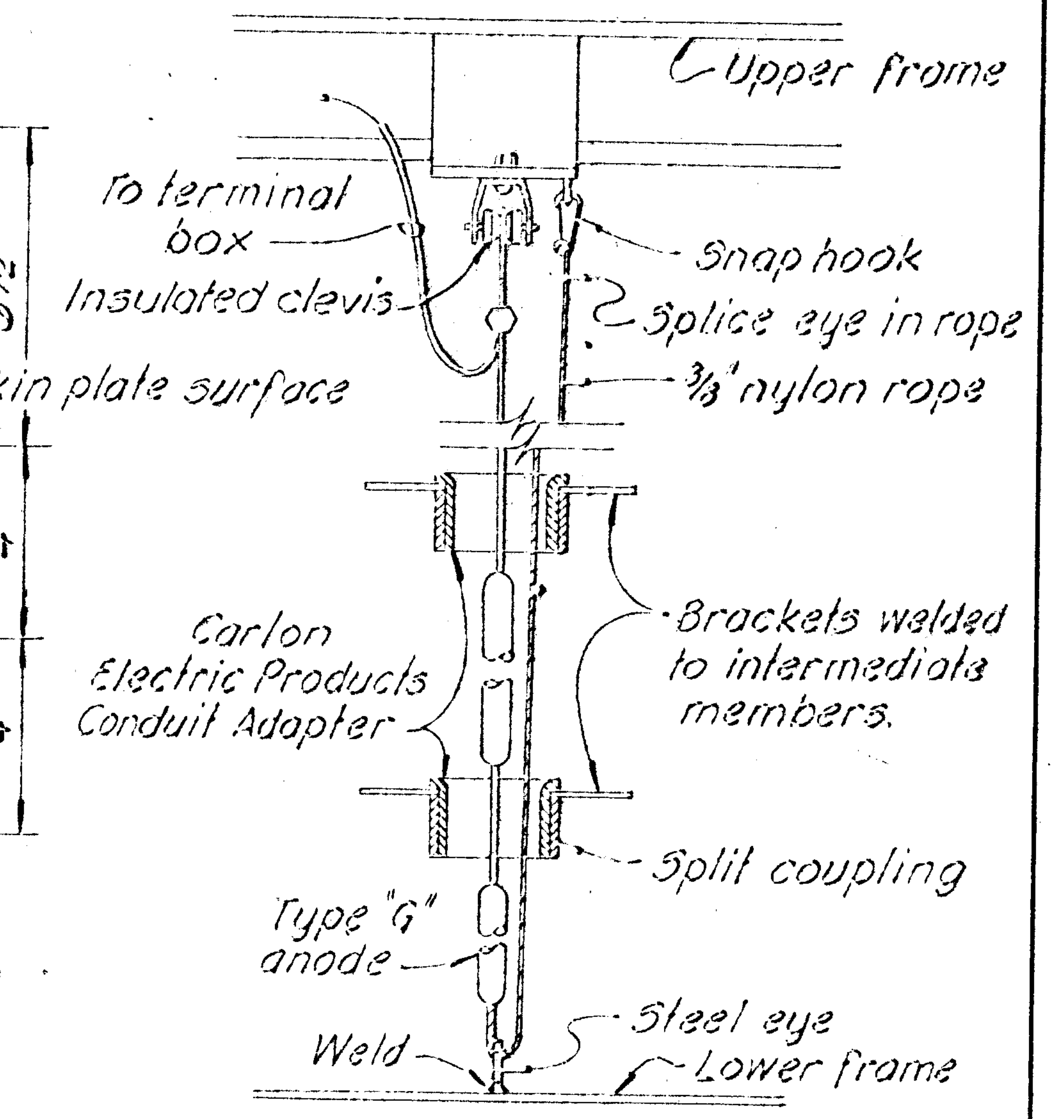
MOUNTING FOR SKIN PLATE ANODE STRINGS

SCALE $\frac{3}{4}$" = 1'-0"



SKIN PLATE MOUNTING DETAIL

SCALE 3" = 1'-0"



CABLE SUSPENSION AND STAY LINE ANCHORAGES FOR FRAME ANODE STRINGS

NOT TO SCALE

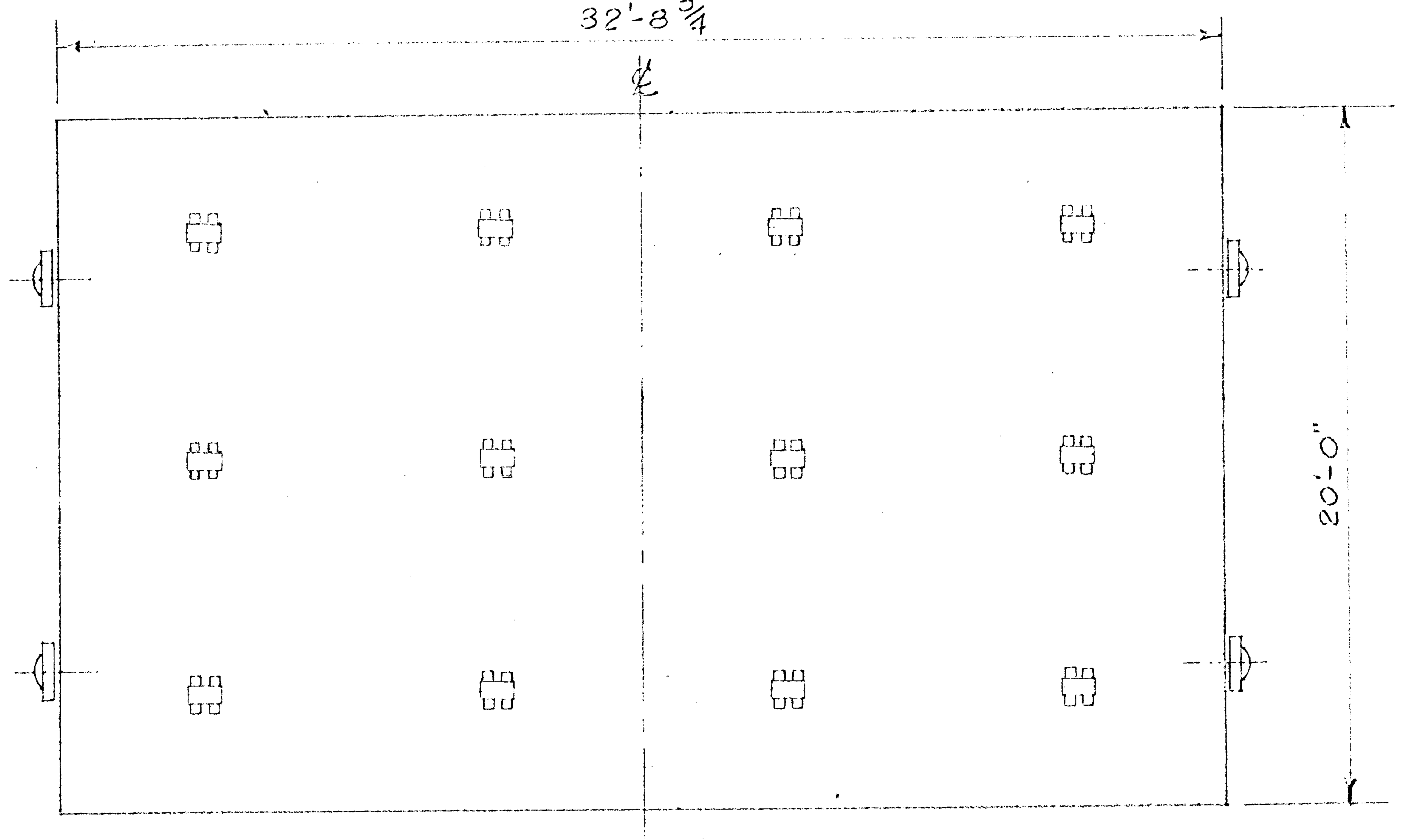
Note: Elevations are in feet Mean Sea Level.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

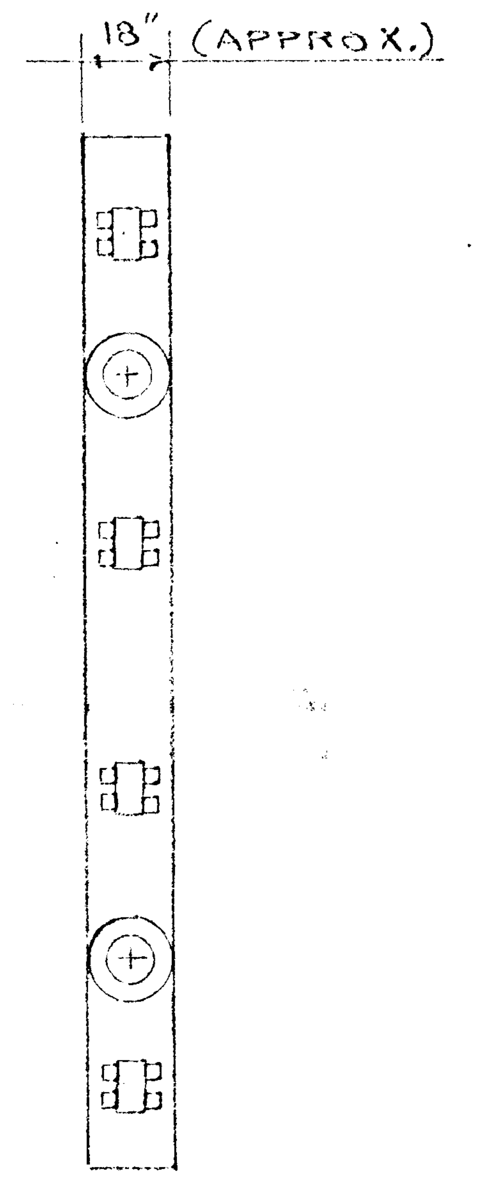
CATHODIC PROTECTION DETAILS

U.S. ARMY ENG. DISTRICT MOBILE, ALA.	U.S. ARMY ENG. DISTRICT NEW ORLEANS, LA.
DATE:	FILE NO. H-2-23683



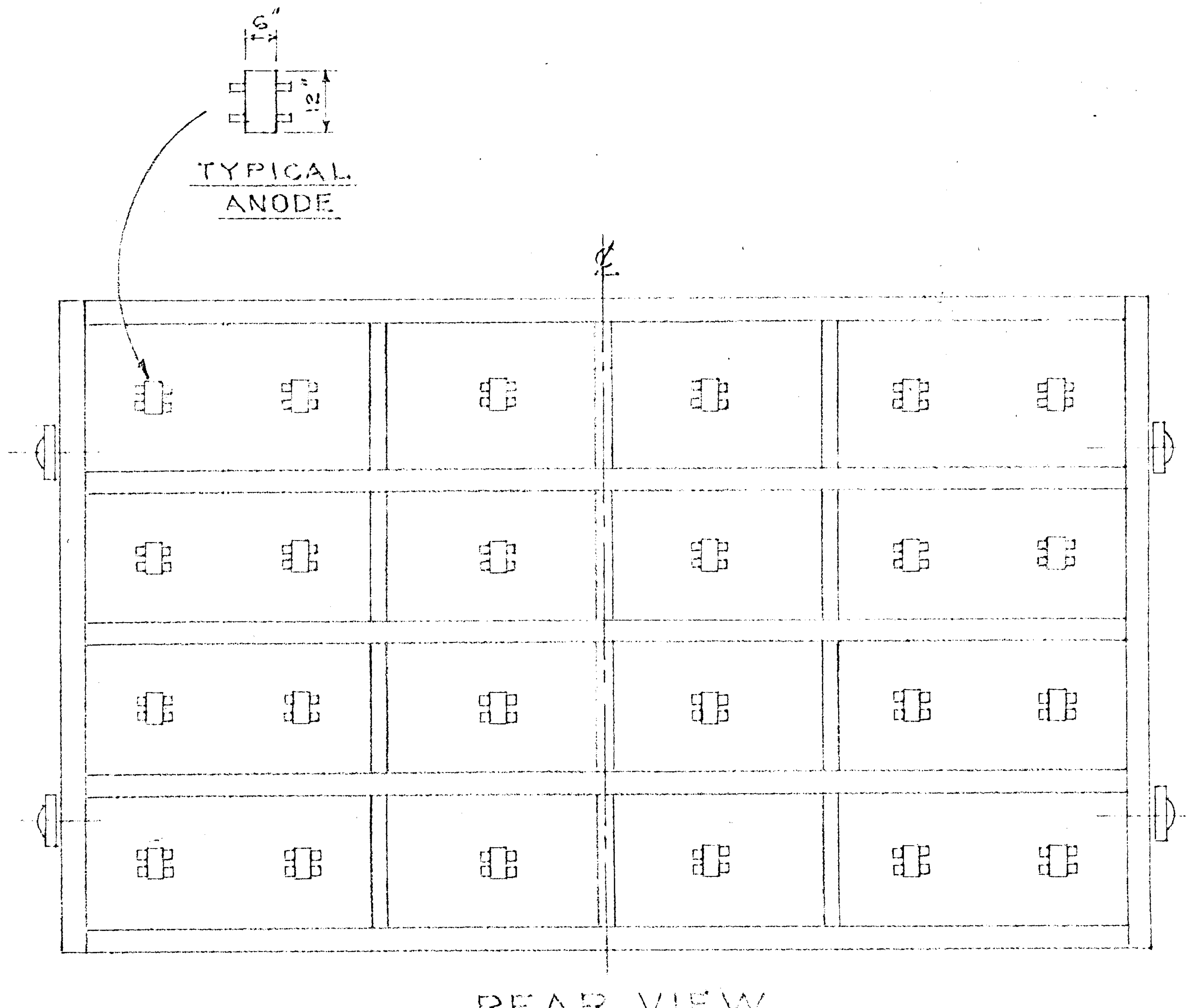
FRONT VIEW

NO SCALE



SIDE VIEW

NO SCALE



REAR VIEW

NO SCALE

NOTES:

1. ALL ANODES ARE TO BE AZ.CO TYPE 8-24 ZINC HULL ANODES,
2. ANODE STRAPS WILL BE - WELDED TO GATE STEEL AT ENDS, AFTER GATES ARE PAINTED.
3. WELD SPATTER SHALL BE CLEANED, AND WELDS TOUCHED UP WITH SAME FINISH AS GATE.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

CATHODIC PROTECTION
(VERTICAL LIFT GATES)

U.S. ARMY ENG. DIST. MOBILE, ALA.	U.S. ARMY ENG. DIST. NEW ORLEANS, LA.
DATE:	FILE NO. H-2-23658

PLATE 6

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

APPENDIX A
HYDRAULICS

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont.)

TABLES

PLATES

APPENDIX A

SECTION I
FLOOD RELIEF MITIGATION

1. General.

a. The "Lake Pontchartrain, La. and Vicinity" project, authorized 27 October 1965, consists of two independent features: the Lake Pontchartrain Barrier Plan and the Chalmette Area Plan. Only the former is pertinent to this report and, in particular, the Seabrook Complex feature of the Barrier Plan. The rationale of the Barrier Plan is to limit the uncontrolled entry of hurricane tides into Lake Pontchartrain, thereby controlling the average lake stage while, at the same time, preserving navigational access to the lake. To prevent any interchange of flow between Lake Pontchartrain and the IHNC (Inner Harbor Navigation Canal) during a hurricane, a controlling elevation of 13.2[1] was established for the Seabrook Complex.

b. The term "controlling elevation" as used herein refers to the elevation at which freeflow over the Seabrook Complex will commence. With respect to hurricane flood control objectives, the crest elevation of the rock dike is considered to be the controlling elevation of the Complex, inasmuch as the required elevations of the chamber walls, gates, and gate bays must be based on considerations relating to the safe and efficient operation of the lock in compliance with navigational needs. Elevation 13.2 is required to prevent freeflow over the rock dike and lock during passage of the SPH (Standard Project Hurricane) critical to the IHNC; i.e., a hypothetical hurricane on track F. Refer to plate A-1. The SPH on track F produces the highest stages in the IHNC. Stages resulting from the SPH on tracks critical to the south shore of Lake Pontchartrain are not high enough to overtop the dike from the lakeward direction.

2. Measures to afford stage relief in the IHNC.

a. All development located immediately outside of the authorized plan of protection is subject to the hazards of flooding from hurricane-generated surges. The present industrial development located along the IHNC is particularly subject to severe flooding hazards since this industry is located at approximate elevation 5.0 on the floodside of existing and authorized protective works. Consequently, the Seabrook Complex feature of the authorized plan has inspired much inquiry by the IHNC industries as to whether or not the Seabrook Complex would increase the susceptibility of the industries to flooding in the event of a hurricane.

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[1] All elevations used herein are in feet and refer to mean sea level datum unless otherwise noted.

b. As originally authorized, the Seabrook Complex would be closed throughout the duration of a hurricane, thus limiting inflow from the MR-GO (Mississippi River-Gulf Outlet) and IHNC to Lake Pontchartrain. Careful review of this plan revealed that retarding the hurricane surge in the IHNC and permitting a stage of 13.0 in the canal would greatly increase flood hazards to development along the canal. Consequently, studies were made to determine an optimum operational scheme for the Seabrook Complex; that is, a method which would not appreciably alter the existing flood relief conditions.

3. Lowering the controlling elevation.

a. The passage of Hurricane Betsy in September 1965 demonstrated that, under certain conditions, permitting flow to enter Lake Pontchartrain from the IHNC is advantageous, in that it affords stage relief to development along the IHNC. Studies were made to determine whether or not the controlling elevation of Seabrook Complex could be lowered thus allowing lakeward overtopping in the event of a hurricane and providing better relief to the industries on the canal.

b. A detailed report on the controlling elevation is presented in LMNED-PP letter dated 19 October 1966 subject, "Lake Pontchartrain, La. and Vicinity - Report on Controlling Elevation of Seabrook Lock." This report, approved by LMVD on 12 January 1967, established elevation 7.2 as the optimum grade for the rock dike. This elevation would allow overtopping from the canal to the lake, thus affording better stage relief than the authorized plan, and would not significantly increase the level of Lake Pontchartrain.

c. The project document plan contemplated that the top of the lock walls lakeward of the barrier be at elevation 7.2. As described in the letter report cited in paragraph 3b above, a reevaluation of factors pertinent to safe and efficient operation of the lock revealed that the top of the lock walls should provide at least 10 feet of freeboard above normal high tides. Based on records from 1922 to 1965 at nearby locations, the mean elevation of annual high tides at Seabrook, excluding hurricane tides, is 3.5. Irrespective of hurricane conditions and based on the foregoing, the height of the lock walls was established at elevation 13.5.

4. Outlet structure.

a. In an attempt to provide additional flood relief, studies were made to determine whether or not stages in the canal could be further lowered by passing flows through the Complex during a hurricane. The most feasible means was to construct an outlet structure adjacent to the lock and allow it to discharge flows into the lake for a relatively short period of time provided that the discharge would not significantly increase the average lake stage and alter grade requirements for lakefront levees. Computations disclosed that passing flows through an outlet structure for the condition which produced the maximum average lake stage, i.e., the SPH on track A (see plate A-1) the average lake level would be increased by only 0.06 feet, consequently, not affecting lakefront levee grades. For this same condition it was revealed that this modified plan, in conjunction with other features of the Barrier Plan, would serve to decrease stages in the IHNC by some 2 to 3 feet below existing conditions. For hurricanes approaching on track F (see plate A-1), it was determined that this modified plan would produce stages essentially the same as those which prevail under existing conditions. This modified plan for the Seabrook Complex, and the remaining features of the Barrier Plan, would not increase the susceptibility of the industries to flooding.

b. A gated outlet structure as described in the text of this design memorandum is proposed as the most feasible measure for discharging hurricane flows in the IHNC. Other purposes regarding the necessity for an outlet structure are presented in paragraph 12 of this report. The proposed structure will be comprised of three gate bays, each bay 32 feet wide with a sill elevation of -15.8. The gates will be of the vertical lift type so that they may be regulated to provide any flow as may be required.

5. Design operational plan.

a. Based on the studies described above, a plan for operating the gated outlet structure to reduce flood stages in the IHNC during hurricanes has been formulated. The operating procedure for stage relief in the IHNC provides for the gates of the outlet structure to be fully opened when a stage of 3.5 occurs on the IHNC side of the lock and to remain open for the duration of the hurricane. Flows from the IHNC will pass into the lake continually throughout the hurricane regardless of the track. Lockages will continue for some period of time after the outlet structure gates have been fully opened and cease when a stage of 5.0 is obtained on the IHNC side of the lock. At this time, the lock personnel will fully close both sets of lock gates and leave the lock site. The Complex will remain unmanned throughout the duration of the hurricane.

b. To satisfy riparian flow requirements discussed in Section II of this appendix, the outlet structure will normally be operated to provide an opening of approximately 260 square feet. One gate bay will be partially opened on a continual basis to provide this flow area. After completion of the Seabrook Complex, a data collection system will be established, and it will serve to determine any future modifications to the operating procedure described herein as may be necessary. The Complex will otherwise be operated as described above.

6. Conclusions.

a. By lowering the controlling elevation of Seabrook Complex and by modifying the Seabrook Complex to include the gated outlet structure, hurricane flooding along the IHNC will be about the same or lower (depending on the hurricane track) than under existing conditions. Plate A-2 shows stage-hydrographs for existing and project conditions for the SPH on track F critical to the IHNC. Based on these hydrographs, flow profiles for the IHNC were computed and are shown on plate A-3. These profiles represent maximum stages that would result from an occurrence of the SPH on track F. Four profiles are depicted: the profile for existing conditions; the profile for the project as originally authorized, with the rock dike at elevation 13.2; the profile with the rock dike at elevation 7.2; and the profile with the rock dike at elevation 7.2 and the gated outlet structure open. The latter three conditions were computed with the barrier structures at the Chef Menteur and Rigolets Passes in place. The last profile shows that with the rock dike at elevation 7.2 and the gated outlet structure open, stages in the IHNC will be about the same as under existing conditions for track F.

b. Plate A-4 shows the stage hydrographs for existing and project conditions for the SPH on track A critical to the south shore of Lake Pontchartrain. Based on these hydrographs, flow profiles for the IHNC were computed to reflect the maximum stages likely to occur in the canal for the SPH on track A (see plate A-5). The profiles depicted are for the same conditions as described in paragraph 6a above. These profiles show that with the rock dike at elevation 7.2 and the gated outlet structure, open stages in the IHNC will be generally 2 to 3 feet lower than under existing conditions for track A. About two-thirds of all experienced hurricanes in the Lake Pontchartrain area have come from a generally southerly direction (track A) and about one-third has come from the east (track F). It is therefore concluded that the present plan for the Seabrook Complex to include a gated outlet structure in conjunction with the remainder of the barrier plan will, on balance, decrease the flood hazard to IHNC industries on the floodside of the authorized protective works.

SECTION II

DETERMINATION OF FLOW AREA REQUIRED FOR RIPARIAN USE AND SALINITY CONTROL

7. General. The authorizing document provides for construction of a sector-gated navigation lock at the Lake Pontchartrain terminus of the IHNC. The authorized dimensions of the lock are 84 feet wide by 800 feet long, with a sill elevation of -15.0 m.l.g. Extensive model investigations were conducted in connection with the pre-authorization studies to determine the salinity regimen that would result with the overall project in place. Subsequent to project authorization, extensive additional coordination concerning operation of the Seabrook Lock for salinity control and canal flow for riparian needs was accomplished. The Federal and state fish and wildlife agencies have approved the salinity regimen developed in the model studies for operation of the authorized Seabrook Lock with all gates fully open on a continuous basis. The NOPSI (New Orleans Public Service, Inc.), the electric utility for the city of New Orleans, which operates steam electric generating stations on the IHNC and MR-GO and whose needs for cooling water dwarf all other riparian needs, has agreed to the flow regimen corresponding to the above operation of the authorized lock (see paragraph in text of this design memorandum entitled "Coordination with Other Agencies"). A discharge-duration diagram depicting the flow regimen for the typical tidal cycle with the lock operated as described above is included herein as plate A-6.

8. IHNC traffic.

a. Traffic entering and leaving the IHNC at Seabrook includes barge tows, work boats, pleasure craft, and other miscellaneous craft. The Board of Commissioners of the Port of New Orleans (Dock Board) has, since 1946, maintained records of openings of the old Seabrook Bridge and the number of bottoms passed. For the years 1946-1961, annual totals only are available; subsequent to this date, monthly records are available. Bridge openings and bottoms passed are, for all practical purposes, a direct measure of the traffic which would traverse the lock since the restricted vertical clearance of the bridge (2 feet above mean high tide) precludes the passage, with the bridge closed, of all traffic except very small craft such as skiffs and outboard hulls.

b. Opening data for the 21-year period, 1946-1966, are shown in table A-1. The number of openings per year varied from a minimum of 4,497 in 1947 to 12,602 in 1957. The average number of openings per year is indicated to be 7,991. For the period 1962-1966, for which monthly totals are available, openings per month vary from a minimum of 422 in January 1966 to a maximum of 1,147 in May 1965. The above data, plotted by years, are shown on plate A-7.

c. Table A-1, in addition to presenting data on bridge openings, also shows the bottoms passed. For the period 1946-1966, the bottoms passed per year varied from a minimum of 8,177 in 1947 to a maximum of 29,449 in 1957. The average number of bottoms passed per year for the above period is indicated to be 19,157. For the period 1961-1966, the monthly totals varied from a minimum of 1,056 in December 1962 to a maximum of 3,075 in July 1966. Data indicating the number of independent bottoms passed are not available; however, most openings appear to be for a single powered vessel towing one or more barges.

d. Study of existing records and analyses of the various factors relating to traffic indicate little likelihood for radical changes in traffic at the Seabrook site.

9. Flow requirements. As previously stated, the respective interests have agreed that the needs for salinity control and riparian use will be adequately met by maintaining the flow regimen shown on plate A-6. The needs for salinity control and riparian use tend to be in conflict: satisfying the former requires a sharp reduction in existing flows while any reduction is undesirable from the standpoint of riparian use. As has been previously indicated, the NOPSI need for flow for cooling water is the controlling riparian use and its satisfaction will insure that all riparian needs are adequately provided for. NOPSI would prefer a minimum average flow of about 7,000 c.f.s. A flow of this magnitude would, however, produce excessive salinities. As previously indicated, NOPSI has agreed to a flow regimen producing a substantially smaller average flow. Requirements for cooling water are highest in the month of Augusut when both power demand and water temperature in the canal are at their maximum values.

10. Reduction in flow due to traffic.

a. The theoretical capacity of the authorized lock for passing flow is subject to reduction by interruptions occasioned by use of the lock to pass navigation traffic. The magnitude of the reduction would be a function of the number of lockages required and the average lockage time.

b. The maximum number of potential lockages for the month of August, based on the bridge opening data for the 5-year period for which monthly information is available, is indicated to be 1,110 (August 1965, see table A-1). The average number of potential lockages per day, based on the above, is approximately 36.

c. Experience at the Calcasieu and Vermilion Locks on the Gulf Intracoastal Waterway has demonstrated that normal barge traffic may safely transmit an open lock provided the average velocity of flow in the lock is not in excess of about 3 feet per second. Based on the flow regimen shown on plate A-6 and the average lock area of approximately 1,430 square feet, the average velocity would exceed 3 feet per second 29 percent of the time. The average number of lockages per day for the month in question would, therefore, be about 11, assuming that the passages are distributed uniformly throughout the day.

d. The maximum number of bottoms passed in the month of August during the 5-year period for which monthly information is available was 3,017 (August 1966, see table A-1). The number of bridge openings in the same month was 1,072. The approximate number of bottoms per opening was, accordingly, three. As previously stated, the information to determine the independent number of bottoms per opening is not available. In the absence of such information, average lockage time has been estimated as the time required to lock through a tug with two barges. The average lockage will be accomplished against a small head, inasmuch as the head producing 3 feet per second in the lock is 0.4 foot and a head of 3 feet will almost never be exceeded. The estimated average lockage time is developed in the following table:

Operation	Time required--mins.
Close far gate	3
Tow enters lock	5
Close near gate	3
Moor tow	3
Open far gate	3
Tow leaves lock	3
Total	20

e. Based on 11 lockages per day and an average lockage time of 20 minutes, the total time required for lockages is 3.65 hours per day. The flow lost by interruption due to lockages is, therefore, represented by 52% of the area under the discharge-duration diagram for the 7-hour period when the velocity through the lock is in excess of 3 feet per second (see plate A-8). This area represents approximately 75,000,000 cubic feet of discharge or 24.1 percent of the total flow through the lock in an average day. A reduction of this magnitude in the theoretical flow approved by NOPSI is clearly unacceptable.

11. Outlet structure. The availability of sufficient flow to meet the regimen agreed to by fish and wildlife interests and NOPSI can be insured by the addition to the authorized Seabrook Complex of a gated outlet structure. Studies indicate that a gated structure of the same width and depth as the authorized lock will pass approximately 27 percent more flow than the lock under any given head. The additional area required in a gated structure to compensate for the lockage loss will be $\frac{1428}{1.27}$ x 0.241 or approximately 270 square feet.

12. Conclusion. The outlet structure will be located adjacent to and on the east side of the lock. The purposes of the structure will be several: to pass hurricane flows thus providing high stage relief to the IHNC in the event of a hurricane critical to the canal; to guarantee adequate flow for riparian users by compensating for losses in total flow due to lockages; and to maintain salinity control. The outlet structure would be comprised of three bays 32 feet wide with a sill elevation of -15.8. Since the gates are of the vertical lift type and since the available flow area far exceeds the flow area needed for riparian users and for salinity control, as previously determined, the gates could be regulated to satisfy any flow requirements as would be necessary to satisfy these purposes.

TABLE A-1
TABULATION OF SEABROOK BRIDGE OPENINGS AND NUMBER OF VESSELS PASSED FROM 1946 TO 1966

Year	Month	Vessels passed	Openings
1966	Jan	1,722	422
	Feb	2,078	474
	Mar	2,725	552
	Apr	2,436	531
	May	2,428	529
	Jun	2,766	694
	Jul	3,075	1,106
	Aug	3,017	1,072
	Sept	2,397	907
	Oct	2,288	754
	Nov	2,147	727
	Dec	2,024	990
	Total	29,103	8,758
1965	Jan	2,022	791
	Feb	1,762	701
	Mar	2,002	801
	Apr	2,710	929
	May	2,981	1,147
	Jun	2,687	1,051
	Jul	2,523	1,054
	Aug	2,715	1,110
	Sept	1,466	522
	Oct	2,206	491
	Nov	2,298	502
	Dec	1,760	438
	Total	27,132	9,537
1964	Jan	1,352	601
	Feb	1,489	679
	Mar	2,236	981
	Apr	2,231	991
	May	2,279	949
	Jun	2,705	1,013
	Jul	2,870	1,073
	Aug	2,666	1,074
	Sept	2,747	1,074
	Oct	2,429	896
	Nov	2,641	1,026
	Dec	2,341	935
	Total	27,986	11,292

TABLE A-1 (cont'd)

Year	Month	Vessels passed	Openings
1963	Jan	1,263	575
	Feb	1,256	530
	Mar	1,953	747
	Apr	2,043	847
	May	2,054	879
	Jun	2,078	956
	Jul	1,798	863
	Aug	2,105	908
	Sept	1,711	817
	Oct	1,866	839
	Nov	1,390	659
	Dec	1,129	558
	Total	20,646	9,178
1962	Jan	1,371	594
	Feb	1,506	732
	Mar	2,021	875
	Apr	2,078	868
	May	2,217	973
	Jun	2,096	903
	Jul	2,012	895
	Aug	1,899	815
	Sept	1,711	741
	Oct	1,579	655
	Nov	1,440	600
	Dec	1,056	503
	Total	20,986	9,154
1961		20,631	9,514
1960		26,818	11,151
1959		23,833	8,738
1958		27,128	9,660
1957		29,449	12,602
1956		25,210	10,221
1955		18,979	7,222
1954		13,754	6,068
1953		12,896	5,567
1952		14,410	6,026
1951		14,387	6,152
1950		10,222	5,441
1949		10,776	5,509
1948		10,270	5,276
1947		8,177	4,497
1946		9,508	6,244

SECTION III
DETERMINATION OF DESIGN DIFFERENTIAL HEADS

13. General.

a. Studies were performed to determine the different head differentials across the Seabrook Complex for various conditions. These studies involved the analysis of: (1) real time relationships between design hurricane stage-hydrographs in Lakes Borgne and Pontchartrain produced by hurricanes approaching on tracks considered critical to the project area; and (2) the computation of backwater stages through the IHNC from its terminus at Lake Pontchartrain to its junction with the MR-GO. The synthetically computed stage-hydrographs at each terminus of the canal for each track considered are shown on plates A-2, A-4, and A-9. The Lake Borgne stage-hydrograph represents stages in the IHNC at its junction with the MR-GO, and the Lake Pontchartrain stage-hydrograph represents the stages at the junction of the IHNC and the lake. These hydrographs show some lead or lag time in occurrence of peak stages at each terminus and demonstrate that peaks do not necessarily occur coincidentally even though this is possible.

b. The maximum differential stages were determined for each hydrograph and then backwater computations were performed through the IHNC from the lower to the higher stage to determine the stage on the IHNC side of the complex. The response of the stages in the IHNC was assumed to occur instantaneously with fluctuation of the stages in Lakes Borgne and Pontchartrain.

c. Normal maximum tidal fluctuations, as well as the direct and reverse head differentials caused by the design hurricanes, were investigated in these studies. As later tabulated, the normal tidal fluctuations produce differential heads of 4.0 feet across the Complex for both the direct and reverse head conditions. In this report, a direct head is considered to produce lakeward loading; that is, the condition in which the stage on the IHNC side of the Complex is higher than the concurrent stage on the Lake Pontchartrain side. Reverse head is that condition producing gulfward loading; that is, the condition in which the stage on the lakeside is higher than the concurrent stage on the IHNC side.

14. Flow line computations.

a. Since the lock gates will be closed during the discharge of hurricane flows, the control structure must pass all flows until the rock dike is overtopped, at which point the rock dike serves as a weir. For free discharge through the outlet structure, the discharge was computed by the rating curves for the structure. For the condition where free discharge occurs over the rock dike, the discharge over the rock dike was computed with the use of the weir equation $Q = CLH^{3/2}$ where C = 3.0 and L and H are the width of the weir crest and the differential head across the weir, respectively, expressed in feet. For submerged weir conditions, the discharges were computed with the use of the curve shown on figure 5-5 of the fifth edition of Handbook of Hydraulics by Horace Williams King and Ernest F. Brater. When the dike is overtopped the outlet structure is said to be in a submerged condition, i.e., the tailwater depth is approximately 70 percent of the headwater depth. For this condition, the discharges through the structure were determined with the use of results of model studies performed by WES (Waterways Experiment Station), Technical Report 2-633 and Technical Report 2-655.

b. The Manning formula was used in the determination of friction losses along the channel of the IHNC, where the value of "n" selected for the channel was 0.035.

c. Flows in the IHNC will be restricted by the bridges for U. S. Highway 90, the Southern Railway, and the Louisville and Nashville Railroad. Low steel on the railroad bridges is at an elevation of 5 feet and these bridges will be overtopped by tides from critical hurricanes. Flows through the submerged bridge openings were computed with the use of the orifice formula $Q = CA(2gH)^{1/2}$, where A is the area of the submerged opening in square feet, g is the acceleration due to gravity, and using a conservative coefficient of 0.70. Flows over the bridges were computed by means of the weir formula $Q = 2.9LH^{1.5}$. This procedure is outlined in EM 1110-2-1409, Backwater Curves in River Channels.

15. Tabulation of design differential heads. Tables A-2 and A-3 reflect the direct and reverse head stages, respectively, for the design hurricane tracks investigated and additionally for the normal high tide fluctuations, and further reflect the maximum differential head caused by each condition.

TABLE A-2
DIRECT DIFFERENTIAL HEADS

Condition	Track or Direction	Stage on IHNC Side of Complex	Stage on Lake Side of Complex	Differential Head
SPH	F	8.0	-0.2	8.2
SPH	A	6.6	4.5	2.1
SPH	C	7.0	-7.9	14.9
Normal high tide	Lakeward flow	4.0	0.0	4.0

TABLE A-3
REVERSE DIFFERENTIAL HEADS

Condition	Track or Direction	Stage on IHNC Side of Complex	Stage on Lake Side of Complex	Differential Head
SPH	A	7.6	7.9	0.3
Normal high tide	Gulfward flow	0.0	4.0	4.0

16. Maximum design water elevations. In addition to the maximum differential heads likely to occur across the complex are the maximum design water elevations which might obtain at the Complex with the barrier at the Chef and Rigolets Passes in place. The maximum design water elevation is 8.6 on the IHNC side of the Complex and 8.5 on the Lake Pontchartrain side. These stages are both produced by the SPH on track A (see plates A-4 and A-5).

SECTION IV
WAVE DATA

17. Wave data.

a. The parameters which determine wave characteristics are the fetch length, windspeed, duration of wind, and the average depth of water over the fetch. In determining the design wave characteristics, it was assumed that steady state conditions prevail; i.e., the windspeed is constant in one direction over the fetch and blows long enough to develop a fully risen sea. The windspeed (U), the average velocity over the fetch (F), is obtained from isovel patterns for the synthetic hurricane chosen as being critical to the location of interest. The average depth of fetch (d) is the average depth of water as shown by the charts and maps for the area plus the increase in water elevation caused by wind setup. Data necessary to determine design wave characteristics in the vicinity of the structures are shown in table A-4.

TABLE A-4
DATA USED TO DETERMINE WAVE CHARACTERISTICS

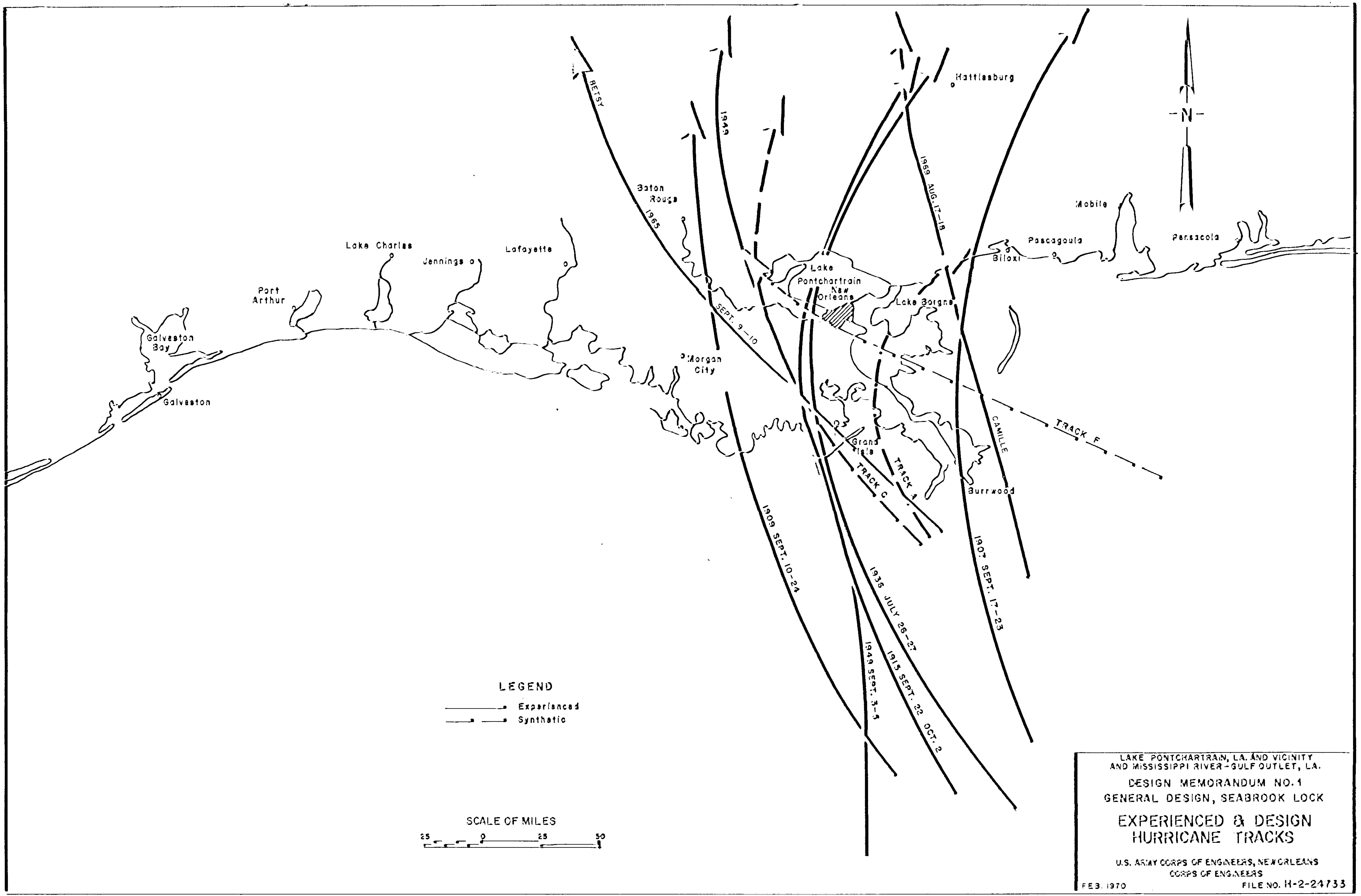
F - Length of fetch	5 miles
U - Windspeed	85 m.p.h.
swl - Stillwater level	8.5 ft. on lakeside of structure site 8.0 ft. avg. for 5-mi. fetch
d - Average depth of fetch	23.6 ft.

The significant wave height (H_s) and wave period (T) were determined from the data in table A-4 above. The equivalent deepwater wave height (H_o') was determined from table D-1 of Technical Report No. 4 prepared by the Coastal Engineering Research Center, June 1966, which relates d/L_o to H/H_o'. The deepwater wave length (L_o) was determined from the equation: $L_o = 5.12\ T^2$. Wave characteristics for the design hurricane which are pertinent to the design of the structure are shown in table A-5 below:

TABLE A-5
WAVE CHARACTERISTICS - DESIGN HURRICANE

H_s	- Significant wave height	8.0 ft.
T	- Wave period	7.1 sec.
L_o	- Deepwater wave length	258 ft.
d/L_o	- Relative depth	0.09147
H_s/H_o'	- Shoaling coefficient	0.9406
H_o'	- Deepwater wave height	8.51 ft.
H_o'/T^2	- Wave steepness	0.169
d_b	- H_o' breaking depth	10.34 ft.
H_{10}	- Average of highest 10% of all waves	10.16 ft.
H_1	- Average of highest 1% of all waves	13.36 ft.

b. The stillwater level used for determination of wave characteristics (el. 8.5) is very nearly equal to the stillwater level at which extreme maximum reverse loading will occur (el. 7.9 lakeside); consequently, the wave characteristics shown above will be used to calculate extreme reverse loading.



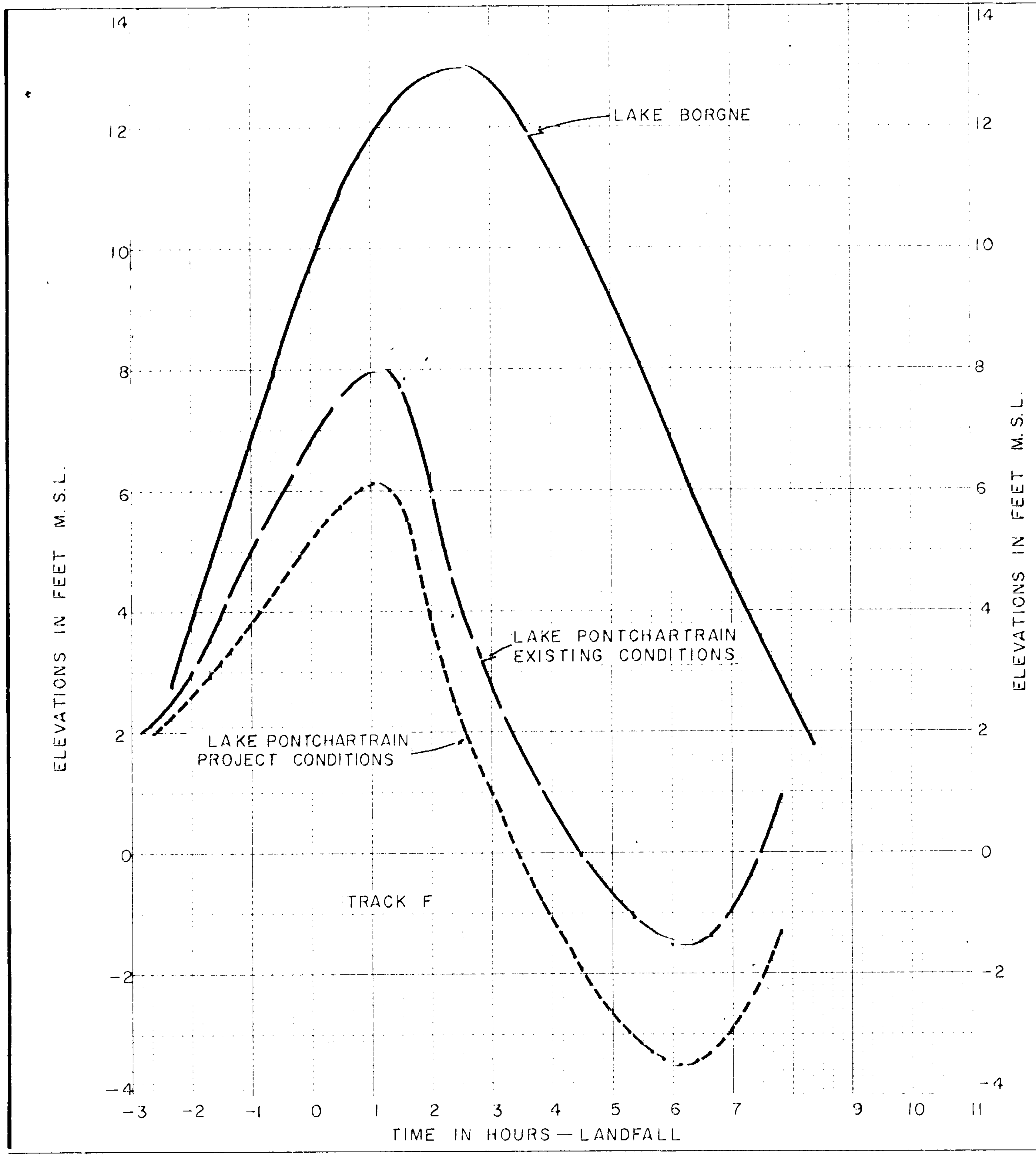
LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER–GULF OUTLET, LA.

DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

EXPERIENCED & DESIGN HURRICANE TRACKS

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

FEB. 1970 FILE NO. H-2-24733

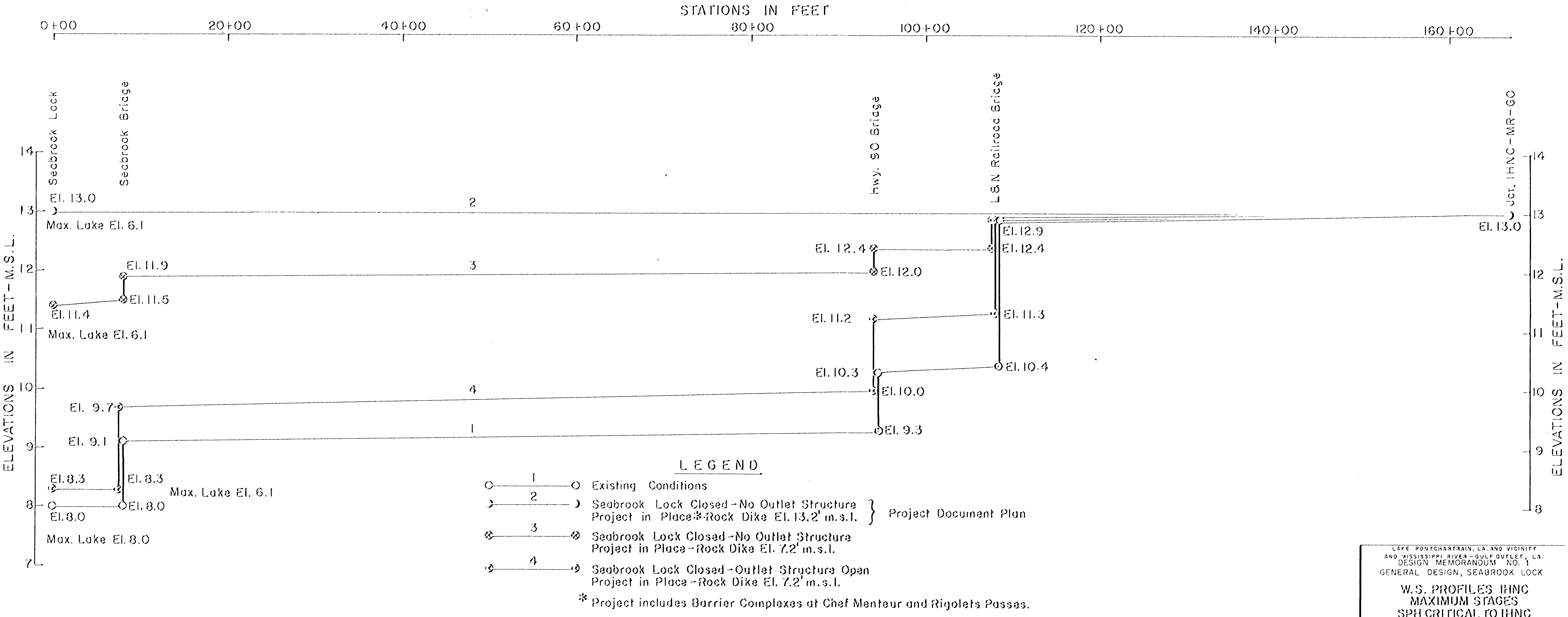


LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

SYNTHETIC STAGE HYDROGRAPHS
LAKES BORGNE AND PONTCHARTRAIN
TRACK F SPH

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

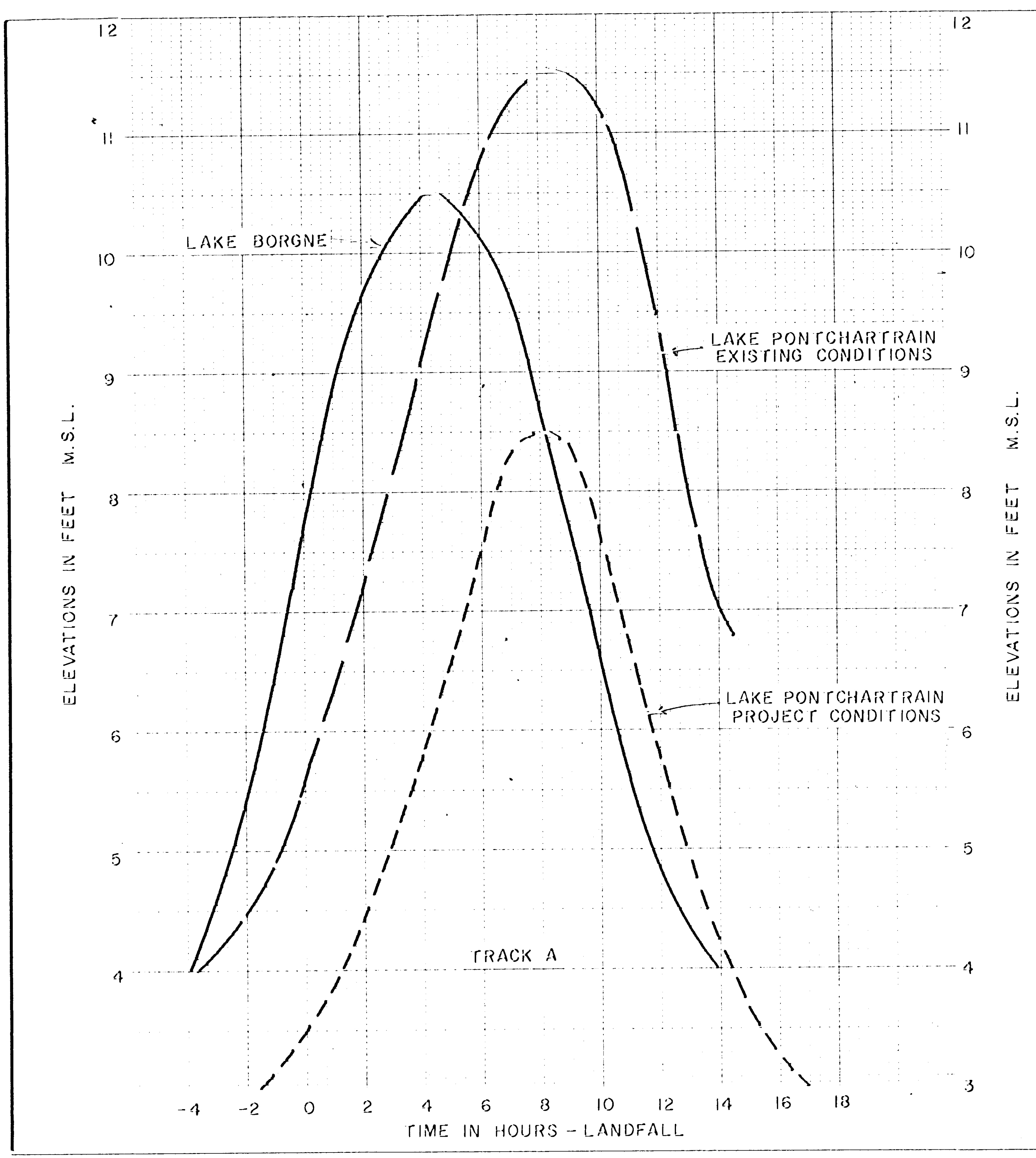
FEB. 1970
FILE NO. H-2-24733



LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

W.S. PROFILES IHNC
MAXIMUM STAGES
SPH CRITICAL TO IHNC

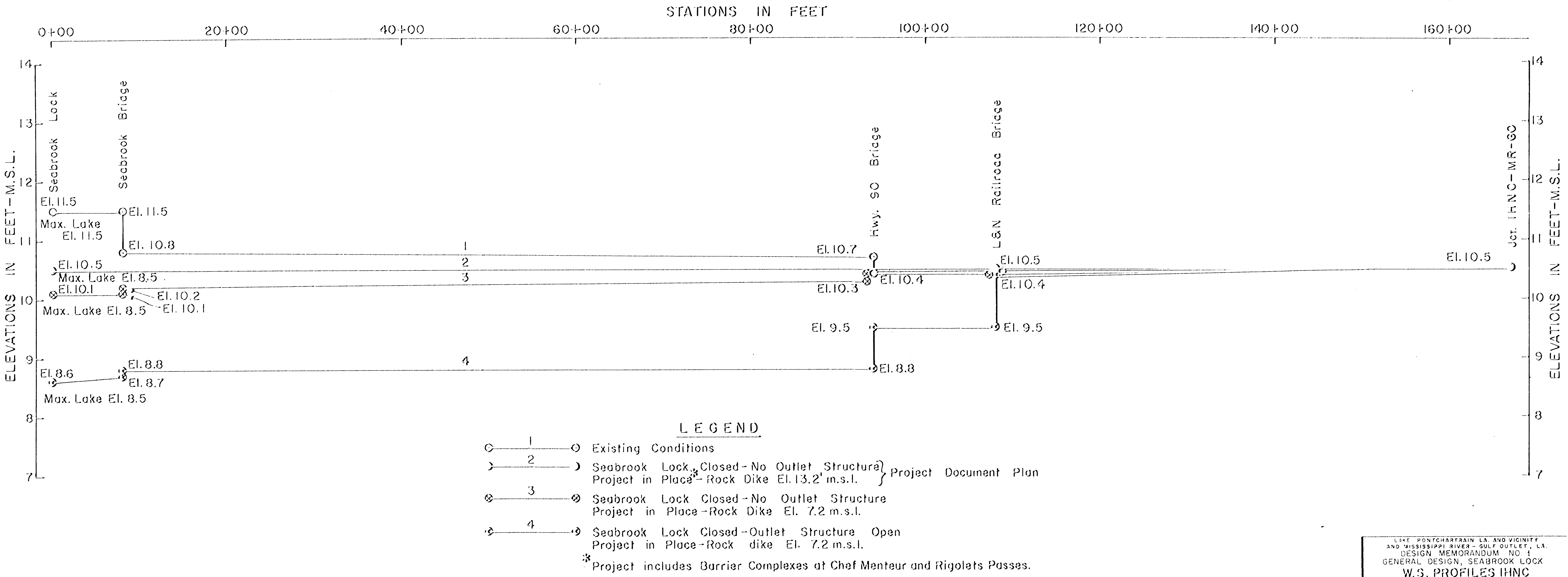
U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970 FILE NO. H-2-24733



LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

SYNTHETIC STAGE HYDROGRAPHS
LAKES BORGNE & PONTCHARTRAIN
TRACK A SPH

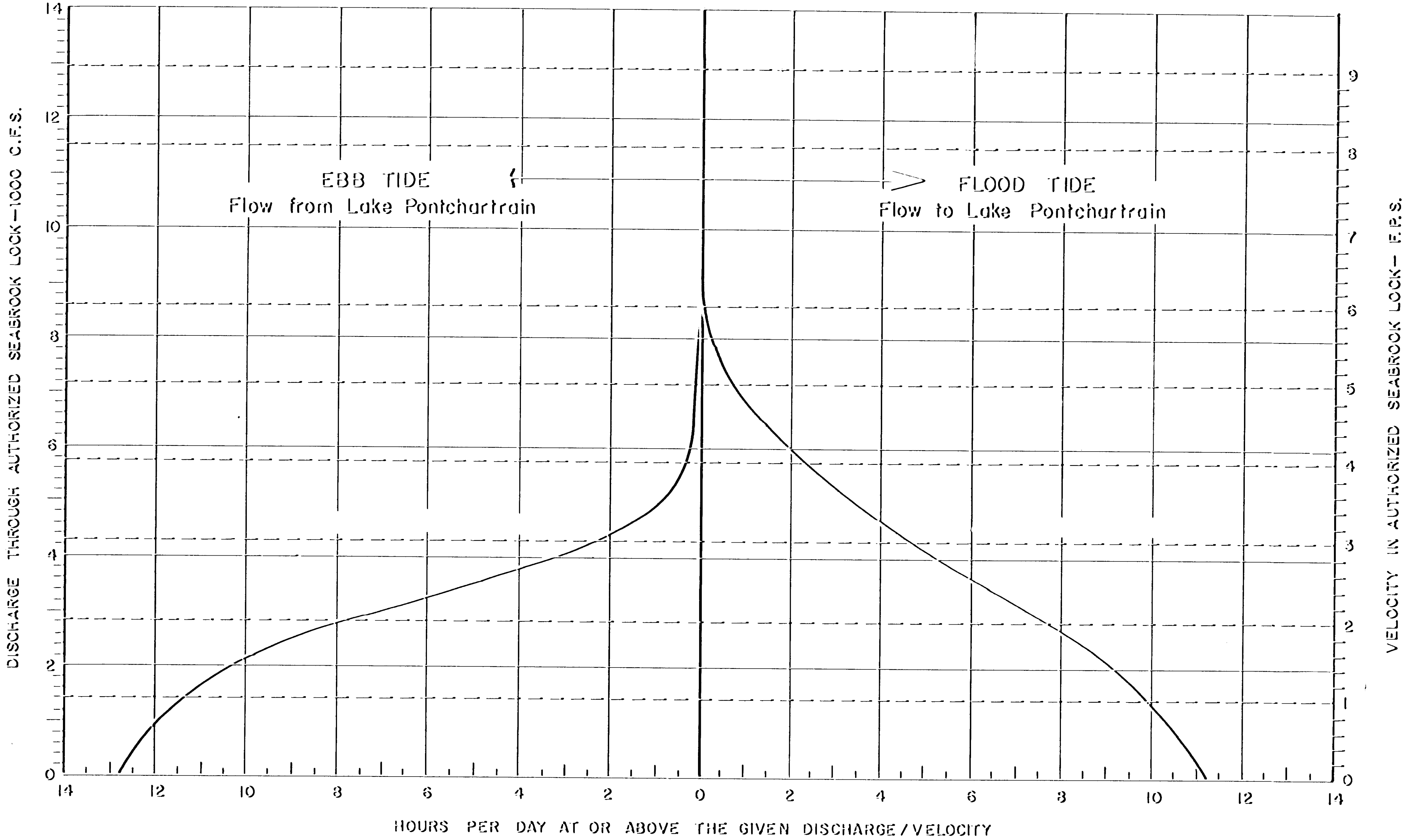
U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970 FILE NO. H-2-24733



LAKE PONTCHARTRAIN LA. AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

W.S. PROFILES IHNC MAXIMUM STAGES SPH CRITICAL TO S. SHORE LAKE PONTCHARTRAIN

U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970
FILE NO. H-2-24733



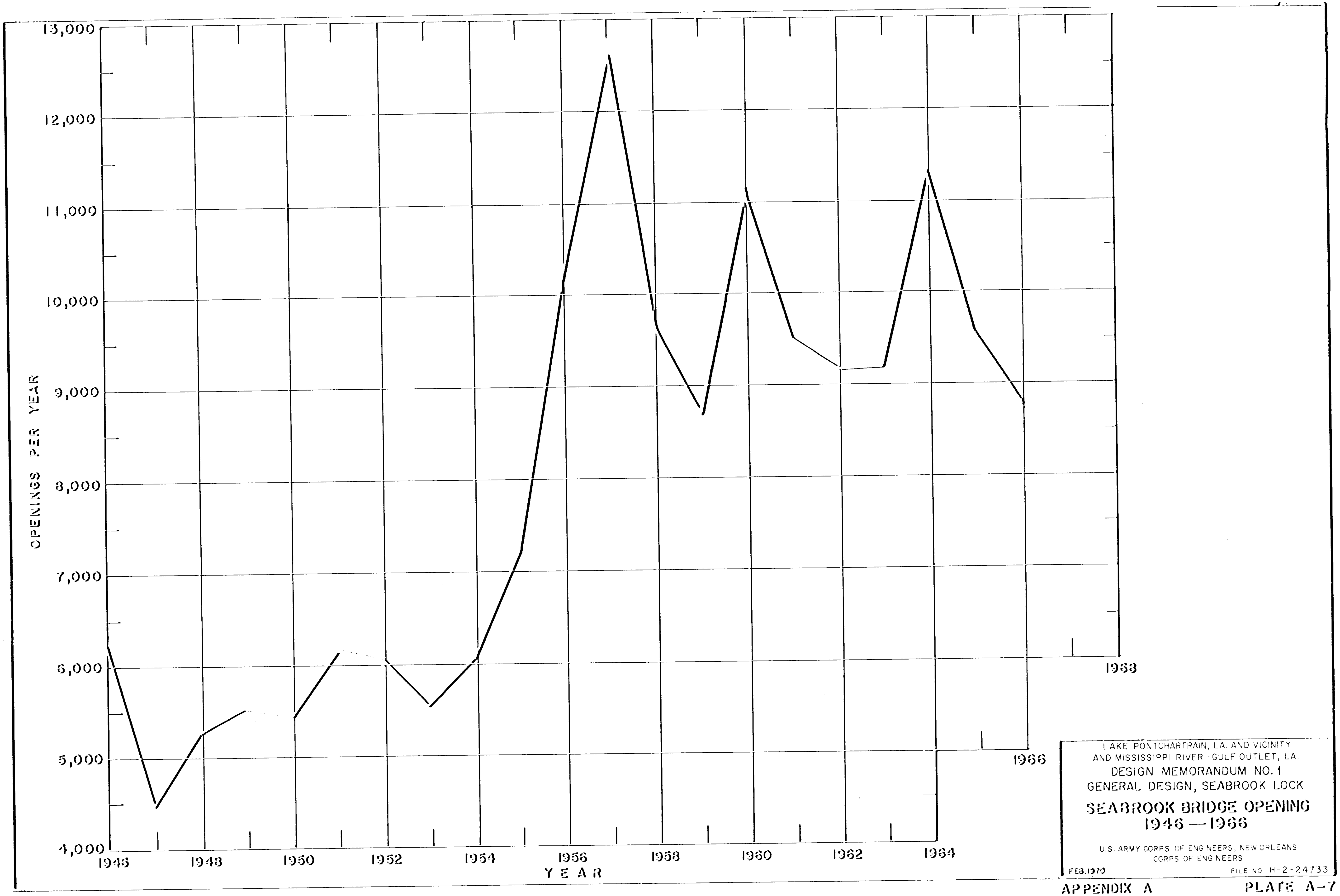
NOTE:

Velocity in lock based on average cross sectional area of 1428 square feet.

LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

DISCHARGE — DURATION DIAGRAM

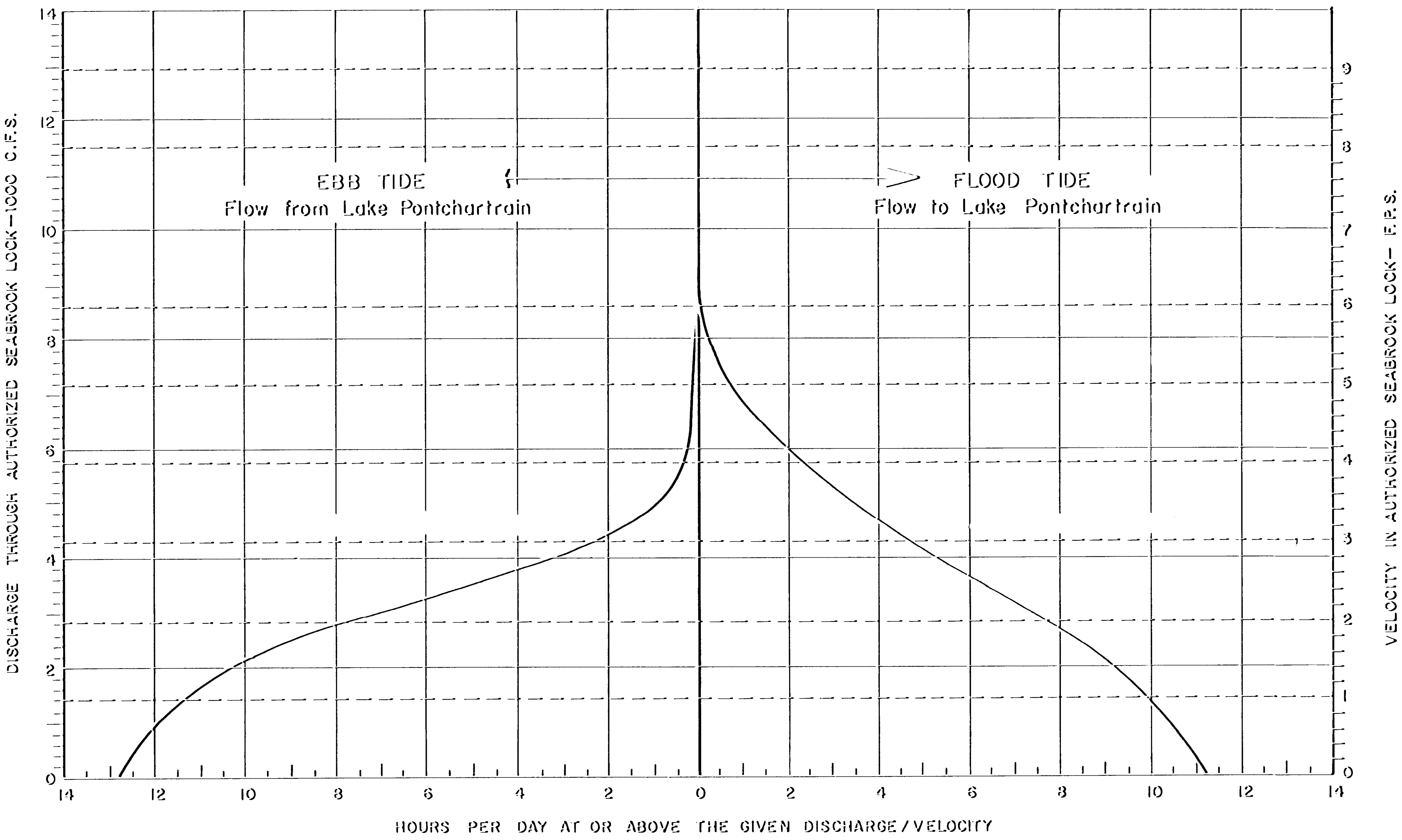
U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970
FILE NO. H-2-24733



LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

SEABROOK BRIDGE OPENING
1946 — 1966

U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970
FILE NO. H-2-24733



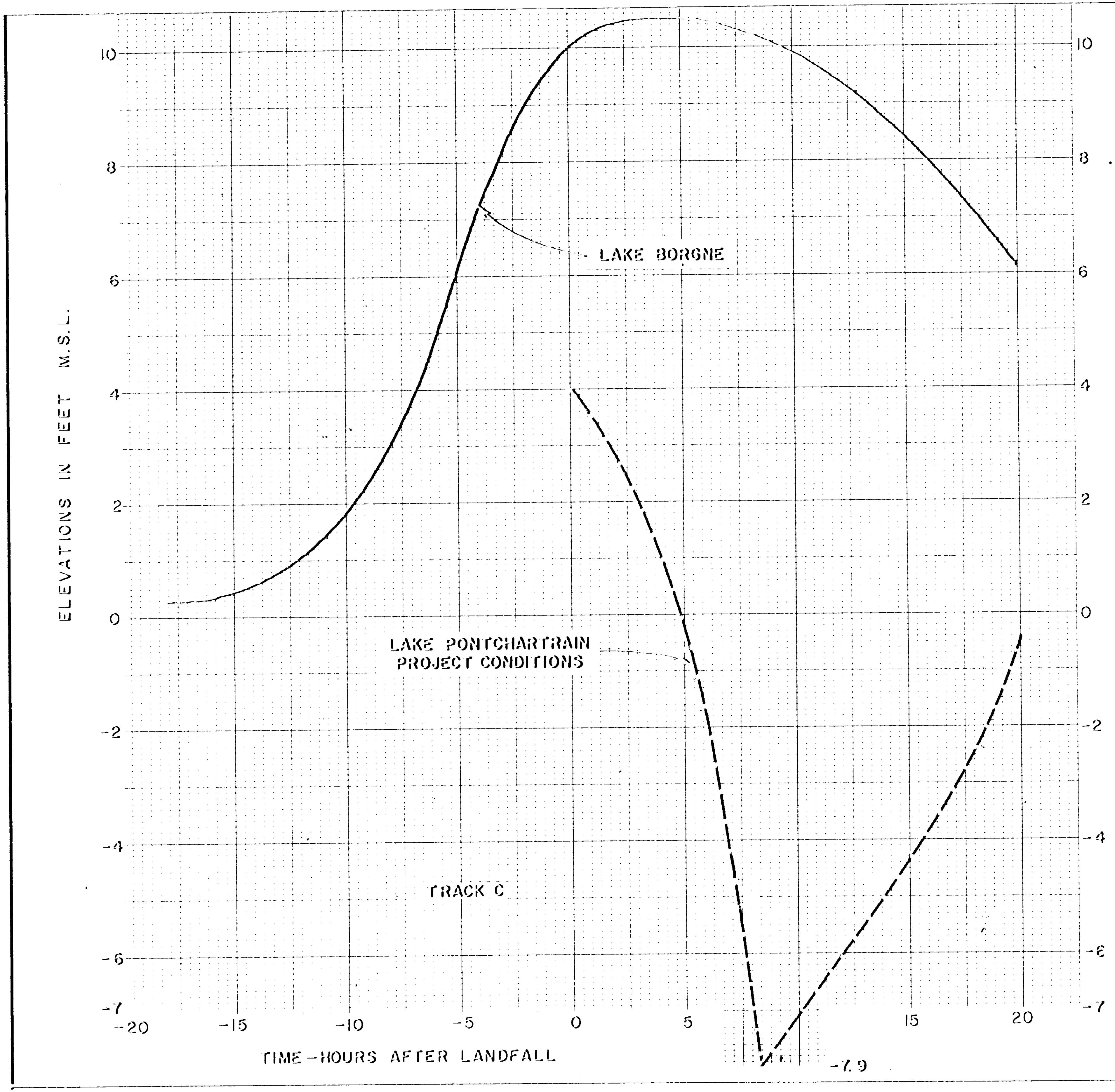
NOTE:

Velocity in lock based on average cross sectional area of 1428 square feet.

LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK

DISCHARGE — DURATION DIAGRAM

U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970
FILE NO. H-2-24733



LAKE PONTCHARTRAIN, LA. AND VICINITY
AND MISSISSIPPI RIVER-GULF OUTLET, LA.
DESIGN MEMORANDUM NO. 1
GENERAL DESIGN, SEABROOK LOCK
SYNTHETIC STAGE HYDROGRAPHS
LAKES BORGNE AND PONTCHARTRAIN
TRACK C SPH
U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1970
FILE NO. H-2-24733

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

APPENDIX B
CORRESPONDENCE RELATIVE TO COORDINATION WITH OTHER AGENCIES

Letters to U.S. Fish and Wildlife Service, Atlanta, Georgia, dated 4 November 1966, 18 January 1967, 17 April 1967 and 19 April 1967; and replies of 26 April 1967 and 7 June 1967.

Letter to Federal Water Pollution Control Administration, Dallas, Texas, dated 19 April 1967, reply dated 23 June 1967; and acknowledgement dated 25 July 1967

Letter to Louisiana Wild Life and Fisheries Commission, New Orleans, Louisiana, dated 17 April 1967; reply dated 2 May 1967; and acknowledgment dated 8 May 1967.

Letter to New Orleans Public Service, Inc., New Orleans, Louisiana, dated 1 February 1967 and reply dated 10 February 1967

LMNED-PP 4 November 1966

Mr. Walter A. Gresh, Regional Director
U. S. Department of the Interior
Fish and Wildlife Service
Bureau of Sport Fisheries and Wildlife
Peachtree-Seventh Building
Atlanta, Georgia 30323

Dear Mr. Gresh:

As you know, this office is now engaged in detailed planning for the "Lake Pontchartrain, La. and Vicinity," hurricane protection project. One feature of the project is the Seabrook Lock--a multipurpose structure at the lakeward end of the Inner Harbor Navigation Canal (IHNC) for control of hurricane inflow, and for mitigating undesirable alterations in the salinity regimen in Lake Pontchartrain and the marshes adjacent to Lake Borgne and excessive currents in the IHNC, both of which have developed as a result of the construction of the Mississippi River-Gulf Outlet (MR-GO). Based on studies made in connection with detailed planning, a tentative operating plan for the lock has been developed which we are presenting herein for your consideration and comment.

It should be borne in mind that any operating plan must provide some flow for riparian uses. Consideration of data developed in the model studies made prior to authorization of the project indicates that the requirements for salinity control are in conflict with those for riparian use. Generally speaking, the requirements for salinity control would dictate use of the lock to drastically reduce interchange of flow between Lake Pontchartrain and the MR-GO, while those for riparian use would dictate minimum interference by the lock to such interchange, consistent with adequate control of currents in the IHNC. A summary of salinity data for the high and low inflow years used in the model studies is inclosed. These data show that if all interchange between the IHNC and Lake Pontchartrain were eliminated, salinity conditions would approximate those which obtained prior to construction of the MR-GO, while, if the lock were allowed to remain open on a continuous basis, average salinities in Lake Pontchartrain would be from two to three times higher. In deriving a plan of operation, therefore, a compromise between the conflicting requirements must be reached.

Model data are not available for operation of the lock for discharge settings other than full diversion capacity. However, such data may be obtained by interpolation between model data for the lock closed and fully open. Salinity data obtained in this manner for operation at one-third of full capacity are shown on the inclosure. It will be noted that this operation would result in salinities in the marsh adjacent to the MR-GO which are generally little different than those obtaining prior to construction of the MR-GO. Salinities in Lake Pontchartrain would, however, be substantially higher, particularly in high inflow years.

Specific model data concerning operation of the lock to provide a diversion rate on the flooding period of the tidal cycle different than that on the ebb are not available. However, it seems evident that, if the discharge for ebb flow (flow from Lake Pontchartrain into the IHNC) were increased without changing the discharge for flood flow, little change would result in the salinities shown for the one-third capacity operation on the inclosed summary, and that any change which might result would tend to be in the direction of lower, rather than higher, salinities. Such an operation would better serve the needs for riparian water use, inasmuch as it would roughly double the average flow in the IHNC and MR-GO.

The matter of operation of Seabrook Lock and the requirements for salinity control were discussed at some length during a meeting held in this office on 11 May 1966, at which representatives of your Service and the Louisiana Wild Life and Fisheries Commission were present. As we understand it, your office considers that some increase in salinities over those representative of pre-MR-GO conditions would probably be desirable, but that radical increases should be avoided. We propose to operate the lock so as to utilize its full discharge capacity during periods when the flow is moving from Lake Pontchartrain into the IHNC, and to reduce the diversion to one-third of full capacity during periods of opposite flow. We consider that the data shown on the survey for the one-third capacity operation are representative of what salinity conditions would be under the operation proposed.

Your comments regarding the proposed operating procedure are requested. Inasmuch as further planning is dependent upon resolution of this matter, your cooperation in furnishing comments at the earliest practicable date will be very much appreciated. We shall be pleased to meet with you in this office, at your convenience, to discuss the proposed procedure if you feel that such discussions would be of value.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

Incl
  Salinity summary

Copies furnished: w/incl
  U.S.Fish & Wildlife Svc, Vicksburg
  La. Wild Life & Fish. Comm.

Note: Interruptions for lockages have been neglected. Such interruptions would not significantly alter the data shown.

4 Nov 66

LAKE PONTCHARTRAIN, LA. & VICINITY
SEABROOK LOCK

Salinity Summary

Source: Technical Report No. 2-636 dtd Nov 1963
U. S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi

	Average salinity (PPM) Lake Pontchartrain Min.	Max.	Avg.	Lake Borgne (marsh adjacent to MR-GO) Min.	Max.	Avg.
	HIGH INFLOW YEAR					
High inflow year base test (Before MR-GO)	650	1850	1056	850	6750	2564
Seabrook Lock in place and:						
1. Operated at full disch. capacity	1950	3800	2622	2600	6250	3707
2. Operated at 1/3 disch. capacity*	1117	2533	1593	1367	6283	2856
3. Closed continuously	700	1900	1079	750	6300	2430
	LOW INFLOW YEAR					
Low inflow year base test (Before MR-GO)	1675	3550	2278	3275	10125	6463
Seabrook Lock in place and:						
1. Operated at full disch. capacity*	3433	5508	4129	3050	10400	6833
2. Operated at 1/3 disch. capacity*	2261	4203	2895	3467	10217	6595
3. Closed continuously*	1675	3550	2278	3275	10125	6463

*Interpolated from available model study data.

LMNED-PP 19 January 1967

Mr. Walter A. Gresh, Regional Director
U. S. Department of the Interior
Fish and Wildlife Service
Peachtree-Seventh Building
Atlanta, Georgia 30323

Dear Mr. Gresh:

Please refer to our letter dated 4 November 1966 which forwarded a tentative operating procedure for the Seabrook Lock which was authorized for construction under the "Lake Pontchartrain, La. and Vicinity," project.

A meeting to discuss the tentative procedure was held in this office on 17 January 1967. Your office was represented by Messrs. Smith and Cammberlain representatives of the Bureau of Sports Fisheries and Wildlife in Vicksburg, Mississippi, the Bureau of Commercial Fisheries in Galveston, Texas, and the Louisiana Wild Life and Fisheries Commission also participated in the meeting. The U. S. Army Corps of Engineers was represented by Mr. J. C. Baehr and other personnel of this office, and Mr. Henry Simmons of the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Available data regarding effects of the "Mississippi River-Gulf Outlet, La.," and "Lake Pontchartrain, La. and Vicinity," projects on salinities in Lake Pontchartrain and the marshes adjacent to the Mississippi River-Gulf Outlet are contained in Technical Report No. 2-636, dated November 1963, which was published by the Waterways Experiment Station and contains the results of extensive model investigations undertaken by them in connection with the preauthorization planning for the "Lake Pontchartrain, La. and Vicinity," project. In a base test reflecting conditions prior to construction of the Mississippi River-Gulf Outlet, maximum, minimum, and average salinities in Lake Pontchartrain and in the marshes adjacent to the Mississippi River-Gulf Outlet were determined for both a high inflow and a low inflow year. Additional tests were run to determine salinities with the Mississippi River-Gulf Outlet in place, both with and without control works at Seabrook. A gated control structure was used in the model tests and salinities determined for the full discharge capacity of the structure, and for one-third and two-thirds reductions in the discharge capacity of the structure. The results for the base test (before Mississippi River-Gulf Outlet) and the controlled case with Mississippi River-Gulf Outlet in place and structure capacity reduced by two-thirds are summarized below:

Average Salinity (PPM)

High Inflow Year

	Lake Pontchartrain			Marsh adjacent to MR-GO		
	Min.	Max.	Avg.	Min.	Max.	Avg.
Base test	650	1,350	1,056	850	6,750	2,564
Controlled case-- two-thirds reduction in structure capacity	1,950	3,800	2,622	2,600	6,250	3,707

Low Inflow Year*

	Lake Pontchartrain			Marsh adjacent to MR-GO		
	Min.	Max.	Avg.	Min.	Max.	Avg.
Base test	1,675	3,550	2,278	3,275	10,125	6,463
*Controlled case-- two-thirds reduction in structure capacity	3,433	5,508	4,120	3,850	10,400	6,830

*Interpolated from model data.

It will be noted that the above values for the controlled case are the same as those previously furnished for the controlled case with the authorized Seabrook Lock operated at full discharge capacity, it having been determined by the Waterways Experiment Station that the lock operated at full discharge capacity is equivalent to the control structure used in the model test operated with two-thirds reduction in discharge capacity.

Based on our discussions at the meeting, we understand that the salinity conditions represented by the data for the controlled case as given above are considered by you to be acceptable insofar as the preservation and/or enhancement of fish and wildlife values is concerned. We further understand that you consider the details of the control works necessary to produce, in the prototype, the salinity conditions corresponding to the model data shown, to be a matter for engineering determination by the Corps of Engineers.

It is recognized that there is some element of uncertainty in regard to how closely actual conditions subsequent to construction will follow the results indicated by the model tests. It is, accordingly, agreed that corrective action would have to be taken in the event that postconstruction experience should indicate conditions markedly different from those indicated by the model data.

We are presenting flow data based on the above considerations to the New Orleans Public Service, Inc., and upon receipt of their concurrence, shall resume detailed planning for the Seabrook Lock. We shall be pleased to keep you informed as the design progresses.

Your cooperation in resolving this matter is appreciated.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

Copies furnished:
U.S. Fish & Wildlife Service, Vicksburg, Miss.
La. Wild Life & Fisheries Comm. New Orleans, La.

WES, ATTN: Mr. Henry Simmons

Ch, Hydraulics Br., Engrg. Div.

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
PEACHTREE-SEVENTH BUILDING
ATLANTA, GEORGIA 30323

April 26, 1967

District Engineer
U. S. Army, Corps of Engineers
P. O. Box 60267
New Orleans, Louisiana 70160

Dear Sir:

Reference is made to your letter of January 18, 1967, regarding tentative operating procedures for Seabrook Lock, a feature of the Lake Pontchartrain, Louisiana, and Vicinity project. Based on applicable model studies and our discussions in your office on January 17, it appears at this time that operation of the lock at full discharge capacity throughout the full tidal cycle can be tolerated insofar as fish and wildlife resources are concerned.

The Louisiana Wild Life and Fisheries Commission, in its letter of April 13 commenting on your letter of January 18, points out, however, that the 5.5 p.p.t. salinity indicated for Lake Pontchartrain in a low inflow year approaches the upper tolerance limit for the extremely valuable Rangia clam resource. In view of this, and since, as you point out, there is some uncertainty as to how closely post-construction conditions will follow the results indicated by the model, we strongly feel that operations must be modified if necessary to insure continued desirable salinity levels.

We suggest, therefore, that conditions in Lake Pontchartrain be monitored periodically after construction, and request that the need for corrective action be determined jointly by this Bureau, the Bureau of Commercial Fisheries, and the Louisiana Wild Life and Fisheries Commission. In this regard, we again call to your attention the need for maintaining full flexibility of operations to meet the requirements both of riparian use and of salinity control.

Your letter of April 17 regarding alterations in lock design has been received and is now being reviewed. Personnel of our Vicksburg, Mississippi, field office will be contacting your staff to obtain additional information as necessary. We will provide our comments as soon as we possibly can.

Sincerely yours,

Walter A. Gresh
Regional Director

LMNED-PP

17 April 1967

Mr. Walter A. Gresh, Regional Director
U. S. Department of the Interior
Fish and Wildlife Service
Bureau of Sport Fisheries and Wildlife
Peachtree-Seventh Building
Atlanta, Georgia 30323

Dear Mr. Gresh

This office is presently engaged in preparing a general design memorandum for the Seabrook Lock, construction of which was authorized by the Flood Control Act of 1965 (Public Law 89-298, approved 27 October 1965).

The lock is to serve the multiple purposes of hurricane flood control, salinity control, and current control. The general layout of the lock is shown on inclosure 1, and additional descriptive material is contained in House Document 231, 89th Congress, 1st Session. The following significant alterations in the authorized lock will be incorporated into the design:

a. The Chief of Engineers has approved a change in the controlling elevation of the lock from 13.2 feet mean sea level to 7.2 feet mean sea level. This change will be effected by lowering the crown of the rock dike which will tie the lock to the levee system.

b. An auxiliary structure will be provided to permit diversions for salinity control and riparian use during periods when the lock is passing traffic. Your attention is invited to our meetings and correspondence relative to the matter of salinity control, and in particular to our letter of 18 January 1967 which sets forth the salinity regimen that the Seabrook works will be operated to maintain.

Because of the urgent nature of the work covered by the design memorandum, we are operating on a much compressed planning schedule. It will, accordingly, be very much appreciated if your comments are provided not later than 14 July 1967.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

1 Incl
Dwg - Seabrook Lock
(file H-2-22077, plate 9)

Copies furnished: w/o incl
U.S.Fish & Wildlife Service.
Vicksburg, Miss.
La. Wild Life & Fish.
N.O.,La.

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
PEACHTREE-SEVENTH BUILDING
ATLANTA, GEORGIA 30323

June 7, 1967

District Engineer
U. S. Army, Corps of Engineers
P. O. Box 60267
New Orleans, Louisiana 70160

Dear Sir:

This letter has been prepared in response to your request of April 17, 1967, for Bureau comments on design alterations in Seabrook Lock, a feature of the authorized Lake Pontchartrain, Louisiana, and Vicinity project. These are submitted in accord with the Fish and Wildlife Coordination Act (48 Stat. 401, as amended; 16 U.S.C. 661 et seq.).

According to your letter, the following changes have been made in the design of the structure:

a. The crown of the rock dike tying the lock to the Lake Pontchartrain levee system has been lowered from a controlling elevation of 13.2 feet m.s.l. to 7.2 feet m.s.l.

b. An auxiliary water-control structure will be located in the rock dike. This structure will permit flow diversions for riparian use when the lock is passing traffic and also provide salinity control.

Review of these modifications indicates that lowering the controlling elevation of the rock dike to elevation 7.2 feet will have no effect on fish and wildlife resources. On the other hand, the auxiliary water-control structure should provide a more flexible system for salinity control in Lake Pontchartrain.

Your letter to us of January 18, 1967, set forth maximum facilities predicted by model tests with the Seabrook structure in place. The salinity ranges as predicted appear to be acceptable for the preservation of fish and wildlife resources. To assure that proper salinity ranges are maintained, we wish to take this opportunity to suggest a salinity surveillance system be located in Lake Pontchartrain after the Seabrook structure is in place. The monitoring of this system would provide a basis for maintaining lake salinities through operation of the Seabrook Lock.

It is suggested that your agency, the Louisiana Wild Life and Fisheries Commission, and this Bureau jointly develop a plan for the establishment of a salinity surveillance system.

We appreciate this opportunity to comment on your general design memorandum. A copy of this letter is being sent to the Louisiana Wild Life and Fisheries Commission.

Sincerely yours,

Walter A. Gresh
Regional Director

LMNED-PP

19 June 1967

Mr. Walter A. Gresh
Regional Director
U. S. Department of the Interior
Fish and Wildlife Service
Bureau of Sport Fisheries and Wildlife
Peachtree-Seventh Building
Atlanta, Georgia 30323

Dear Mr. Gresh:

Thank you for your letter dated 7 June 1967 relative to alterations in the authorized Seabrook Lock feature of the "Lake Pontchartrain, La. and Vicinity," project.

Our current data collection program includes extensive coverage of Lake Pontchartrain salinities. Upon completion of the lock, we shall expand this program, if necessary, to present an adequate evaluation of the effects of lock operation on the salinity regimen, and a determination as to the extent that the lock operation is producing the salinity regimen indicated by the model data previously furnished you. We are pleased with your suggestion that your agency and the Louisiana Wild Life and Fisheries Commission participate in the development of a salinity surveillance system and shall contact you further in this regard at an appropriate time.

Again, your cooperation in providing comments on the Seabrook Lock is very much appreciated.

Sincerely yours,

GEORGE H. HUDSON
Acting District Engineer

LMNED-PP 19 April 1967

Mr. Jerome H. Svore, Regional Director
U. S. Department of the Interior
Federal Water Pollution Control Administration
1114 Commerce Street
Dallas, Texas 75202

Dear Mr. Svore:

This office is presently engaged in preparing a general design memorandum for the Seabrook Lock, construction of which was authorized by the Flood Control Act of 1965 (Public Law 89-298, approved 27 October 1965).

The lock is to serve the multiple purposes of hurricane flood control, salinity control, and current control. The general layout of the lock is shown on inclosure 1, and additional descriptive material is contained in House Document 231, 89th Congress, 1st Session. The following significant alterations in the authorized lock will be incorporated into the design:

a. The Chief of Engineers has approved a change in the controlling elevation of the lock from 13.2 feet mean sea level to 7.2 feet mean sea level. This change will be effected by lowering the crown of the rock dike which will tie the lock to the levee system.

b. An auxiliary structure will be provided to permit diversion for salinity control and riparian use during periods when the lock is passing traffic. In connection with the operation of Seabrook Lock, your attention is invited to our letter of 18 January 1967 (inclosure 2) to the U. S. Fish and Wildlife Service, Atlanta, Georgia, indicating the salinity regimen that the lock will be operated to maintain.

Because of the urgent nature of the work covered by the design memorandum, we are operating on a much compressed planning schedule. It will, accordingly, be very much appreciated if your comments are provided not later than 14 July 1967.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

2 Incl
Dwg - Seabrook Lock
(file H-2-22077, plate 9)
Cy ltr 18 Jan 67

UNITED STATES
DEPARTMENT OF THE INTERIOR
FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
South Central Region
1114 Commerce Street
Dallas, Texas 75202

June 23, 1967

Re: LMNED-PP

District Engineer
U. S. Army Engineer District, New Orleans
P. O. Box 60267
New Orleans, Louisiana 70160

Dear Sir:

Reference is made to your letters of April 19 and April 21, 1967 initiating coordination of the general design memorandum for the Seabrook Lock and the Lake Pontchartrain Barrier Plan.

We have had an opportunity to review the information submitted in accordance with Executive Order 11288, Section 1, paragraph (7) and Section 6 and find as follows:

> Every attempt should be made to minimize water quality degradation during actual construction and to control spoils that would cause highly turbid waters.
>
> It is desirable that the water quality control structures be constructed and operated so as to prevent changes in present water quality and to insure that ecological conditions remain unchanged.
>
> The Louisiana State Board of Health commented on the lack of information regarding insect control. If the water level in Lake Pontchartrain is raised so as to flood the lowlands bordering the lake, severe mosquito breeding problems may result.
>
> All contractors should take precautions to prevent water pollution by accidental spillage of petroleum products or other harmful materials i.e. insecticides. Also, all contractors should provide and maintain sanitation facilities that will adequately treat domestic wastes to conform with Federal and local health regulations.

Please advise this office (Attention: Federal Activities Coordinator) of significant changes from the plan presented.

The comments of the State of Louisiana Stream Control Commission and the Louisiana State Board of Health have been incorporated in our review.

Your cooperation in carrying out the requirements of the Order is appreciated.

Sincerely yours,

John P. [illegible]

for WILLIAM C. GALEGAR
Regional Director

cc: Louisiana State Board of Health
Louisiana Stream Control Commission

LMNED-PP

25 July 1967

Mr. William C. Galegar, Regional Director
U. S. Department of the Interior
Federal Water Pollution Control Administration
1114 Commerce Street
Dallas, Texas 75202

Dear Mr. Galegar:

Thank you for your letter dated 23 June 1967 relative to the general design memorandum for the Lake Pontchartrain Barrier Plan and Seabrook Lock features of the "Lake Pontchartrain, La. and Vicinity," project.

Provisions to ensure that the objectives of your comments relative to water quality degradation during construction, control of accidental spillages, and maintenance of adequate sanitary facilities by construction contractors will be incorporated into our construction plans and specifications. With respect to the concern of the Louisiana State Board of Health relative to mosquito breeding problems in the event that the average level of Lake Pontchartrain is raised, we would observe that the plan will not result in any increase in the average lake level, but will serve only to lower lake stages during hurricanes.

The Seabrook Lock will be operated to provide a desirable salinity regimen in Lake Pontchartrain. The plan of operation will be developed with the advice of the state and Federal fish and wildlife agencies. We shall be pleased to seek the advice of your agency also when the plan is prepared.

Your cooperation in providing comments on the project is very much appreciated.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

LMNED-PP 17 April 1967

Mr. Leslie L. Glasgow, Director
Louisiana Wild Life and Fisheries Commission
400 Royal Street
New Orleans, Louisiana 70130

Dear Mr. Glasgow:

This office is presently engaged in preparing a general design memorandum for the Seabrook Lock, construction of which was authorized by the Flood Control Act of 1965 (Public Law 89-298, approved 27 October 1965).

The lock is to serve the multiple purposes of hurricane flood control, salinity control, and current control. The general layout of the lock is shown on inclosure 1, and additional descriptive material is contained in House Document 231, 89th Congress, 1st Session. The following significant alterations in the authorized lock will be incorporated into the design:

a. The Chief of Engineers has approved a change in the controlling elevation of the lock from 13.2 feet mean sea level to 7.2 feet mean sea level. This change will be effected by lowering the crown of the rock dike which will tie the lock to the levee system.

b. An auxiliary structure will be provided to permit diversions for salinity control and riparian use during periods when the lock is passing traffic. Your attention is invited to our meetings relative to the matter of salinity control, and to our letter of 13 January 1967 to the U. S. Fish and Wildlife Service, Atlanta, Georgia, which sets forth the salinity regimen that the Seabrook works will be operated to maintain.

Because of the urgent nature of the work covered by the design memorandum, we are operating on a much compressed planning schedule. It will, accordingly, be very much appreciated if your comments are provided not later than 14 July 1967.

Sincerely yours,

1 Incl
Dwg - Seabrook Lock
(file H-2-22077, plate 9)

THOMAS J. BOWEN
Colonel, CE
District Engineer

Copy furnished: Louisiana Wild Life & Fisheries Commission
River Basin Section
Baton Rouge, La. 70804 w/o incl

U.S. Fish & Wildlife Service w/o incl
Atlanta, Ga., & Vicksburg, Miss.

LOUISIANA WILD LIFE AND FISHERIES COMMISSION
WILD LIFE AND FISHERIES BUILDING
400 ROYAL STREET
NEW ORLEANS, LOUISIANA 70130

May 2, 1967

District Engineer
U. S. Army Corps of Engineers
New Orleans District
P. O. Box 60267
New Orleans, Louisiana 70160

Dear Sir:

Reference is made to your letter of April 20, 1967, and your letter of January 18, 1967, concerning the Lake Pontchartrain Barrier Plan feature of the Lake Pontchartrain, Louisiana, and Vicinity Project and for the Seabrook Lock segment of this same project.

After reviewing the information contained in the barrier plan, we do not have any specific considerations or additional recommendations regarding this segment of this project. However, we are extremely concerned about the salinity level and the opportunity to provide for passage of water at the Seabrook Lock to control salinities and allow continued water exchange.

In your letter of January 18, 1967, the average salinities given for Lake Pontchartrain with the Seabrook Lock structure in place are within ranges considered necessary to maintain present fish and wildlife resources associated with this area. However, the 5.5 p.p.t. maximum salinity indicated for a low inflow year is approaching the upper tolerance level for the Rangia clam which is an extremely valuable resource associated with Lake Pontchartrain and the basis for a sizeable industry in Louisiana.

We are naturally concerned about the possible effect higher salinities will have on future clam production. It is known that the Rangia species are brackish water clams and can survive salinities approaching 8 p.p.t. However, we doubt seriously if they will continue to reproduce and survive in the higher salinity ranges. Therefore, we recommend that salinities for Lake

Pontchartrain be maintained as near as possible to conditions existing prior to the construction of the Mississippi River Gulf-Outlet Project and that the maximum salinity range not be allowed to exceed 8 p.p.t. for any extended period of time.

In the event the actual conditions of the Mississippi River Gulf-Outlet Project in place do not closely follow the model test results, we strongly recommend that provisions be installed in the Seabrook Lock to allow salinities in Lake Pontchartrain to be adjusted as may be necessary for the maintenance of fish and wildlife resources. We feel the inclusion of an auxiliary control structure in the lock design is necessary to provide for the passage of water for salinity control and other uses when the lock is handling traffic and cannot serve this important purpose.

We appreciate the opportunity to review and comment on these segments of the Lake Pontchartrain and Vicinity Project. In addition, we request to be kept informed on the progress of this work and in the event additional modifications are contemplated, we would like the opportunity to review and offer additional comments.

Sincerely yours,

Leslie L. Glasgow
Director

LLG:MWS/js

cc: U. S. Fish and Wildlife Service
Bureau of Sport Fisheries and Wildlife
Atlanta, Georgia

LMNED-PP

8 May 1967

Mr. Leslie L. Glasgow, Director
Louisiana Wild Life and Fisheries Commission
400 Royal Street
New Orleans, Louisiana 70130

Dear Mr. Glasgow:

Thank you for your letter dated 2 May 1967 relative to the Lake Pontchartrain Barrier Plan, and the Seabrook Lock features of the "Lake Pontchartrain, La. and Vicinity," project.

Our current data collection program includes extensive coverage of Lake Pontchartrain salinities. Upon completion of the lock, we shall expand this coverage, if necessary, to permit an adequate evaluation of the effects of lock operation on the salinity regimen, and a determination as to the extent that the lock operation is producing the salinity regimen indicated by the model data previously furnished you. We shall, of course, consult with your agency in making the above determination, and in the subsequent development of modifications as may be found necessary.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

Copy furnished:
Louisiana Wild Life & Fisheries Commission
River Basin Section
Baton Rouge, Louisiana 70804

LMNED-PP 1 February 1967

Mr. L. J. Cucullu, Vice-President
and Chief Engineer
New Orleans Public Service, Inc.
P. O. Box 60340
New Orleans, Louisiana 70160

Dear Mr. Cucullu:

Please refer to our letter dated 4 November 1966 relative to the Seabrook Lock feature of the "Lake Pontchartrain, La. and Vicinity," project which forwarded a tentative operating procedure for the lock. Please refer also to your reply to the above letter dated 21 November 1966.

Based on additional discussions with biologists of the U. S. Fish and Wildlife Service and the Louisiana Wild Life and Fisheries Commission, we have concluded that operation of the lock at full discharge capacity throughout the full tidal cycle can be tolerated. Such operation would result in trebling the flows for the flooding portion of the tidal cycle as compared with those under the tentative operating procedure described in our letter of 4 November. (See inclosure to above letter, file No. H-2-24053.) Flows on the ebbing portion of the cycle would remain the same as shown on the drawing.

As noted, the data on file No. H-2-24053 neglect interruptions due to lockages. Present and prospective traffic will be analyzed in connection with the detailed design studies for the lock, and the authorized lock structure modified, if required, to insure the flow regimen described.

We are hopeful that you will find the above proposed flow regimen acceptable and await your early reply.

Sincerely yours,

THOMAS J. BOWEN
Colonel, CE
District Engineer

Copy furnished:
Ch, Hydraulics Br.
U.S. Fish & Wildlife Service, Atlanta, Ga.
La. Wild Life & Fish. Comm, N.O.,La.

NEW ORLEANS PUBLIC SERVICE INC.

POST OFFICE BOX 60340

NEW ORLEANS, LOUISIANA 70160

L. J. CUCULLU
VICE PRESIDENT & CHIEF ENGINEER

AREA CODE 504 529-4545
317 BARONNE STREET

February 10, 1967

Colonel Thomas J. Bowen, C. E.
District Engineer, New Orleans District
Corps of Engineers
Department of the Army
P. O. Box 60267
New Orleans, Louisiana 70160

PROPOSED CONTROL BARRIER AT
LAKE PONTCHARTRAIN AND SEABROOK
NEW ORLEANS, LOUISIANA

Dear Colonel Bowen:

Please refer to your letter of February 1, 1967 and to our previous correspondence concerning the effect at our generating stations of the proposed control barrier at Seabrook and Lake Pontchartrain.

Concerning your proposed operating procedure, we are in agreement that the lock should be operated at full discharge capacity on flood tide as well as ebb tide. The careful study by your engineers, which resulted in the conclusion that such operation is acceptable, is greatly appreciated.

It is apparent from the drawing, file No. H-2-24053, which you previously sent us, that the lock, when so operated, will reduce flow quantities from their present magnitudes to approximately those existing at Seabrook before the Mississippi River-Gulf Outlet was opened. It appears that inlet water temperatures to the generating stations will be increased over those now existing but we anticipate that the resultant temperatures probably will permit operation of the existing units within design limitations.

The State of Louisiana Stream Control Commission is presently developing criteria for thermal pollution in compliance with the U.S. Water Quality Act of 1965. We have confidence that present stream temperatures in the vicinity of our stations will comply with criteria to be adopted but also are concerned with the possibility that restriction of flow in the Industrial Canal and Mississippi River-Gulf Outlet could jeopardize our conformance to such criteria.

We do not know if the design of the lock is final or if any further increase in flow quantities can be tolerated. However, to the extent that such proposals can be considered at this stage, we recommend that the lock be of sufficient size to permit as large a volume of unobstructed flow of water as possible within the framework of other requirements which it must meet.

We appreciate the cooperation given to us by you and your engineers in the study of the installation of the proposed lock at Seabrook and, if desired, will be pleased to meet for further discussions at their convenience.

Very truly yours,

L. J. Cucullu

L. J. Cucullu

LJC.s
cc - Messrs. M. C. Abrahm
M. J. Cade
J. F. Vogt

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL

APPENDIX C
INVESTIGATIONS OF ALTERNATIVE DESIGNS AND LAYOUTS

TABLE OF CONTENTS

TABLES

TABLE OF CONTENTS CONT'D

TABLES

PLATES

APPENDIX C
INVESTIGATIONS OF ALTERNATIVE DESIGNS AND LAYOUTS

C1. Alternative types of lock chamber walls. Four types of lock chamber walls have been selected for preliminary designs and estimates. Each is considered to be a technically feasible construction alternative that can be constructed in the wet (a necessary capability because of the considerably higher cost that a cofferdam would entail) and can be designed to withstand forces resulting from the loading cases to be used. The following criteria were applied as basic requirements:

a. Each type of wall must include a cutoff extending to elevation -50.8. This requirement, stipulated by Waterways Experiment Station and based on the foundation exploration by that office, will guarantee a continuous cutoff extending down to the clayey and relatively impervious layer of Nearshore Gulf Deposits.

b. All steel piling that is to become a permanent part of the wall will be sand blasted to near-white-metal, primed with a zinc-rich primer to provide a degree of autogenous cathodic protection supplemental to the impressed system, coated with a 20-mil coal tar epoxy system, and protected in the zone from mud line to water line with an impressed current system of cathodic protection. This extensive amount of corrosion protection is considered necessary because of the corrosive environment and the fact that repair of paint coatings cannot be accomplished except in areas above the water line.

C2. The four types of wall considered are:

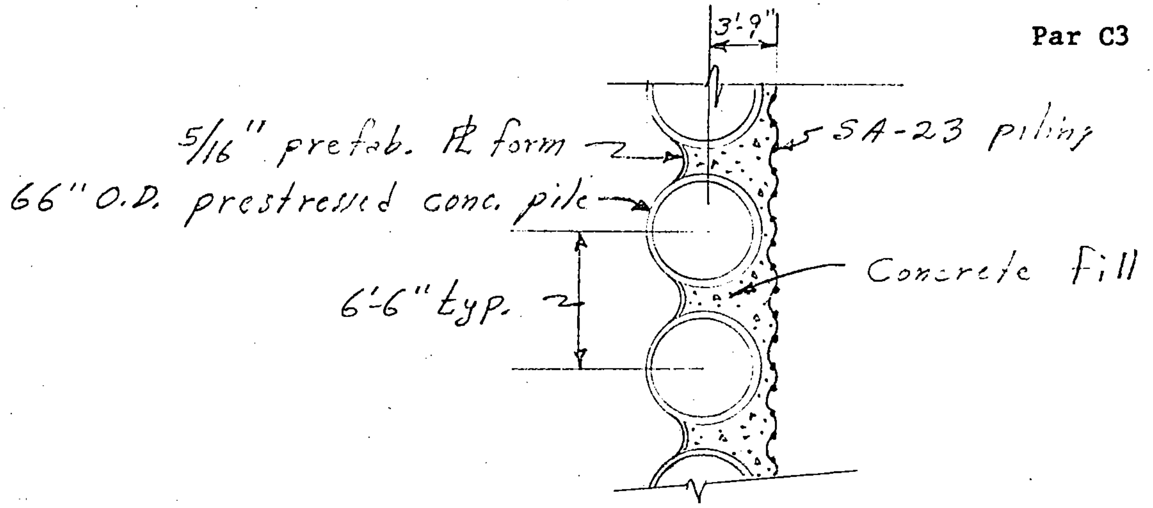
a. Type A - A cantilever type wall comprised of a single row of closely spaced prestressed concrete piles very similar to the plan used in the survey report. Because the design head differential now being used is much higher than was assumed in the survey report, it has been found necessary to place a considerable quantity of fill behind the wall to partially offset the maximum hydrostatic force and reduce the bending to a value compatible with the prestressed pile's bending resistance. The pile section was checked for bending and the required penetration was determined by conventional methods. The relatively deep penetration is needed because of low strengths in the soil layers encountered.

b. Type B - A cantilever type wall similar to the one described above but with a reinforced concrete superstructure replacing the top portion of pile.

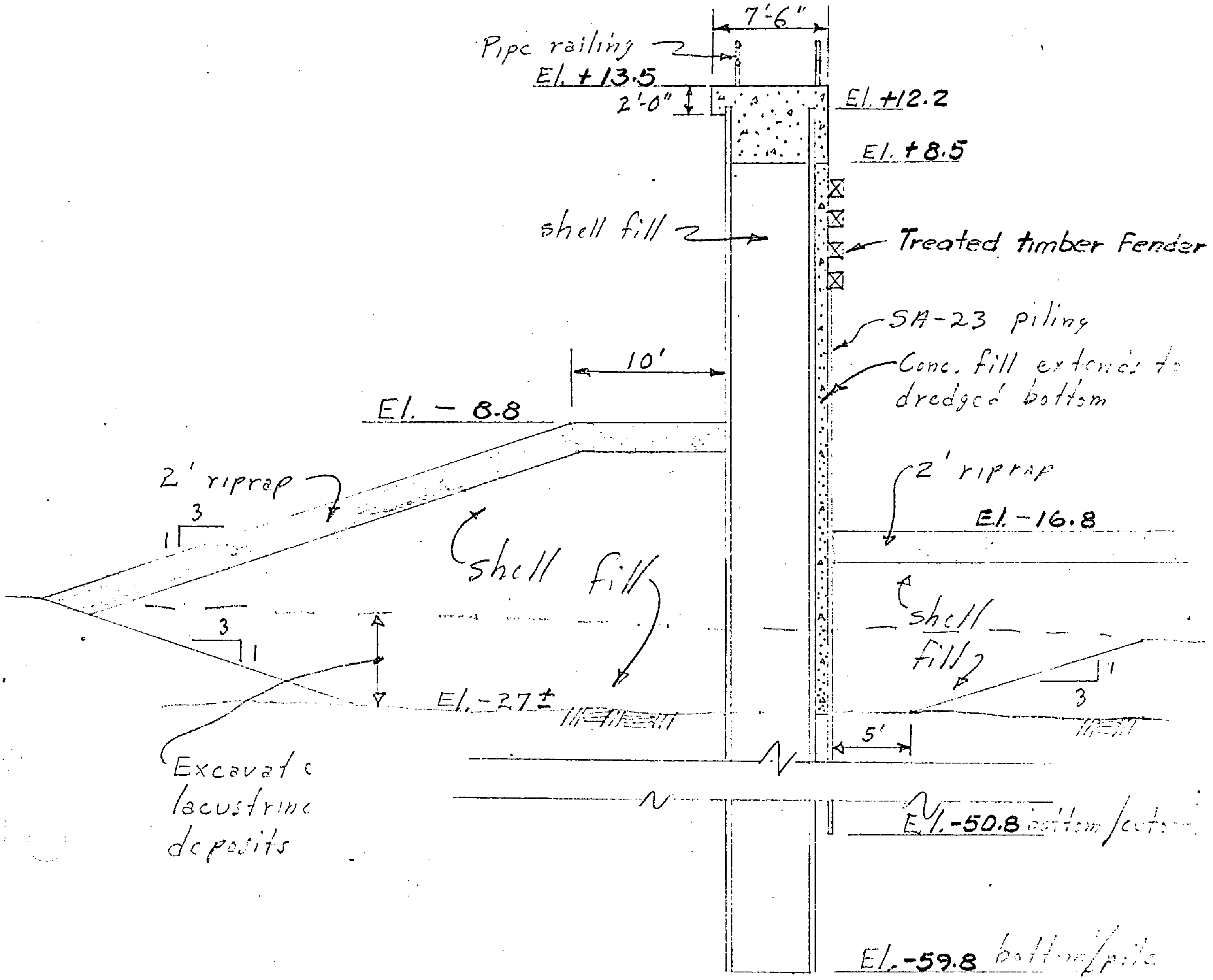
c. Type C - A steel sheet pile cellular wall analyzed in accordance with Cummings' method of calculating tilting resistance. This is a relatively simple gravity type wall but requires a great amount of piling.

d. Type D - A steel sheet pile parallel wall structure, with a tie rod and wale system. This wall, also analyzed by Cummings' method, requires less piling but more fill and considerable miscellaneous metal in the way of tie rods, bolts and wales.

C3. Sketches of the four types of wall considered, and the comparative estimate of cost, are shown on the following pages. Because there is no reliable way of estimating damage frequency (the prestressed pile types would be much more susceptible to boat impact damage) a meaningful estimate of annual cost for each type of wall cannot be made and the estimated first costs are considered to be reliable indicators of relative economy. It may be noted, in this regard, that the annual maintenance and operations cost for the cathodic protection required for steel in the chamber walls and gate structures as proposed in this design memorandum is estimated as less than $1,000 per year. The two sheet pile walls (Types C and D) are considered to be structurally and functionally superior to the cantilever types. Based upon the preliminary estimates of cost, the Types A and D walls would cost about the same. However, because the Type A wall is not considered structurally equal and in the final analyses might require a separate barrier structure to protect from vessel impact on the lakeward side, the Type D wall is selected as the most appropriate for the conditions anticipated on this project

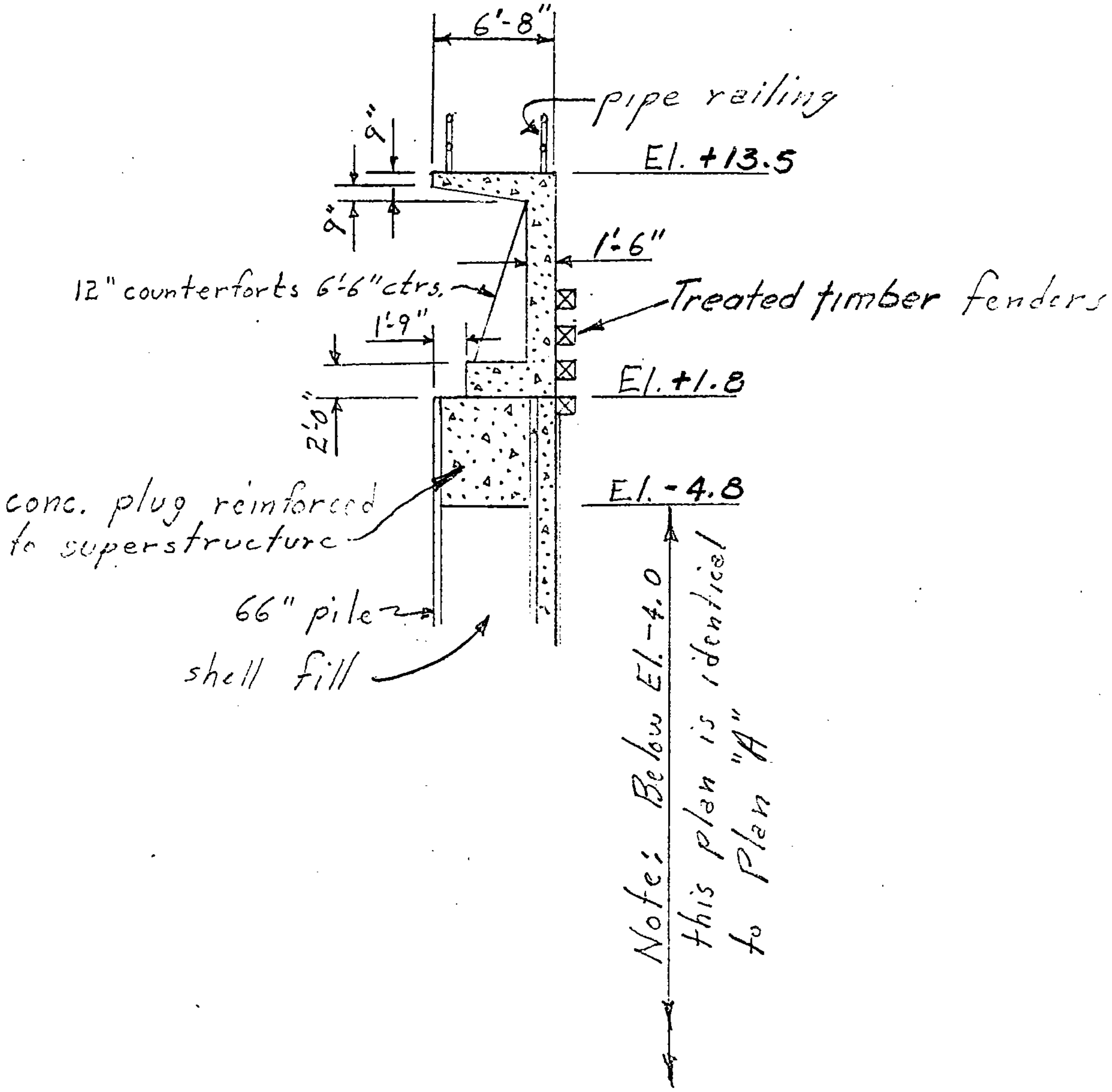


TYP. HORIZ. SECTION



PRESTRESSED CONCRETE PILE CANTILEVER WALL

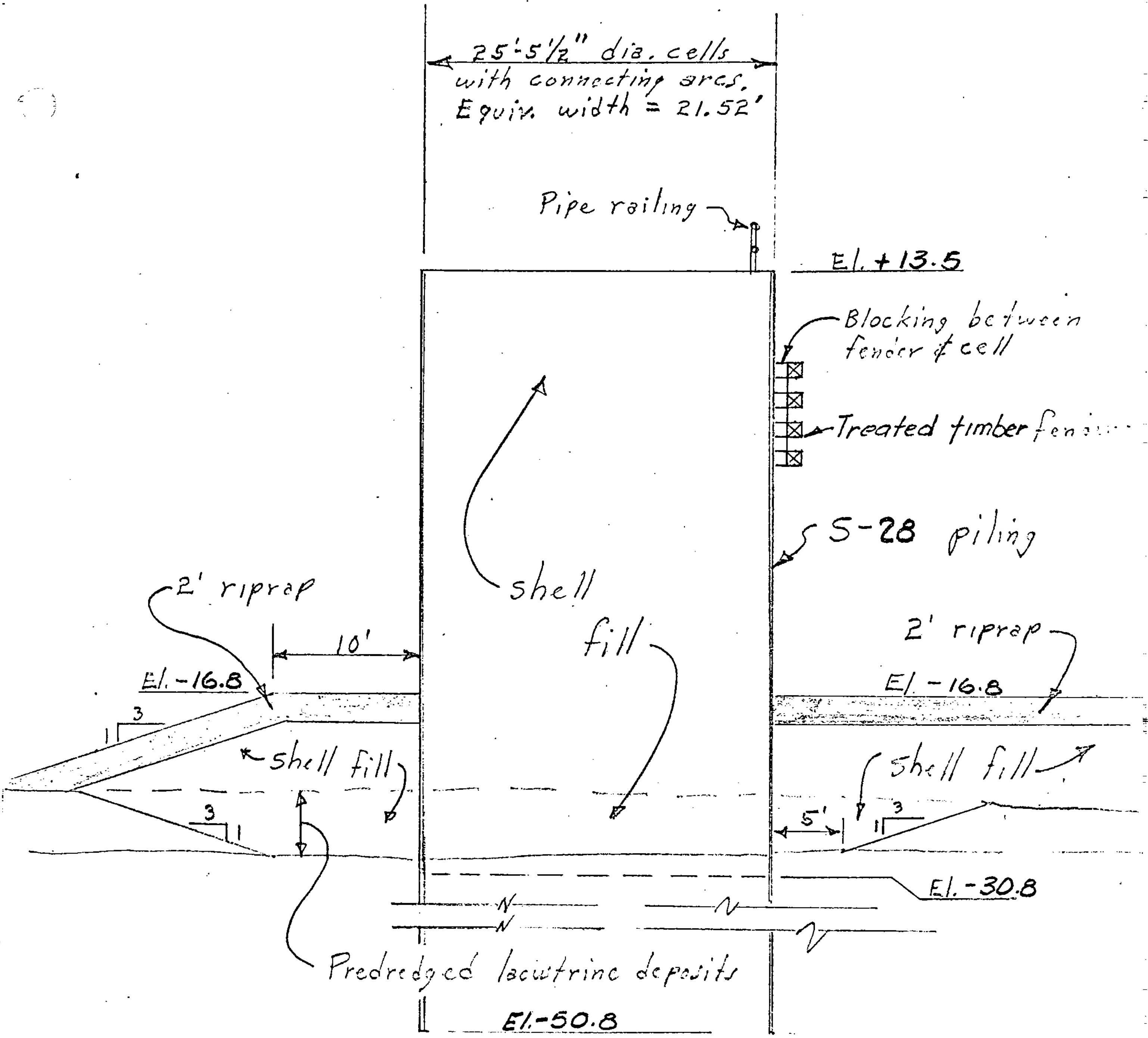
TYPE "A"



PRESTRESSED CONCRETE PILE CANTILEVER WALL

WITH POURED CONCRETE SUPERSTRUCTURE

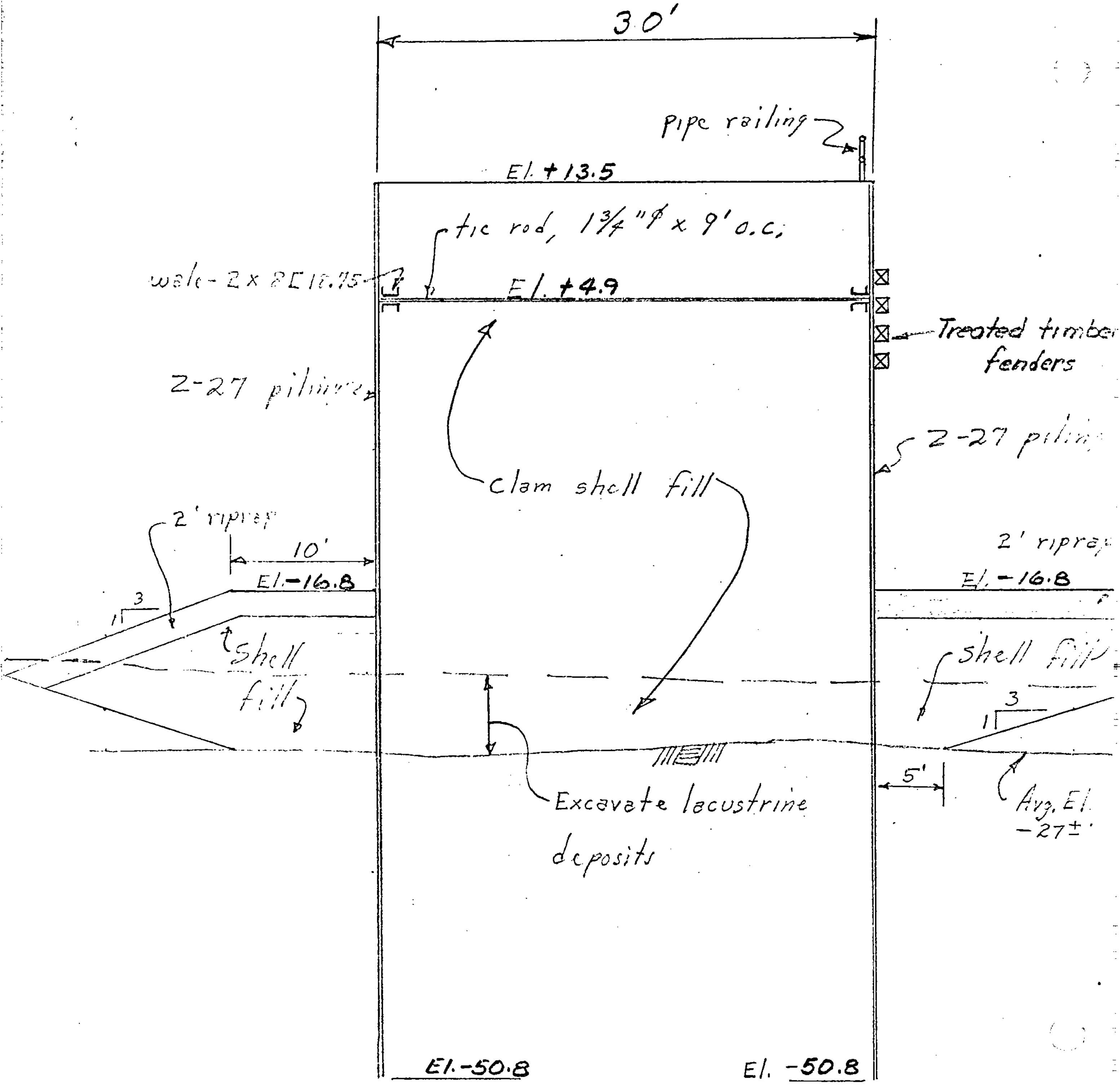
TYPE "B"



TYPICAL SECTION THRU CELLULAR LOCK WALL

TYPE "C"

Note: Required width of wall is greater than in Plan "C" because there is no interlock friction to resist tilting (Analysis is by Cummings method)



TYPICAL SECTION THRU PARALLEL TIE-BACK WALL

TYPE "D"

TABLE C1
PRELIMINARY ESTIMATES OF COST
ALTERNATIVE TYPES OF LOCK CHAMBER WALLS

Item No.	Description	Quantity	Unit	Unit cost	Amount
TYPE A - Cantilever concrete piles				$	$
1	Excavation	14,100	c.y.	4.00	56,400
2	Concrete piles	17,280	l.f.	53.00	915,840
3	Steel sheet piling, SA-23	79,877	s.f.	5.00	399,385
4	Concrete, elev. -27 to +1	3,320	c.y.	65.00	215,800
5	Concrete, elev. +1 to +12.3	1,730	c.y.	65.00	112,450
6	Concrete cap	865	c.y.	60.00	51,900
7	Shell fill	34,325	c.y.	3.90	133,870
8	Riprap	8,000	ton	10.50	84,000
9	Timber fender	91	Mfbm	840.00	76,440
10	Pipe railing	3,100	l.f.	12.00	37,200
	Total				2,083,285
TYPE B - Cantilever concrete piles/poured concrete superstructure					
1	Excavation	14,100	c.y.	4.00	56,400
2	Concrete piles	14,640	l.f.	53.00	775,920
3	Steel sheet piling, SA-23	79,877	s.f.	5.00	399,385
4	Concrete, elev. -27 to +2	4,220	c.y.	60.00	253,200
5	Structural concrete	1,875	c.y.	150.00	281,250
6	Steel reinforcement	188,600	lb.	0.18	33,950
7	Shell fill	32,302	c.y.	3.90	125,980
8	Riprap	8,000	ton	10.50	84,000
9	Timber fender	91	Mfbm	840.00	76,440
10	Pipe railing	3,100	l.f.	12.00	37,200
	Total				2,123,725
TYPE C - Cellular steel sheet piling					
1	Excavation	17,500	c.y.	4.00	70,000
2	Steel sheet piling, S-28	322,749	s.f.	5.75	1,855,810
3	Tee piling	25,206	l.f.	11.00	277,270
4	Shell fill	67,886	c.y.	3.90	264,755
5	Riprap	4,200	ton	10.50	44,100
6	Timber fender	91	Mfbm	840.00	76,440
7	Pipe railing	1,550	l.f.	12.00	18,600
8	Paint piling		l.s.		340,000
9	Cathodic system		l.s.		25,000
	Total				2,971,975

TABLE C1 (Cont.)

Item No.	Description	Quantity	Unit	Unit cost	Amount
				$	$
TYPE D - Parallel, tie-back, steel sheet piling					
1	Excavation	19,500	c.y.	4.00	78,000
2	Steel sheet piling, Z-27	214,761	s.f.	5.35	1,148,970
3	Tie rods, bolts, and wales	230,000	lb.	0.40	92,000
4	Shell fill	84,345	c.y.	3.90	328,945
5	Riprap	4,200	ton	10.50	44,100
6	Timber fender	91	Mfbm	840.00	76,440
7	Pipe railing	1,550	l.f.	12.00	18,600
8	Paint piling		l.s.		260,000
9	Cathodic system		l.s.		20,000
	Total				2,067,055

C4. Alternative types of foundation piles. Closely underlying the Seabrook Lock gate bay sites are two clayey layers (Nearshore Gulf Deposits and zone 1 of the Pleistocene Prairie Formation). Application of significant permanent loading to these strata is very likely to cause soil consolidation and result in gate bay settlement. Consequently it is considered advisable to drive bearing piles completely through these strata and transfer all pile loads to zone 2 of the Pleistocene Prairie Formation (a fairly strong, sandy stratum). Following this procedure will mean that every pile will have a majority of its length acting as a column but not transferring load to the surrounding soil. Only the relatively short length embedded in zone 2 will be considered as transferring load to the soil. It appears possible, therefore, that by using a few very high capacity piles the proportion of piling actively transferring load to the soil will increase, the proportion of piling simply acting as a column will decrease, and the total pile cost may decrease also. Because of this possibility it was decided to make comparative estimates for several different types of piles.

C5. The types of piles studied and their characteristics are as follows:

a. Timber pile (assumed treated). Capacity per pile taken as 20 tons. Nine inch tip diameter. Required penetration in sand is 19' per pile. Length of each pile is 50'. Number of piles required is 1,284.

b. Precast concrete pile assumed to be 14½ inches square. Allowable stress is 900 p.s.i. which gives a pile capacity of 94.6 tons. Required penetration in sand is 44' and length of pile is 74'. Number required is 280.

c. Precast prestressed hollow pipe piles assumed to be 54 inches O.D. with 4 inch walls. Allowable stress is 1,350 p.s.i. and pile capacity is 212 tons. Required penetration in sand is 34' and length of pile is 64'. Number required is 122.

d. Steel H-piles assumed to be 14BP-73. Capacity per pile taken as 86 tons. Required penetration in sand is 40' and length of pile is 70'. Number required is 300.

C6. Regardless of pile type used, actual length needed will be governed by load tests at time of construction. Required penetrations cited above are based on 900 lbs. per sq. ft. assumed skin friction. The number and lengths cited are expected to provide equivalent foundation support and, thus, are directly comparable. Related estimates of costs are given in Table C2.

TABLE C2
PRELIMINARY ESTIMATES OF COST
ALTERNATIVE TYPES OF FOUNDATION PILES

Description	Quantity	Unit	Unit cost $	Amount $
Timber piles, treated 1284 @ 50'	64,200	1.f.	3.50	224,700
Precast concrete piles 280 @ 74'	20,720	1.f.	9.95	206,164
Prestressed concrete piles 122 @ 64'	7,808	1.f.	35.00	273,280
Steel H-piles (14 BP-73) 300 @ 70'	21,000	1.f.	9.50	199,500

C7. It is concluded that steel H-piles are the least costly of the four types studied. Because of permanent embeddment and lack of oxygen in the underlying strata or shell fill, it is believed that corrosion would not be a problem. Additionally, the steel piles would undoubtedly be the easiest type to drive through layers of shell fill and into the dense underlying sands. For these reasons the steel piles will be used.

C8. Alternative types of Dolphins. To protect the lakeward gate bay from damage by loose or uncontrolled vessels, it was found necessary to place dolphins around the gate bay. Comparative estimates were made for two types of dolphins, both spaced so as to prevent a larger vessel from reaching the gate bay. The types of dolphins studied were:

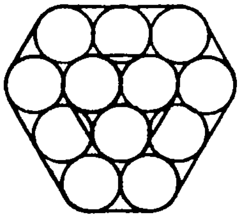
a. Timber Pile Cluster - The pile cluster consists of 12 treated timber piles, approximately 52 feet long; 3 vertical and 9 battered, wrapped with 1 inch diameter galvanized cable. Number of pile clusters required is 12. See sketch of timber pile cluster dolphin following estimate.

b. Sheet Pile Cell - The cells are 33.42 feet in diameter, made up of 84 sheets of S-28 piling and filled with clam shells. The piling is approximately 47 feet long. Number of cells required is 6. See sketch of sheet pile cell dolphin following estimate.

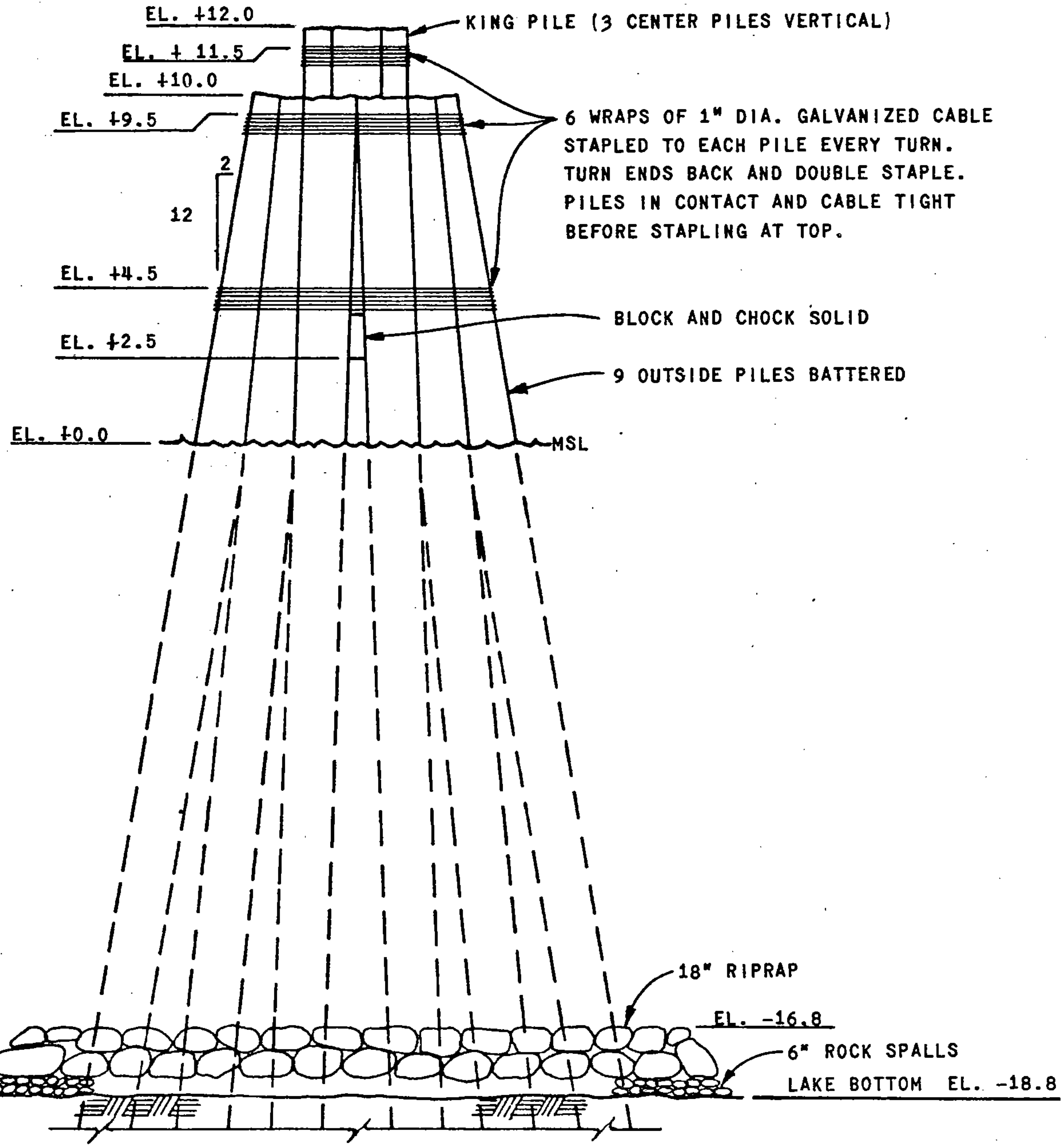
C9. Relative Estimates of Cost are:

Type of Dolphin	Number	Unit Cost	Amount
Timber Pile Cluster	12	$ 3,000.00	$ 36,000.00
Sheet Pile Cell	6	$20,700.00	$124,200.00

Based upon the preliminary cost estimate the timber pile clusters are the least expensive and therefore will be used.



PLAN

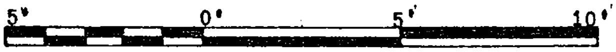


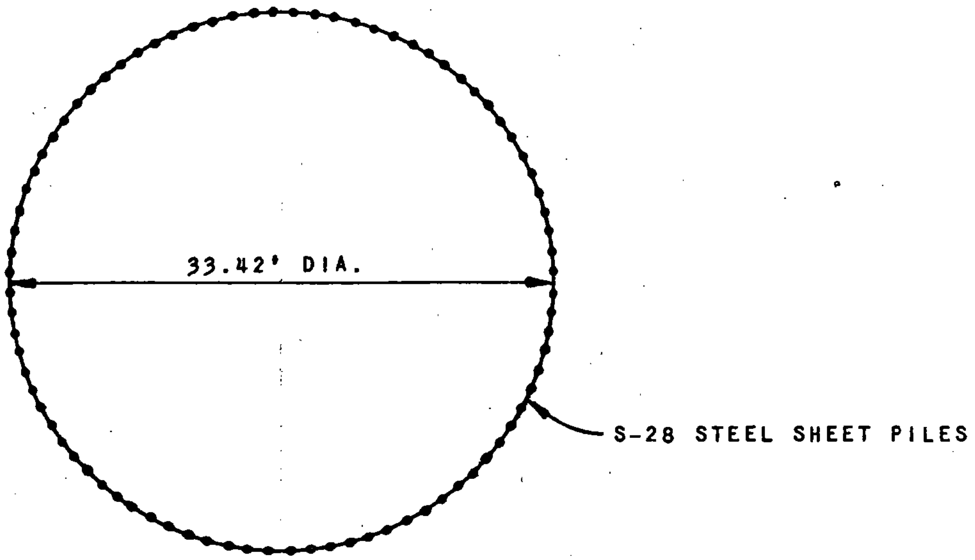
NOTE: BOTTOMS OF ALL PILES AT EL. -40.0

ELEVATION

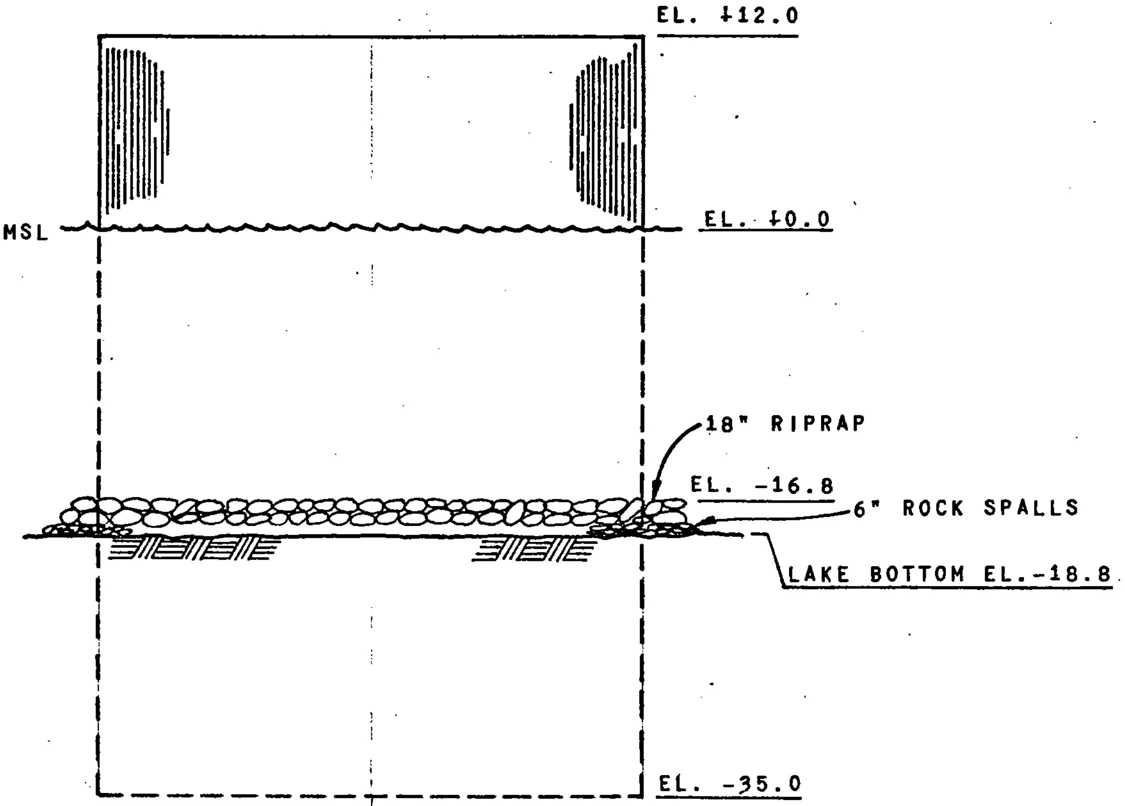
TIMBER PILE CLUSTER DOLPHIN

SCALE 1" = 5'





PLAN



ELEVATION

SHEET PILE CELL DOLPHIN

SCALE 1" = 12'

12' 0 12' 24'

C10. Alternative layouts of the lock complex. Four basic layouts were considered for the lock complex (lock, outlet structure or supplemental flow structure and rock and shell dam). All four of these alternative plans were considered capable of comparable functional operation as described in paragraph 19 of the main text of this report.

C11. The four alternative layouts are as follows:

a. Plan I - Plan I, is the recommended layout as described previously in this report and shown on plates 2 and 3.

b. Plan II - The Plan II lock complex layout is shown on plate C1 of this Appendix. This layout incorporates the identical outlet structure (except for changes necessary to accommodate the vehicular access bridges) proposed for the recommended plan but located on the west side of lock. The lock and connecting dam for Plan II are located approximately 200 feet more landward than for the recommended plan.

c. Plan III - The Plan III lock complex layout is shown on plate C2 of this Appendix. This layout incorporates two outlet structures with the lock located between them. Again, the westerly outlet structure will incorporate a vehicular access bridge. The combined capacity of the two outlet structures will be equivalent to the single outlet structure used for Plans I and II. The location of the lock in Plan III is identical to that for Plan II.

d. Plan IV - The Plan IV lock complex layout is shown on plate C3 of this Appendix. The Plan IV complex differs from Plans I, II and III both in components and actual physical operation. The original concept for an outlet structure and the one on which contacts with local interests were based, was for a relatively small facility that could supplement the outlet capacity inherent in an open lock. The Plan IV complex basically reflects this concept, substituting a supplemental flow structure for the outlet works used in Plan I, II and III and providing a more structurally sophisticated lock. The supplemental flow structure was designed to operate in normally an open position to provide the normal interchange between Lake Ponchartrain and the IHNC. During a hurricane, this structure would supplement the capacity of an open lock and would help to reduce IHNC flood stages. The combined stage relief capacities of the supplemental flow structure and the open lock would be the same as the capacity of the outlet structures for Plans I, II and III. Both the lock and supplemental flow structure for Plan IV would require stilling basins and other protective features needed to insure their safety during hurricane generated discharges. The lock for Plan IV is located in the same location as for Plans II and III with the supplemental flow structure sited immediately adjacent to the lock landward gate bay on the east side.

C12. The preliminary cost estimate for the recommended lock complex layout (Plan I) is given in paragraph 67 of the main text of this report. Preliminary comparative cost estimates for Plans II, III and IV are shown in Tables C3, C4 and C5 respectively. Cost considerations that may not be readily apparent from the drawings, are nevertheless pertinent to any economic comparison of the four plans and are discussed as follows:

a. Cofferdams for construction of the lock gate bays and outlet or supplemental flow structures constitute one of the principal items of cost. Preliminary cellular cofferdam layouts have been made for each of the alternative plans. Following are the total number of individual cofferdam cells required for construction of the lakeward gate bay and the landward gate bay and outlet or supplemental flow structures:

	COFFERDAM CONSTRUCTION (NUMBER OF CELLS REQUIRED)	
PLAN	LAKEWARD GATE BAY	LANDWARD GATE BAY AND OUTLET OR SUPPLEMENTAL FLOW STRUCTURES
I	14	25
II	14	35
III	14	35
IV	21	19

NOTES:

1. All cells were assumed to be 58.89 feet in diameter with bottom elevation at -60.0 and top elevation at +8.0.
2. Cofferdams for construction of the Plans II and III landward gate bays and outlet structures would be in two stages.

b. Outlet structures located west of the lock (Plans II and III) interrupt top of dam access to the lock. Consequently these structures must provide vehicular bridges not required for outlet structures located east of the lock.

c. Excavation, required for both Plans II and III, is significantly greater than for Plans I and IV. This is because the westerly outlet structure location is in an area where existing lake bottom is relatively high and is underlain by relatively deep Lacustrine deposits.

d. The recommended layout (Plan I) will require approximately 400 feet of additional approach wall construction to tie into existing walls at the bridge. This additional wall was required due to the lakeward location of the lock in Plan I as compared to the other three alternative layouts. The Plan I lakeward location was required to allow construction of the outlet structure without undermining existing easterly shoreline facilities.

e. During hurricane generated discharges, the lock for Plan IV will pass water at very high velocities. To protect against the erosive effect of these high velocities, it has been determined that a complete lock floor consisting of cast-in-place concrete blocks will be required. The construction of this lock floor will have to be accomplished in the dry resulting in a chamber wall (wall between gate bays) that must act as a cofferdam during initial construction and for possible future maintenance unwatering. This requirement resulted in a lock chamber wall 39 feet in width compared to the 30 foot wide walls required for the other three alternative plans, with additional internal bracing between walls.

f. For Plan IV, the costs of unwatering during construction are significantly greater than for the other three plans. This is due to the following:

(1) Plan IV requires complete unwatering of the lock chamber for construction of the cast-in-place concrete floor blocks. This additional work area, to be unwatered, is not required by the other three plans.

(2) The lock chamber walls serve as the cofferdam during chamber unwatering. Well points would have to be installed within the lock wall shell fill to reduce the saturation line to allowable limits for design of the steel sheet piling.

(3) Special treatment will be required along the easterly shore facilities which form the landward closure for the cofferdam to construct the landward gate bay and supplemental flow structure. This will require installation and pumping of deep wells to reduce the saturation line behind the existing facilities to tolerable levels.

TABLE C3
ESTIMATE OF FIRST COSTS FOR CONSTRUCTION OF ALTERNATIVE LOCK LAYOUT - PLAN II

Line Item	Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
					$	$
01		LANDS AND DAMAGES				
	1.	Land Cost				
		(a) Unpaved	11,000	S.F.	1.25	13,750
		(b) Paved	1,900	S.F.	2.25	4,275
05		LOCK				
	2.	Excavation-Lacustrine Deposits	225,200	C.Y.	1.25	281,500
	3.	Shell Fill	246,200	C.Y.	3.45	849,390
	4.	Cofferdams				
		(a) Steel Sheet Piling S-28 (24 Mo. Rental)	5,107	TONS	176.00	898,832
		(b) Set, Drive & Pull	744,300	S.F.	3.10	2,307,330
		(c) Shell Cell Fill	155,800	C.Y.	4.20	654,360
	5.	Dewatering				500,000
	6.	Excavations - Structure	387,000	L.S.	2.10	81,270
	7.	Riprap Stone - In Wet	46,900	TON	10.50	492,450
	8.	Derrick Stone				
		(a) Stone in Dry	11,300	TON	13.65	154,245
		(b) Stone in Wet	22,300	TON	10.50	234,150
	9.	Rock Spalls	21,400	TON	8.95	191,530
	10.	Lock Chamber Walls				
		(a) Plain Steel Sheet Piling Z-27	212,200	S.F.	5.35	1,135,270
		(b) Fabricated Piling	2,100	S.F.	7.95	16,695
		(c) Metal Work	264,000	LB.	0.37	97,680
		(d) Shell Fill	55,500	C.Y.	3.90	216,450
		(e) Timber Fenders	12,360	L.F.	10.00	123,600
	11.	Guide and Guard Walls				
		(a) Treated Timber "Marine Piling"	35,000	L.F.	4.00	140,000
		(b) Treated Timber "Framing & Planking"	58	MFBM	840.00	48,720
		(c) Timber Fenders	4,000	L.F.	10.00	40,000
	12.	Mooring Dolphins				
		(a) Plain Steel Sheet Piling S-28	10,100	S.F.	5.50	55,550
		(b) Shell Cell Fill	2,100	C.Y.	3.90	8,190
		(c) Timber Pile Clusters "Marine Piling"	12	EA.	3,000.00	36,000

TABLE C3 (Cont.)

Line Item: Item No.	Description	Estimated Quantity	Unit	Unit Cost $	Amount $
13.	Steel Bearing Piles				
	(a) 14BP73	28,920	L.F.	9.50	274,740
	(b) 12BP53	1,800	L.F.	7.45	13,410
14.	Pile Loading Test	4	EA.	3,045.00	12,180
15.	Concrete				
	(a) Gate Bays				
	(1) Slab	5,180	C.Y.	45.00	233,100
	(2) Walls	3,600	C.Y.	75.00	270,000
	(b) Outlet Structure & Stilling Basin				
	(1) Slab	3,460	C.Y.	45.00	155,700
	(2) Walls	2,400	C.Y.	75.00	180,000
	(c) Retaining Walls	140	C.Y.	75.00	10,500
16.	Portland Cement	20,325	BBL.	5.50	111,788
17.	Steel Reinforcement	2,210,000	LB.	0.16	353,600
18.	Cut Off Walls-Steel Sheet Piling MA-22	56,000	S.F.	4.75	266,000
19.	Sector Gates 788,000 Lbs. - Steel (Including Painting & Embedded Metals)		L.S.		604,000
20.	Vertical Lift Gates (123,000 Lbs. Gate (Including Painting & Embedded Metals)		L.S.		93,000
21.	Needle Girders	64,000	LB.	0.45	28,800
22.	Miscellaneous Metal Work (Including Handrailing)		L.S.		63,000
23.	Cathodic Protection System		L.S.		107,200
24.	Sandblasting & Coaltar-Epoxy Painting Steel Surface	780,320	S.F.		507,208
25.	Control Houses	2	EA.	3,700.00	7,400
26.	Operating Machinery		L.S.		310,000
27.	Engine Generator Set	1	EA.	12,000.00	12,000
28.	Electrical Work		L.S.		282,000
29.	Water Distribution & Fire Protection System		L.S.		32,000
30.	Sewage System		L.S.		22,500
31.	Natural Gas System		L.S.		21,000
32.	Foot Bridges & Access		L.S.		33,900
33.	Operations Building		L.S.		32,000
34.	Paint Buildings		L.S.		1,600
	SUBTOTAL				12,619,863
	Contingencies @ 20%±				2,480,137
	Total Estimated Contractors Earnings Plus Contingencies				15,100,000

TABLE C4
ESTIMATE OF FIRST COSTS FOR CONSTRUCTION OF ALTERNATIVE LOCK LAYOUT - PLAN III

Line Item	Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
					$	$
01		<u>LANDS AND DAMAGES</u>				
	1.	Land Cost				
		(a) Unpaved	11,000	S.F.	1.25	13,750
		(b) Paved	1,900	S.F.	2.25	4,275
05		<u>LOCK</u>				
	2.	Excavation-Lacustrine Deposits	211,200	C.Y.	1.25	264,000
	3.	Shell Fill	231,200	C.Y.	3.45	797,640
	4.	Cofferdams				
		(a) Steel Sheet Piling S-28 (24 Mo. Rental)	4,140	TON	176.00	728,640
		(b) Set, Drive & Pull	744,300	S.F.	3.10	2,307,330
		(c) Shell Cell Fill	155,800	C.Y.	4.00	623,200
	5.	Dewatering		L.S.		490,000
	6.	Excavations - Structure	48,300	C.Y.	2.10	101,430
	7.	Riprap Stone - In Wet	48,200	TON	10.50	506,100
	8.	Derrick Stone				
		(a) Stone in Dry	11,900	TON	13.65	162,435
		(b) Stone in Wet	20,400	TON	10.50	214,200
	9.	Rock Spalls	22,000	TON	8.95	204,955
	10.	Lock Chamber Walls				
		(a) Plain Steel Sheet Piling Z-27	212,200	S.F.	5.35	1,135,270
		(b) Fabricated Piling	2,100	S.F.	7.95	16,695
		(c) Metal Work	264,000	LB.	0.37	97,680
		(d) Shell Fill	55,500	C.Y.	3.90	216,450
		(e) Timber Fenders	12,360	L.F.	10.00	123,600
	11.	Guide & Guard Walls				
		(a) Treated Timber "Marine Piling"	35,000	L.F.	4.00	140,000
		(b) Treated Timber "Framing & Planking"	58	MFBM	840.00	48,720
		(c) Timber Fenders	4,000	L.F.	10.00	40,000
	12.	Mooring Dolphins				
		(a) Plain Steel Sheet Piling S-28	10,100	S.F.	5.50	55,550
		(b) Shell Cell Fill	2,100	C.Y.	3.90	8,190
		(c) Timber Pile Clusters "Marine Piling"	12	EA.	3,000.00	36,000

Par C12 f.

TABLE C4 (Cont.)

Line Item No.	Description	Estimated Quantity	Unit	Unit Cost $	Amount $
13.	Steel Bearing Piles				
	(a) 14BP73	29,400	L.F.	9.50	279,300
	(b) 12BP53	4,500	L.F.	7.45	33,750
14.	Pile Loading Test	4	EA.	3,045.00	12,180
15.	Concrete				
	(a) Gate Bays				
	(1) Slab	5,180	C.Y.	45.00	233,100
	(2) Walls	3,600	C.Y.	75.00	270,000
	(b) Outlet Structure & Stilling Basin				
	(1) Slab	3,990	C.Y.	45.00	179,550
	(2) Walls	3,510	C.Y.	75.00	263,250
	(c) Retaining Walls	210	C.Y.	75.00	15,750
16.	Portland Cement	22,680	BBL.	5.50	124,740
17.	Steel Reinforcement	2,463,500	LB.	0.16	394,160
18.	Cut Off Walls-Steel Sheet Piling MA-22	56,000	S.F.	4.75	266,000
19.	Sector Gates 788,000 Lbs. - Steel (Including Painting & Embedded Metals)		L.S.		604,000
20.	Vertical Lift Gates (128,000 Lbs. Gate Including Embedded Metals & Painting)		L.S.		97,000
21.	Needle Girders	57,000	LB.	0.45	25,650
22.	Miscellaneous Metal Work (Including Handrailing)		L.S.		63,000
23.	Cathodic Protection System		L.S.		107,200
24.	Sandblasting & Coaltar-Epoxy Painting Steel Surface	779,550	S.F.		506,707
25.	Control Houses	2	EA.	3,700.00	7,400
26.	Operating Machinery		L.S.		317,000
27.	Engine Generator Set	1	EA.	12,000.00	12,000
28.	Electrical Work		L.S.		288,000
29.	Water Distribution & Fire Protection System		L.S.		32,000
30.	Sewage System		L.S.		22,500
31.	Natural Gas System		L.S.		21,000
32.	Foot Bridge & Access bridge		L.S.		15,400
33.	Operations Building		L.S.		32,000
34.	Paint Buildings		L.S.		1,600
	SUBTOTAL				12,560,347
	Contingencies @ 20%±				2,439,653
	Total Estimated Contractors Earnings Plus Contingencies				15,000,000

TABLE C5
ESTIMATE OF FIRST COSTS FOR CONSTRUCTION OF ALTERNATIVE LOCK LAYOUT - PLAN IV

Line Item	Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
					$	$
01		<u>LANDS AND DAMAGES</u>				
	1.	Land Cost				
		(a) Unpaved	11,000	S.F.	1.25	13,750
		(b) Paved	1,900	S.F.	2.25	4,275
05		<u>LOCK</u>				
	2.	Excavation-Lacustrine Deposits	203,000	C.Y.	1.25	253,750
	3.	Shell Fill	234,000	C.Y.	3.45	807,300
	4.	Cofferdam				
		(a) Steel Sheet Piling S-28 (24 Mo. Rental)	4,600	TON	176.00	809,600
		(b) Set, Drive & Pull	602,300	S.F.	3.10	1,867,130
		(c) Shell Cell Fill	126,400	C.Y.	4.35	549,840
	5.	Dewatering		L.S.		690,000
	6.	Excavation-structure	27,000	C.Y.	2.10	56,700
	7.	Riprap Stone				
		(a) Stone in Dry		TON	13.65	
		(b) Stone in Wet	36,600	TON	10.50	384,300
	8.	Derrick Stone				
		(a) Stone in Dry	13,800	TON	13.65	188,370
		(b) Stone in Wet	24,300	TON	10.50	255,150
	9.	Rock Spalls	19,700	TON	8.95	176,315
	10.	Lock Chamber Walls				
		(a) Plain Steel Sheet Piling Z-27	216,900	S.F.	5.35	1,160,415
		(b) Fabricated Piling	2,100	S.F.	7.95	16,695
		(c) Metal Work	299,000	LB.	0.37	110,630
		(d) Shell Fill	72,200	C.Y.	3.90	281,580
		(e) Timber Fenders	12,360	L.F.	10.00	123,600
	11.	Guide and Guard Walls				
		(a) Treated Timber "Marine Piling"	35,000	L.F.	4.00	140,000
		(b) Treated Timber Framing & Planking	58	MFBM	840.00	48,720
		(c) Timber Fenders	4,000	L.F.	10.00	40,000
	12.	Barrier Wall				
		(a) Treated Timber "Marine Piling"	9,200	L.F.	4.00	36,800
		(b) Timber Fenders	810	L.F.	10.00	8,100

TABLE C5 (Cont.)

Line Item: Item No.	Description	Estimated Quantity	Unit	Unit Cost	Amount
				$	$
13.	Mooring Dolphins				
	(a) Plain Steel Sheet Piling S-28	10,100	S.F.	5.50	55,550
	(b) Shell Cell Fill	2,100	C.Y.	3.90	8,190
	(c) Timber Pile Clusters	22	EA.	3,000.00	66,000
14.	Steel Bearing Piles 14BP-73	28,080	L.F.	9.50	266,760
15.	Pile Loading Test	4	EA.	3,045.00	12,180
16.	Temporary Struts & Wales		L.S.		163,000
17.	Concrete				
	(a) Gate Bays				
	(1) Slab	5,180	C.Y.	45.00	233,100
	(2) Walls	3,600	C.Y.	75.00	270,000
	(b) Stilling Basin				
	(1) Slab	2,650	C.Y.	45.00	119,250
	(2) Walls	1,850	C.Y.	75.00	138,750
	(c) Retaining Walls	70	C.Y.	75.00	5,250
	(d) Lock Chamber Slab	7,225	C.Y.	45.00	325,125
18.	Portland Cement	28,300	BBL.	5.50	155,650
19.	Steel, Reinforcement	2,272,000	LB.	0.16	363,520
20.	Cut-off Walls Stl. Sht. Piling MA-22	55,500	S.F.	4.75	263,625
21.	Sector Gates (Including Painting)		L.S.		604,000
22.	Bulkheads		L.S.		13,000
23.	Needle Girders	50,000	LB.	0.45	22,500
24.	Miscellaneous Metal Work (Including Handrailing)		L.S.		63,000
25.	Cathodic Protection		L.S.		105,000
26.	Sand Blasting & Coaltar-Epoxy Painting Steel Surfaces	793,000	S.F.	0.65	515,450
27.	Control Houses	2	EA.	3,700.00	7,400
28.	Operating Machinery		L.S.		205,000
29.	Engine Generator Set	1	EA.	12,000.00	12,000
30.	Electrical Work		L.S.		262,000
31.	Water Distribution & Fire Protection System		L.S.		32,000
32.	Sewage System		L.S.		20,000
33.	Natural Gas System		L.S.		21,000
34.	Access Bridges	2	EA.	5,500.00	11,000
35.	Operations Buildings		L.S.		32,000
36.	Paint Buildings		L.S.		1,600
	SUBTOTAL				12,395,920
	Contingencies @ 20%±				2,504,080
	Total Estimated Contractors Earnings Plus Contingencies				14,900,000

C13. Results of alternate layout preliminary cost estimate comparisons - From review of the estimates as reflected by Table 3 of the main text of this report and **Tables C3, C4 and C5 of this Appendix, it is apparent** that first costs of each of the four alternative lock complex layouts will generally be about the same. The Plan IV layout would appear to be the least expensive, followed closely by Plan I (the recommended plan) **and Plan III. Plans I and III although estimated independently resulted in identical first cost totals.**

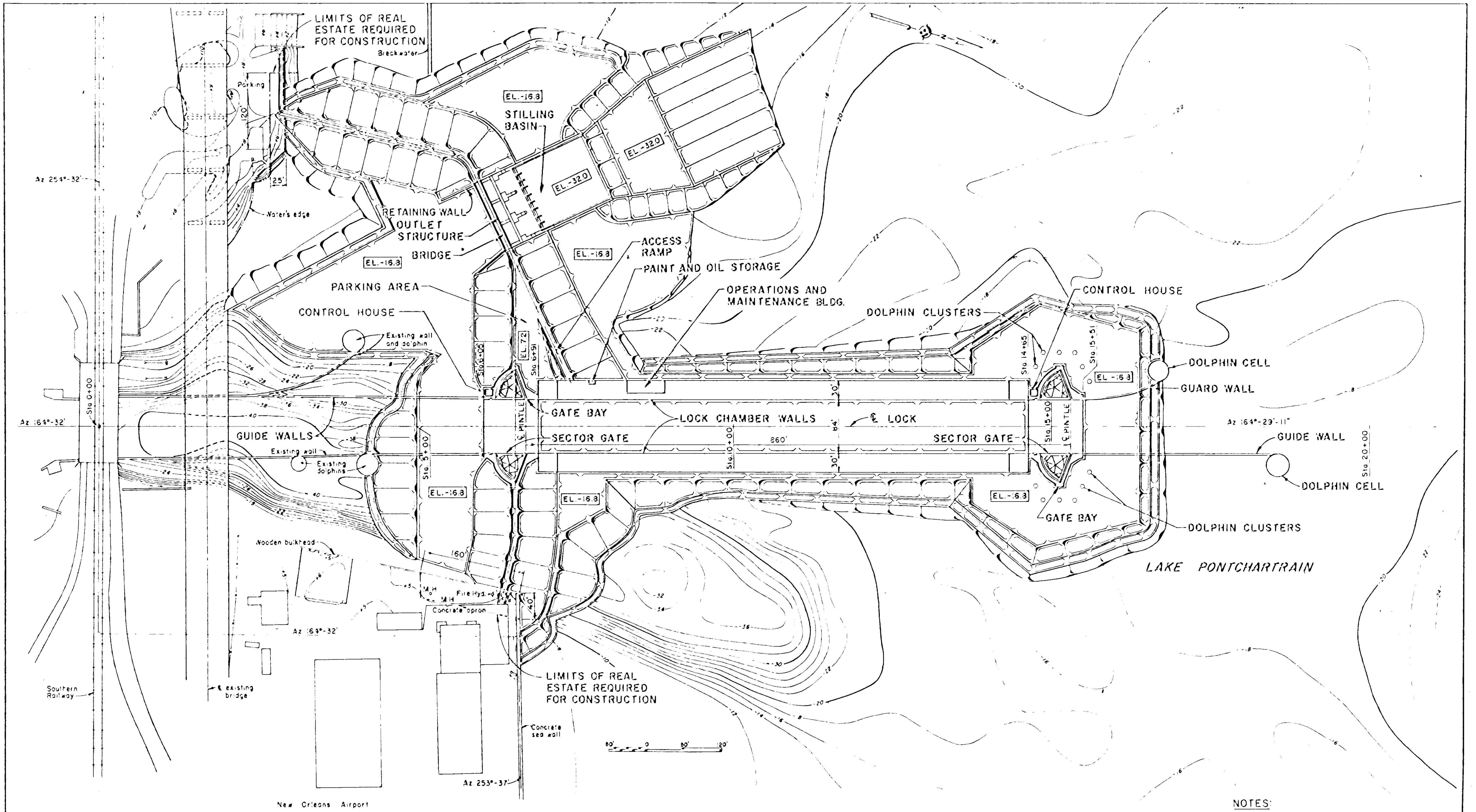
C14. Basis of selection for the recommended lock complex layout - The Plan I layout selection, as the recommended plan, is based on considerations of annual costs, safety and flexibility of operation. Because there would be no apparent differences in annual operating and maintenance costs, amortization of first costs should be the best index of total annual costs. On this basis, the Plan I layout would only be the second least expensive alternative, however; because of the necessarily preliminary nature of these estimates the apparent cost difference is not a compelling factor. It is noted the estimate for Plan I was based upon more detailed design than was available for the other three plans. Consequently, the estimate for the recommended plan does represent a greater degree of precision than that contained in the estimates for the other three alternative plans.

C15. Theoretically all four alternate plans are capable of comparable functions, but practically this is not necessarily true. Based upon flexibility of operation, it is considered that an outlet structure, as provided by Plans I, II and III, will better satisfy the requirements of the Seabrook Lock complex than a supplemental flow structure - open lock as proposed for Plan IV. Specific deficiencies considered inherent in Plan IV and other considerations relative to the recommended plan selection, are as follows:

a. Although lake salinity could be controlled with the Plan IV layout, the precise control that would be inherent in the adjustable gated outlet structures of the other three plans, would be much more difficult to attain.

b. Navigation, for Plan IV, must be discontinued when hurricane generated high stages are being released to the lake in order to insure that the desired stage relief is actually obtained. For Plans I, II and III, however, navigation could continue under emergency conditions.

c. All four alternative plans have supposedly the same degree of protection. However the results of actually passing hurricane generated flows through either the lock and supplemental flow structure for Plan IV or the outlet structure for Plans I, II and III and the effect on future operational capability is unknown. In view of the uncertainties of passing hurricane generated flows through a navigation lock founded in an easily erodible foundation, the relatively small cost savings does not warrant selection of the Plan IV layout.



NOTES:

ELEVATIONS ARE IN FEET, MEAN SEA LEVEL

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL
ALTERNATE PLAN II
SCALES AS SHOWN

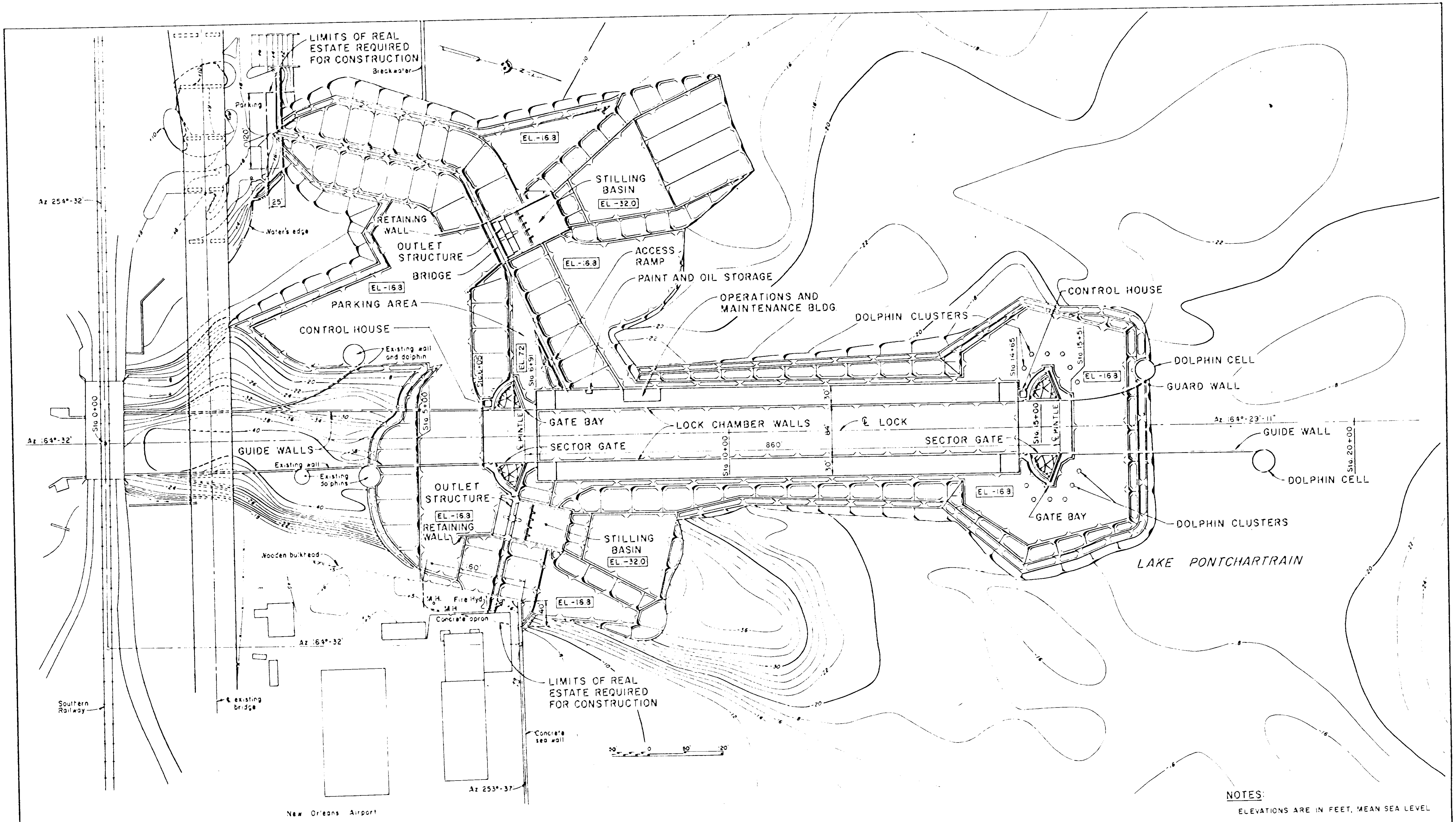
U.S. ARMY ENGINEER DISTRICT, BUFFALO
CORPS OF ENGINEERS, U.S. ARMY
BUFFALO, N.Y.

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

DATE

FILE NO. [illegible]

PLATE C1

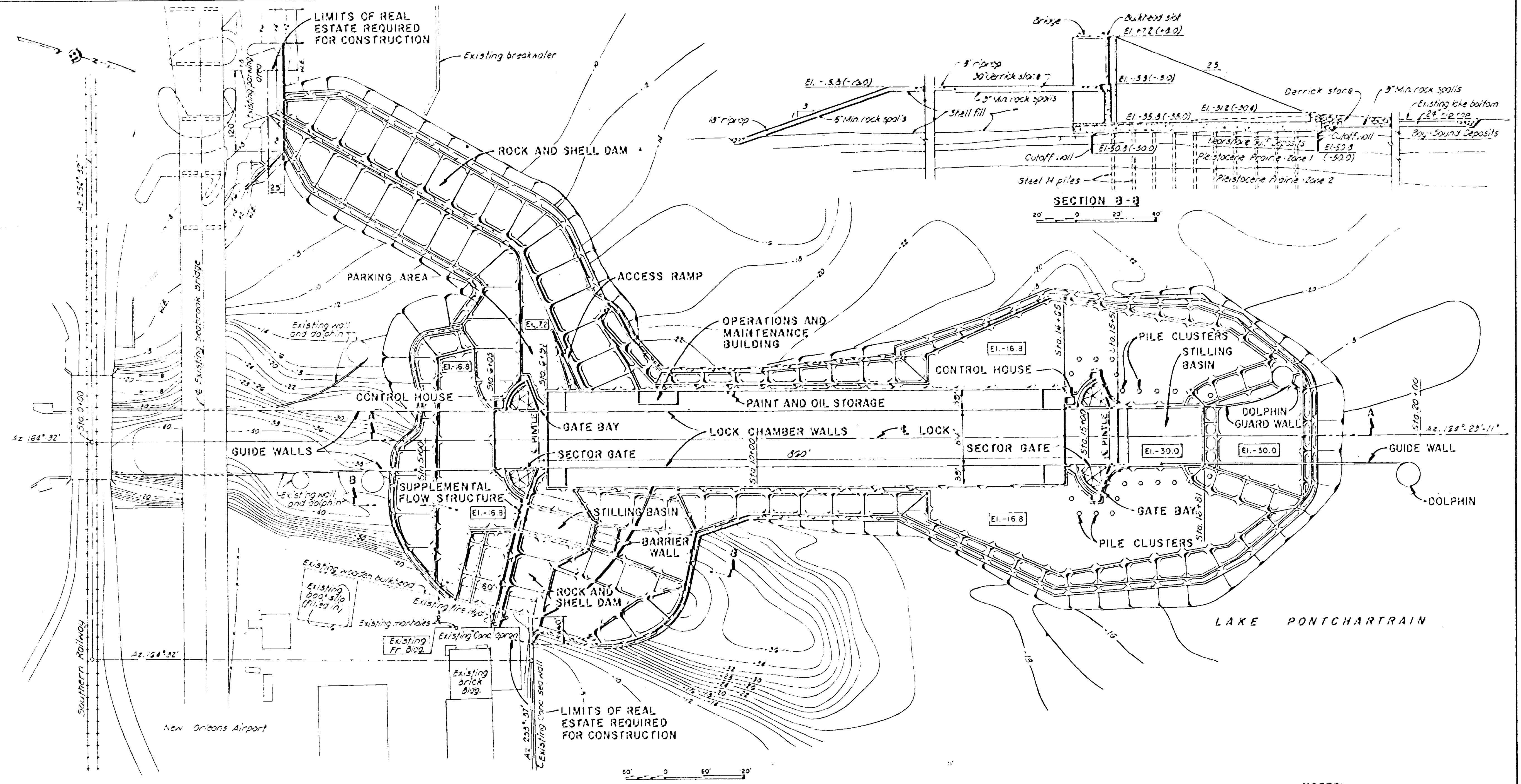


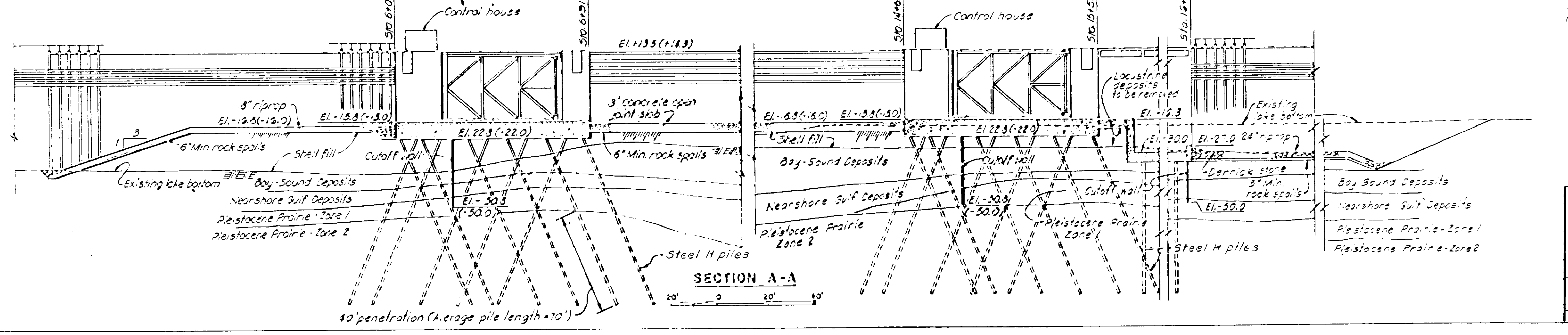
NOTES:

ELEVATIONS ARE IN FEET, MEAN SEA LEVEL

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL
ALTERNATE PLAN III
SCALES AS SHOWN
U.S. ARMY ENGINEER DISTRICT BUFFALO
CORPS OF ENGINEERS, U.S. ARMY
BUFFALO, N.Y.
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS, U.S. ARMY
NEW ORLEANS, LA.
DATE

PLATE C2





SECTION A-A

NOTES:

Elevations are in feet, Mean Sea Level except those in parentheses which are in feet, Mean Low Gulf.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. I, GENERAL

ALTERNATE PLAN IV

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT NEW ORLEANS CORPS OF ENGINEERS U.S. ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23083

PLATE C3

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

APPENDIX D

GEOLOGY AND SOILS

TABLE OF CONTENTS

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

APPENDIX D

GEOLOGY AND SOILS

TABLE OF CONTENTS (CONT'D)

PLATES

EXHIBITS: LABORATORY TEST DATA REPORTS

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

D1 SCOPE OF INVESTIGATIONS

Subsurface investigations performed by the Waterways Experiment Station (WES) at the site between November 1965 and January 1966 included 20 borings ranging in depth from 31 to 148 feet below lake bottom (see Plan of Borings, Plate D). Two piezometers were installed in pervious strata at locations shown in Plate D1. Two of the borings (U-1 and U-2) were drilled near the centers of the proposed gate bays, and undisturbed samples were recovered with 3-inch and 5-inch diameter Shelby tubes. The remaining 18 borings were drilled beneath the proposed lock chamber (borings L-1 through L-10) and the rock dam (borings D-1 through D-8) to depths averaging about 75 feet below lake bottom. Samples were obtained in these borings with a 1-3/8 inch inside diameter split spoon sampler. Additional split spoon sampler borings (S-1 through S-12, S-2A and S-9A) and borings (1 through 12) were completed late in 1968 and early in 1970, respectively, to further define the depth of the soft lacustrine deposits.

D2 Sample classification, water content determinations, grain size analyses, and strength tests on selected samples were performed by the Lower Mississippi Valley Division (LMVD) soils laboratory. Both undisturbed and split spoon samples were visually examined and logged geologically, with representative samples photographed for stratigraphic correlation purposes. The logs of the borings are shown on Plates D2 through D7; the soil borings legend is shown on Plate D8.

D3 AREAL GEOLOGY

Physiography. The proposed Seabrook Lock site is situated in the city of New Orleans at the south shore of Lake Pontchartrain, a large, shallow, oval-shaped, brackish to freshwater lake lying on the northern margin of the Mississippi River deltaic plain. In its natural state, the land along the south shore of the lake was characterized by small, irregular, sand-silt-shell beach south of which were large tracts of near-sea level marshes and swamps. However, the area has been completely altered by man; the original lakeshore area has been extended lakeward and raised to elevations of 5 to 10 feet, mean Gulf level, by the placement of artificial fill behind seawalls or bulkheads. Whereas the natural offshore lake bottom gradually and uniformly shoaled toward the shore from maximum depths of 16 feet, dredging has effectively deepened the entire area and currents associated with the Inner Harbor Navigation Canal have scoured the lake bottom to depths as great as 40 feet (Plate D1).

D4 Stratigraphy and geologic history.

The oldest deposits that were encountered in borings at the site are within the upper part of the several hundred feet thick Prairie formation of the late Pleistocene age. These deposits were laid down during the last major interglacial stage (Bradyan or Peorian) in a deltaic or shallow marine environment, were exposed to weathering and subaerial erosion during the last (Late Wisconsin) glaciation, and have been buried by onlapping Recent deposits within the last 5000 to 6000 years. The surface of the Prairie formation slopes gulfward at a rate of about 2 feet per mile from its outcrop area north of Lake Pontchartrain.

D5 The gulfward-thickening, onlapping wedge of Recent sediments can be subdivided into three lithologically distinctive units on the basis of the environments in which they were deposited. The first sediments deposited on the Prairie formation surface were laid down in a shallow water, nearshore gulf environment at a time when the postglacial rising sea level had risen to within 40 feet of its present level. During the final 40 feet of rise, a large east-west trending sand beach or barrier spit developed immediately south of the proposed site. During the active life of this barrier spit, sedimentation occurred in the site area in a partially sheltered, shallow water bay-sound environment. Termination of bay-sound conditions in the area was eventually brought about by the development of a Mississippi River delta to the south and west of New Orleans. During the life of this delta, and a succeeding one about 1000 years later and continuing to the present, the fine-grained alluvial sediments which were introduced into the area north of the barrier spit were winnowed and reworked on the bottom of a shallow lake (Lake Pontchartrain or its ancestral version) in a lacustrine environment. Thus, the entire Recent sedimentary sequence was deposited in relatively shallow, saline to brackish water and nowhere has been subaerially exposed.

D6 Earthquake history and structure.

The New Orleans area has experienced relatively minor earthquakes (Modified Mercalli Scale intensities of III or IV) on several occasions during the past 50 years; however, no destructive earthquakes have ever been recorded, and this area is classed as one of low seismic probability.

D7 Detailed surface and subsurface investigations have established the presence of numerous normal faults in the Prairie formation in the greater Lake Pontchartrain area. Although the borings at the Seabrook Lock site show no conclusive evidence for faulting, the configuration of the Prairie formation surface and irregularities in the various zones of the Prairie formation near borings U-1 and L-3 are somewhat suggestive of faulting. However, because of the lenticular nature of the deposits, the lack of any indication of recent fault movements in the area, and the probably minor influence on engineering design, no further investigation of possible faults at the site is considered necessary.

D8 The Prairie formation has undergone minor regional tilting and warping since deposition, but has not been affected by local folding or warping. The Recent deposits show no indications of structural deformation of any type.

D9 SITE GEOLOGY

Pleistocene deposits. On the basis of lithology, the uppermost 125 feet of the Prairie formation can be described in terms of four distinct zones. The uppermost zone, varying from about 5 to 20 feet in thickness, is lithologically quite variable, but essentially is composed of fine-grained, cohesive deposits. Mottled gray, gray and brown, or gray and green fat clays (CH) and lean clays (CL) are predominant, while gray or brown silts (ML) and silty sands (SM) are subordinate. Some of the clays are laminated with silts or sands, while others are relatively massive. Large variations occur in both the natural water contents of the clays (20 to 55 percent dry weight) and their consistencies (soft to stiff), and rapid lateral and vertical changes occur in both. The second zone in the Prairie formation lithologically is much more uniform and is characterized by massive brown and gray silty sand (SM). The zone varies in thickness from 15 to 45 feet and is interrupted only by thin layers of silt or layers containing disseminated organic fragments. The third zone, composed of uniform, dense, fine, brown or gray sand (SP) was encountered in most borings, but was penetrated completely only in boring U-2 where it was found to be 80 feet thick. The fourth and lowermost zone, encountered only in boring U-2, is composed of medium to stiff gray clays (CH and CL).

D10 As measured in the borings, the buried surface of the Prairie formation varies in elevation from -40 to -61 feet mean Gulf level. The lower elevations appear to follow a linear trend marked by borings L-3, D-4, and D-1 (Plate D1). This trend probably is a narrow, entrenched, relict stream course.

D11 Recent deposits.

The nearshore gulf deposits that immediately overlie the Prairie formation consist mainly of soft to medium, gray fat clays (CH) or, less frequently, lean clays (CL) with occasional thin sand layers and small shell fragments. Water contents are high, varying from 24 to 69 percent of dry weight and averaging almost 50 percent.

D12 In terms of both lithology and physical properties, these are the most uniform of the Recent deposits. However, the deposits vary irregularly in thickness from 0 to 19 feet, with the average thickness being about 6 feet. The thickest occurrence was encountered in boring D-1, Plate D5.

D13 The bay-sound deposits overlying the nearshore gulf deposits are characteristically noncohesive, consisting of soft to medium gray silt (ML), silty sand (SM), or fine sand (SP). All three types of materials are abundantly fossiliferous; shell fragments and occasionally whole shells are either scattered through the deposits or concentrated in thin layers.

D14 These deposits vary in thickness from 0 to 40 feet with the greater thicknesses being toward the south and the lesser thicknesses toward the north. Borings made by a private foundation engineering firm about 150 feet north of and parallel to the railroad shown in Plate D1 indicate that the bay-sound deposits thicken rapidly and grade laterally into the sands of the large barrier spit (paragraph D5) somewhere in a narrow zone between the railroad and the south end of the lock.

D15 The lacustrine deposits in the site area are composed of soft to very soft gray silt (ML), lean clay (CL), and fat clay (CH) with lesser amounts of silty sand (SM) or fine sand (SP). The sandier soil types usually characterize the actual lake bottom and extend downward for a few feet. All soil types contain abundant quantities of whole shell and/or shell fragments. The silts and lean clays exhibit extremely high water contents (sometimes greater than 100 percent dry weight), were observed to be little more than jelly-like oozes in some cases, and, as evidenced by the hydrogen sulfide (H_2S) and/or methane (CH_4) content, contain significant quantities of decaying organic matter.

D16 Lacustrine deposits are relatively thin or absent near the south end of the proposed lock location (because of a natural thinning plus dredging and/or scouring), but thicken northward and westward to as much as 20 ft. in borings L-8, L-9, D-2, and D-4 (Plates D3, D4 and D5).

D17 GROUNDWATER CONDITIONS

Two units, the Recent bay-sound deposits and zones 2 and 3 of the Prairie formation, will be significant with regard to construction site dewatering.

D18 Although the bay-sound deposits are relatively fine-grained and only moderately permeable, they are hydraulically connected with the water in Lake Pontchartrain and also with the large 40 feet thick, 2 mile wide, and 35 mile long buried sand barrier spit located only a short distance south of the proposed site.

D19 Piezometers were installed with tips at elevations of -66.8 feet (P-1) and -64.17 feet (P-2) near borings D-1 and D-6, respectively (Plate D5), to measure the piezometric surface in the two sandy zones of the Prairie formation which can be considered as a single aquifer. Initial readings indicate the piezometric surface to approximate the mean Gulf level in Piezometer No. 1 and to be about 2 feet higher than that level in Piezometer No. 2. Although artesian conditions exist, significantly higher readings than those observed are not anticipated, since (a) the Prairie formation sands are hydraulically connected with the lake water over large areas north and east of the proposed site and (b) the sands are not part of one of the major artesian aquifers in the New Orleans area.

D20 SOURCES OF CONSTRUCTION MATERIALS

For sources of construction materials refer to: "Lake Pontchartrain, Louisiana and Vicinity, Design Memorandum No. 12, Sources of Construction Materials" approved 30 August 1966.

D21 SUMMARY

The proposed Seabrook Lock site has been adequately explored by borings, and the major stratigraphic units have been identified and delineated.

D22 The lock and rock dam locations are underlain by 5 to 60 feet of Recent deposits which are primarily shell-bearing silts, silty sands, and fine sands but also contain lesser thicknesses of very soft to medium high water content clays.

D23 The Recent deposits are underlain by Pleistocene-Prairie formation deposits which, to a depth of 125 feet, consist of two thin clay zones separated by thick zones of fine sand and silty sand. The uppermost clay zone exhibits widely variable and somewhat typical strengths and water contents.

D24 Control of groundwater during construction will be important, since the sandy zones in the Recent deposits and the Prairie formation are hydraulically connected with the water in the lake. Prairie formation piezometric surface elevations should not exceed mean Gulf level by more than a few feet.

D25 Except for stone, all required construction materials are available locally in adequate quantities.

LABORATORY TESTS

D26 SCOPE

Laboratory testing consisting of visual classification of and natural water content determinations on all samples; mechanical analyses and Atterberg limits on selected samples; shear strength and consolidation tests on selected undisturbed samples; and natural density determinations on undisturbed samples of sand, together with maximum and minimum density determinations on remolded samples. The results of the laboratory tests are shown with the boring logs in Plates D2 through D7, in graphic form in Plates D9 through D11, and in tabular form in Tables 1 and 2. Detailed laboratory test reports are presented in exhibits 1 through 7.

D27 CLASSIFICATION AND DENSITY DATA

Atterberg limits data for the fine-grained Recent and Pleistocene soils are plotted on a plasticity chart in Plate D9. Grain-size curves on selected samples of the Recent deposits are shown in Plate D9; also shown are the range of grain-size curves for zones 2 and 3 of the Prairie formation. Uniformity of the foundation sandy materials in zones 2 and 3 is depicted by the narrow range in gradation for this portion of the foundation. The materials from zone 3 which consist primarily of fine sands (SP) are generally coarser than the materials in zone 2 which are classified as silty sands (SM).

D28 Results of natural density determinations on materials from zones 2 and 3 are shown plotted versus depth in Plate D9. The natural dry densities vary generally from 87 to 101 pcf. with an average value of about 89 pcf. for zone 2 and 97 pcf. for zone 3. The results of the maximum and minimum dry density determinations are also plotted versus depth in Plate D9. Relative density tends to increase with depth and averages about 60 percent, indicating that the foundation sands are in a medium dense condition. Relative densities significantly lower than 60 percent were indicated for three samples; however, the validity of these values appears questionable from inspection of the plots of density versus depth and density versus grain size.

D29 A correlation of natural dry density of the sands with the D_{50} grain size is shown in Plate D9. Also shown is the correlation between the maximum and minimum density determinations and D_{50} size. Both maximum and minimum dry densities increase with increasing grain sizes, as does the natural dry density.

D30 SHEAR STRENGTH TESTS

Shear strength data on the foundation soils are summarized in Plate D10. The shear strength of the foundation clays comprising the nearshore Gulf deposits and zone 1 of the Prairie formation was determined from unconfined compression tests. The unconfined compression test results are plotted versus depth in Plate D10. The shear strengths vary generally between 400 and 1000 psf. The shear strength of the zone 1 Pleistocene clays appears to be about the same as that of the overlying Recent deposits. For design purposes, an average shear strength of $\phi = 0$, $c = 500$ psf. for the Recent and zone 1 Prairie clays appears to be a reasonably conservative assumption. It is to be noted that no shear strength determinations were made for the lacustrine or bay-sound deposits. Although some clays were found in these deposits, these materials were found to be essentially silt, sandy silts, and sands in the areas where undisturbed borings were made. Visual inspection of available samples indicate that the strength of the cohesive bay-sound deposits are essentially the same as that of the near-shore Gulf deposits. However, the lacustrine deposits are extremely soft.

D31 In the New Orleans area, the clays of zone 1 of the Prairie formation typically exhibit relatively high cohesive strengths because of postdepositional oxidation and/or desiccation. Although there can be little doubt that the uppermost Prairie formation deposits at the Seabrook Lock site were oxidized and desiccated, the cohesive strengths appear to be somewhat lower than what is typical. Drainage of saline waters from the clays and replacement with fresh water during the low sea level stands of the last glacial stage has been suggested as a cause for the zones of relatively low-strength clays.

D32 Results of the R and S tests are shown in Plate D10. A single R test on a silty sand with clay seams from the bay-sound deposit indicated a shear strength of Ø = 14 deg. c = 0.2 tsf. Pore pressures measured during this test, as shown in Figure A12, exhibit 5, permitted determination of the effective stresses at failure; the resulting effective shear strength based on the maximum stress ratio, $6'_1/6'_3$ was Ø = 31 deg, c = 0.11 tsf. (see Figure A13, exhibit 5). On the basis of experiences with similar soils, it is considered that an R design shear strength of Ø = 15 deg, c = 0.11 tsf. based on total stresses is reasonable for the silts and silty sands. A single triaxial compression S test (see Figure A15 in exhibit 5) conducted on silt from the lacustrine deposits indicated a shear strength of Ø = 42 deg, c = 0; this value is not considered representative of the actual S shear strength of the silts based on inspection of other samples from these deposits. For design purposes, an S shear strength value of Ø = 25 deg, c = 0 is considered reasonable. Shear strengths of the sands were determined from 4 S tests on undistrubed samples, 2 tests on specimens from 5-inch diameter samples being made using direct shear apparatus and 2 tests on specimens from 3-inch diameter samples being made using triaxial compression apparatus. The S shear strengths of the sands, including the Recent bay-sound deposits, Pleistocene sands, and silty sands, are relatively high with angles of internal friction between 34 and 42 deg, with the higher values being for the triaxial tests. For design purposes, an S shear strength value of Ø = 35 deg, c = 0 appears reasonable for these materials.

D33 CONSOLIDATION TESTS

Consolidation tests were performed on the foundation soils for the purpose of predicting settlements beneath the structure and levee tie-ins to the lock. Pressure-void ratio curves for soils tested are shown in Plate D11. Pertinent data for each sample are summarized in the table on the same plate. Because of the shape of the pressure-void ratio curves, reliable determinations of the compression index could not be made for the sands. For both the Recent and Pleistocene soils tested, indicated preconsolidation pressures were greater than the existing overburden pressures by about 2 tons per sq. ft.

D34 PERMEABILITY

While laboratory or field permeability tests were not conducted, coefficients of horizontal permeability for use in checking the design of the construction dewatering system and also in the design of permanent seepage control measures were derived from extensive empirical relationships between the D_{10} grain sizes and coefficients of permeability developed from field pumping tests on sands in the lower Mississippi River valley.* The silty sands of zone 2, with an average D_{10} size of 0.06 mm, are indicated to have a horizontal coefficient of permeability of 100×10^{-4} cm per sec. The fine sands of zone 3, with an average D_{10} size of 0.13 mm, are indicated to have a coefficient of permeability of 400×10^{-4} cm per sec.

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* U. S. Army Engineer Waterways Experiment Station, Investigation of Underseepage and Its Control, Lower Mississippi River Levees, Waterways Experiment Station Technical Memorandum 3-424, Vicksburg, Miss., October 1956

Sheet 1 of 2
19 MAR. '66

TABLE 1 - TEST DATA SUMMARY

SEABROOK LOCK

FEATURE 441-3843/71.155

BORING NO.	SAM. NO.	DEPTH OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS				ATTERBERG LIMITS		SPECIFIC GRAVITY G	RELATIVE DENSITY DETERMINATIONS				SHEAR DATA												PERMEABILITY		CONSOLIDATION DATA				REMARKS
				GRAVEL %	SAND %	FINES %	D_{10}	LL	PL		[illegible]	MINIMUM DRY DENSITY LBS/CU FT	D_r %	MAXIMUM DRY DENSITY LBS/CU FT	INITIAL e	DRY DENSITY LBS/CU FT	w_I %	w_F %	S_I %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_m T/SQ FT	σ_I* T/SQ FT	c** T/SQ FT	ϕ DEGREES	e	K FT/MIN	P_o T/SQ FT	P_c T/SQ FT	C_c	t_{50}*	
U-1																																	
"	2	4.3-5.2	SAND (SP), gray,	0	98	2		-	-	2.66	Est.					95.6	24.9	26.6	89.9	DS	3.00^2 x 0.502	S	0.50	0.41	0	37							
			contains shell fragments													95.4	24.4	27.5	87.6		"		1.50	1.27									
																96.2	24.8	26.1	90.9		"		3.00	2.05									
"	5	8.9-11.3	Specimen ① - SILTY SAND (SM),							2.65	Est.					101.1	23.5		97.7	UC	1.38 x 3.00		0	0.24	0.12								
			gray, contains several thin horizontal																														
			clayey seams. Specimen ② - SILTY CLAY																														
			(CL), gray, contains sandy pockets					29	21	2.67	Est.					84.8	35.4		97.6		"		0	0.22	0.11								
"	6	11.8-14.2	CLAY (CH), green, contains					—	—	2.70	From	Consolidation Test				70.8	51.3		100	UC	1.38 x 3.00		0	0.82	0.41								
			shells and small sandy pockets													69.1	53.3		99.9		"		0	0.52	0.26								
	6	"	" "		"			54	18	2.70						70.8	50.6	38.7	98.4		2.50 x 0.752	—	—	—	—	Consolidation Test	—	—	—	—	—	—	See test report plate.
"	7	15.1-17.5	CLAY (CL), gray, contains					41	15	2.69	Est.					84.9	36.1		98.5	UC	1.38 x 3.00		0	0.89	0.44								
			shells and sandy pockets													87.3	33.6		100		"		0	1.02	0.51								
"	8	18.1-19.2	CLAY (CL), green, contains					35	17	2.67	Est.					105.0	22.4		100	UC	1.38 x 3.00		0	0.91	0.45								
			small sandy pockets													100.4	23.9		96.8		"		0	0.65	0.32								
"	9	19.3-20.4	CLAY (CH), green, contains					78	20	2.71						92.7	30.0	30.3	98.2		4.25 x 1.200	—	—	—	—	Consolidation Test	—	—	—	—	0.20		
			sandy pockets																														
"	11	22.4-23.3	CLAY (CH), green,					63	19	2.70	Est.					93.1	29.8		99.2	UC	1.38 x 3.00		0	0.45	0.22								
			contains horizontal sandy seams													91.7	30.3		97.7		"		0	0.45	0.22								
"	13	27.1-28.8	SILTY SAND (SM),	0	83	17					92.7	81.3	63.5	100.8																			
			gray																														
"	15	33.1-35.6	SILTY SAND	0	93	7		—	—	2.65	93.7	80.9	59.1	105.2																			
			(SP-SM), brown													92.9	28.6	27.3	97.1		2.50 x 0.753	—	—	—	—	Consolidation Test	—	—	—	—	0.06		
"	17	42.1-44.5	SILTY SAND	0	79	21					72.9	65.9	70.0	87.9																			
			(SM), gray																														

* Or q_u, Unconfined Compressive Stress, T/Sq.Ft

** Or S_u, Undrained Shear Strength T/Sq. Ft.

ENG FORM 1 MAR 53 2086 (PREVIOUS EDITIONS MAY BE USED) (EM 1110-2-1803) (TRANSLUCENT)

TC - Triaxial Compression
UC - Unconfined Compression
DS - Direct Shear
UU - Unconsolidated Undrained
S CD - Consolidated Drained
CU - Consolidated Undrained
*Values at Pressure T/SQ FT

TABLE 1 - SHEET 1

TABLE 1 (Continued) - TEST DATA SUMMARY

SEABROOK LOCK

Sheet 2 of 2
19 MAR. '66

FEATURE 441-3843/71.155

BORING NO.	SAM. NO.	DEPTH OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS				ATTERBERG LIMITS		SPECIFIC GRAVITY G	NAT. DRY DENSITY LBS/CU FT	RELATIVE DENSITY DETERMINATIONS			SHEAR DATA												PERMEABILITY		CONSOLIDATION DATA				REMARKS
				GRAVEL %	SAND %	FINES %	D_{10}	LL	PL			MINIMUM DRY DENSITY LBS/CU FT	D_r %	MAXIMUM DRY DENSITY LBS/CU FT	INITIAL e	DRY DENSITY LBS/CU FT	w_i %	w_F %	S_i %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_m T/SQ FT	σ_1 * T/SQ FT	C** T/SQ FT	ϕ DEGREES	e	K FT/MIN	P_o T/SQ FT	P_c T/SQ FT	C_c	t_{50} *	
U-1																																	
"	21	56.1-58.5	SILTY SAND (SM), gray	0	87	13					91.0	78.9	65.7	99.0																			
"	22	61.1-63.5	SILTY SAND (SM), gray					—	—	2.65						98.1	22.9	24.0	88.0		2.50 x 0.751						Consolidation Test						See test report plate
"	23	66.1-68.5	SAND (SP), brown	0	97	3					89.0	84.6	24.9	105.5																			
"	27	86.1-88.3	SAND (SP), brown	0	97	3					98.7	87.1	64.0	106.7																			

* Or q_u; Unconfined Compressive Stress, T/Sq.Ft.
** Or S_u; Undrained Shear Strength T/Sq. Ft.

ENG FORM 2086 1 MAR 53 (PREVIOUS EDITIONS MAY BE USED) (EM 1110-2-1803) (TRANSLUCENT)

TC – Triaxial Compression
UC – Unconfined Compression
DS – Direct Shear
UU – Unconsolidated Undrained
CD – Consolidated Drained
CU – Consolidated Undrained
* Values at Pressure T/SQ FT

TABLE 2 - TEST DATA SUMMARY

FEATURE 441-3843/71.155

SEABROOK LOCK

Sheet 1 of 2
19 MAR '66

				MECHANICAL ANALYSIS				ATTERBERG LIMITS				RELATIVE DENSITY DETERMINATIONS			SHEAR DATA												PERMEABILITY		CONSOLIDATION DATA				
BORING NO.	SAM. NO.	DEPTH OF SAMPLE	LABORATORY CLASSIFICATION	GRAVEL %	SAND %	FINES %	D_{10}	LL	PL	SPECIFIC GRAVITY G	NAT. DRY DENSITY LBS/CU FT	MINIMUM DRY DENSITY LBS/CU FT	D_r %	MAXIMUM DRY DENSITY LBS/CU FT	INITIAL e	DRY DENSITY LBS/CU FT	w_1 %	w_F %	S_1 %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_m T/SQ FT	σ_1 * T/SQ FT	c ** T/SQ FT	φ DEGREES	e	k FT/MIN	P_o T/SQ FT	P_c T/SQ FT	C_c	t_{50} *	REMARKS
U-2	3A	7.0-9.4	SILT (ML), brown,	0	7	93		27	26	2.66	Est.					92.6	28.6	29.0	95.9	TC	1.40 x 3.00	S	1.0	5.43	0	42							
			contains a few thin													88.3	32.0	31.4	96.6		"		2.0	8.65									
			horizontal sandy layers													93.0	28.6	26.5	96.8		1.39 x 3.00		3.0	15.06									
"	3	14.5-16.8	SILTY SAND	0	73	27		29	22	2.67						81.7	37.1	30.5	95.1		2.50 x 0.751					Consolidation Test							See test report plate
			(SC-SM), brown																														
"	5	18.6-19.5	SILTY SAND (SM), gray, contains					—	—	2.66						86.1	34.9	32.8	99.9	TC	1.39 x 3.00	R	1.0	2.03	0.20	14							Back Pressure
			shell fragments and clayey seams													88.0	33.9	29.2	100		1.38 x 3.00		2.0	3.56									Saturation
																84.7	36.0	30.6	99.8		"		3.0	5.51									(70,60,70, PSI)
"	5	18.6-19.5	SILTY SAND (SM), gray, contains					—	—	2.66	From R Test					90.0	28.5	32.2	89.7	DS	3.00^2 x 0.502	S	0.50	0.34	0	34							
			shell fragments													91.7	28.3	30.3	82.1		"		1.50	0.99									
																89.6	29.2	29.1	91.1		"		3.00	1.64									
"	6	20.5-22.9	SANDY CLAY (CH), gray					—	—	2.70	From Consolidation Test					65.2	56.5		96.6	UC	1.39 x 3.00		0	0.70	0.35								
																66.0	55.8		97.2		"		0	0.91	0.45								
"	6	20.5-22.9	CLAY (CH), gray, contains					—	—	2.70						56.3	73.4	55.0	99.6		2.50 x 0.766					Consolidation Test							See test report plate
			small sandy pockets																														
"	7	23.5-26.0	SILTY CLAY (CL), gray,					—	—	2.67	Est.					81.0	39.0		98.2	UC	1.38 x 3.00		0	0.93	0.46								
			contains small sandy pockets													80.6	39.6		98.8		"		0	0.77	0.38								
"	8	26.5-28.9	Conglomerate of CLAY (CH),					—	—	2.68	From R Test					100.7	23.6		95.7	UC	1.38 x 3.00		0	0.38	0.19	Insufficient material for duplicate test							
			CLAYEY SAND (SC), and SILTY SAND (SM), gray																														
"	8	26.5-28.9	Conglomerate of CLAY (CH),					40	15	2.68						102.7	19.8	21.5	84.4	TC	1.39 x 3.00	R	1.0	2.99	0.10	27							Back Pressure
			CLAYEY SAND (SC), and SILTY													106.2	19.9	20.0	92.8		1.40 x 3.00		2.0	5.62									Saturation
			SAND (SM), gray													108.5	17.2	19.0	85.0		"		3.0	9.96									(70,50,80, PSI)
"	12	38.0-40.5	SANDY SILT	0	45	55		—	—	2.67	88.6	77.6	60	97.9																			
			(ML), brown													93.2	27.5	26.8	93.4		2.50 x 0.753					Consolidation Test							
"	13	41.0-43.5	SILTY SAND	0	76	24		—	—		86.9	79.4	42	99.8										*	Or q_u, Unconfined Compressive Stress, T/Sq.Ft.								
			(SM), brown																					**	Or S_u, Undrained Shear Strength, T/Sq.Ft.								

ENG FORM 1 MAR 53 2086 (PREVIOUS EDITIONS MAY BE USED) (EM 1110-2-1803) (TRANSLUCENT)

TC - Triaxial Compression
UC - Unconfined Compression
DS - Direct Shear
Q - Unconsolidated Undrained
S - Consolidated Drained
R - Consolidated Undrained
* Values at Pressure T/SQ FT

TABLE 2 - SHEET 1

TABLE 2 (Continued) - TEST DATA SUMMARY

SEABROOK LOCK

Sheet 2 of 2
22 MAR '66

FEATURE 441-3843/71.155

				MECHANICAL ANALYSIS				ATTERBERG LIMITS			RELATIVE DENSITY DETERMINATIONS				SHEAR DATA												PERMEABILITY		CONSOLIDATION DATA				
BORING NO.	SAM. NO.	DEPTH OF SAMPLE	LABORATORY CLASSIFICATION	GRAVEL %	SAND %	FINES %	D_{10}	LL	PL	SPECIFIC GRAVITY G	NAT DRY DENSITY LBS/CU FT	MINIMUM DRY DENSITY LBS/CU FT	D_r%	MAXIMUM DRY DENSITY LBS/CU FT	INITIAL e	DRY DENSITY LBS/CU FT	w_i %	w_F %	S_i %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_m T/SQ FT	σ_1 * T/SQ FT	c** T/SQ FT	ϕ DEGREES	e	K FT/MIN	P_o T/SQ FT	P_c T/SQ FT	C_c	t_{50}	REMARKS
U-2	16	50.0-52.5	SAND (SP), gray	0	98	2		—	—	2.65 Est	93.8	81.8	63	102.6		96.1	25.3	26.8	92.4	TC	1.39 x 3.00	S	1.0	5.49	0	41							
																95.8	26.0	26.8	94.4		"		2.0	9.40									
																96.6	25.5	25.3	94.3		"		3.0	14.12									
"	19	63.0-65.5	SAND (SP),	0	97	3		—	—	2.66	93.4	85.1	43	107.2																			
			brown													98.8	25.1	24.1	98.5		2.50 x 0.751						Consolidation Test						
"	21	73.0-75.5	SAND (SP), brown	0	98	2		—	—		101.4	84.6	81	106.4																			
"	23	83.0-85.5	SAND(SP), brown	0	99	1		—	—	2.65 Est.	100.3	84.1	84	104.1		100.3	22.9	26.6	92.8	TC	1.39 x 3.00	S	1.0	5.69	0	42							
																99.7	22.8	26.4	91.1		1.40 x 3.00		2.0	10.10									
																99.7	22.8	24.8	91.1		1.41 x 3.00		3.0	15.00									
"	25	93.0-95.5	SAND(SP), brown	0	98	2		—	—		99.3	85.1	75	105.3																			
"	27	103.0-105.5	SAND(SP), brown	0	97	3		—	—	2.66	100.1	87.1	81	103.8																			
																103.1	20.0	21.0	87.4		2.50 x 0.751						Consolidation Test						
"	33	133.0-135.4	SILTY CLAY(CL),					—	—	2.67 Est.						97.8	25.7		97.5	UC	1.39 x 3.00		0	2.09	1.04								
			gray													98.1	25.4		97.0		"		0	2.18	1.09								

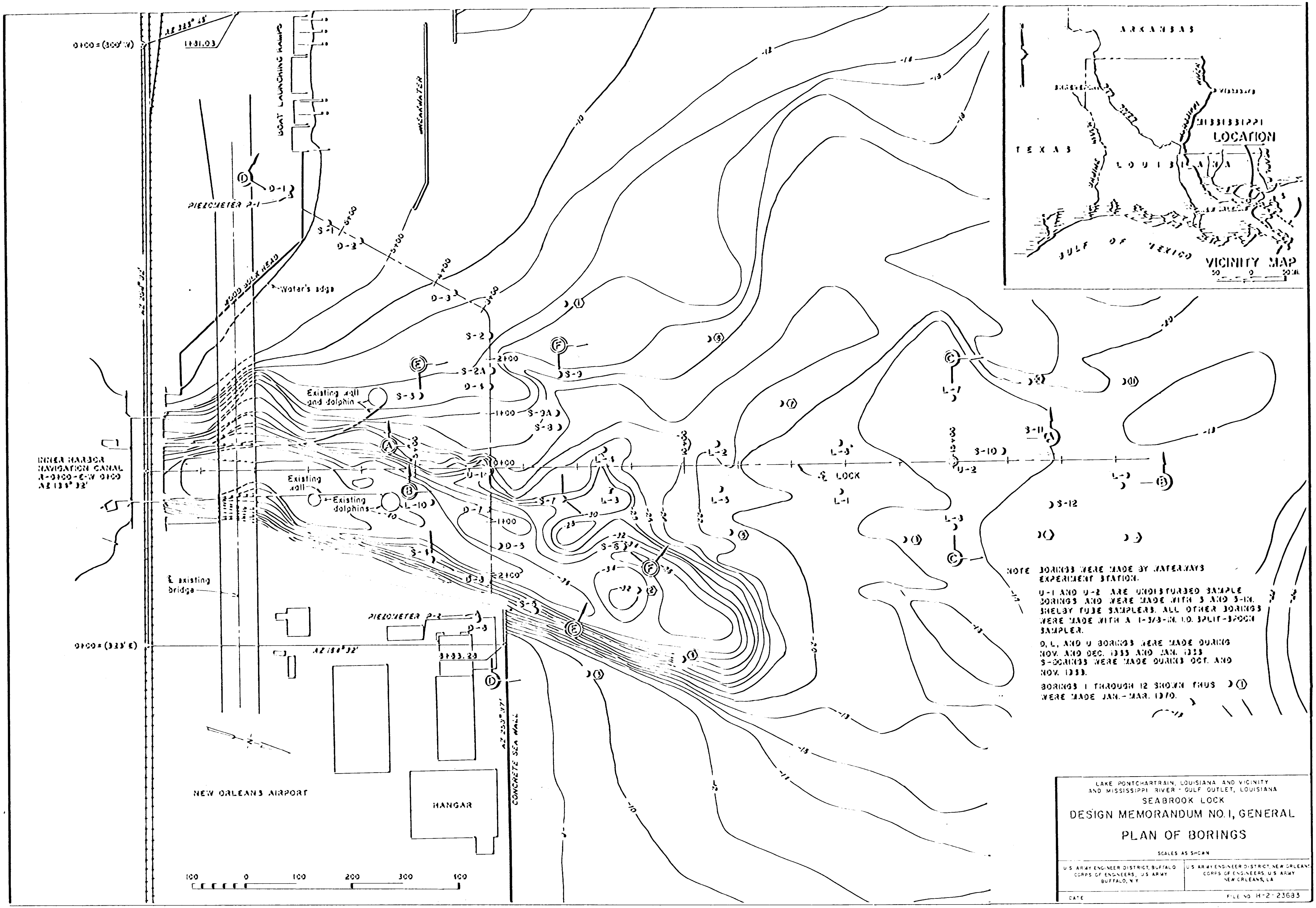
* Or q_u; Unconfined Compressive Stress T/Sq.Ft.

** Or S_u; Undrained Shear Strength T/Sq.Ft.

ENG FORM 2086 1 MAR 53 (PREVIOUS EDITIONS MAY BE USED) (EM 1110-2-1803) (TRANSLUCENT)

TC - Triaxial Compression
UC - Unconfined Compression
DS - Direct Shear
Q - Unconsolidated Undrained
S - Consolidated Drained
R - Consolidated Undrained
*Values at Pressure T/SQ FT

TABLE 2-SHEET 2



NOTE BORINGS WERE MADE BY WATERWAYS EXPERIMENT STATION.

U-1 AND U-2 ARE UNDISTURBED SAMPLE BORINGS AND WERE MADE WITH 3 AND 5-IN. SHELBY TUBE SAMPLERS. ALL OTHER BORINGS WERE MADE WITH A 1-3/8-IN. I.D. SPLIT-SPOON SAMPLER.

D, L, AND U BORINGS WERE MADE DURING NOV. AND DEC. 1955 AND JAN. 1956
S-BORINGS WERE MADE DURING OCT. AND NOV. 1955.

BORINGS 1 THROUGH 12 SHOWN THUS ⓵ WERE MADE JAN.-MAR. 1970.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

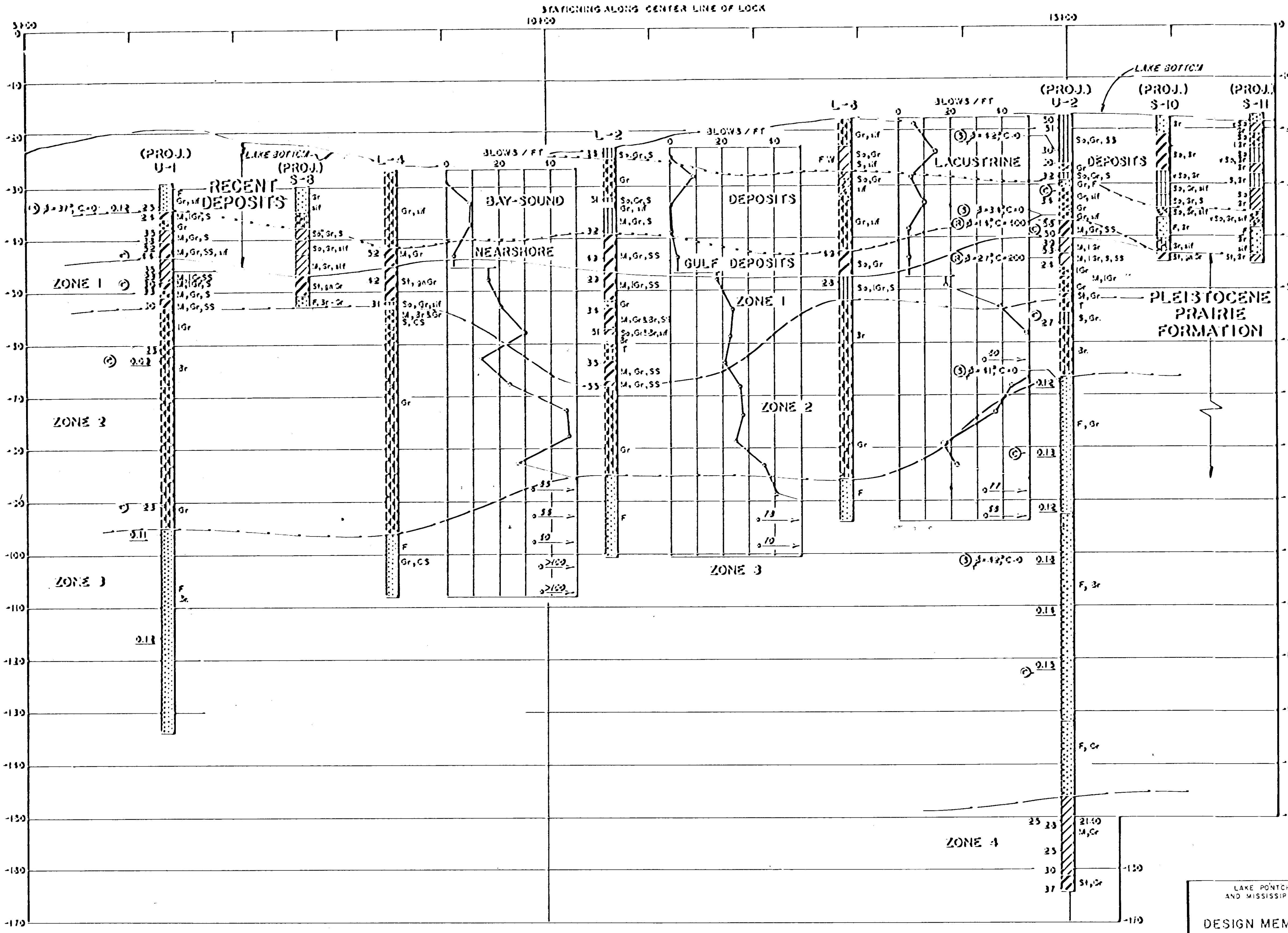
DESIGN MEMORANDUM NO. 1, GENERAL

PLAN OF BORINGS

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U.S. ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683

PLATE D1



LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

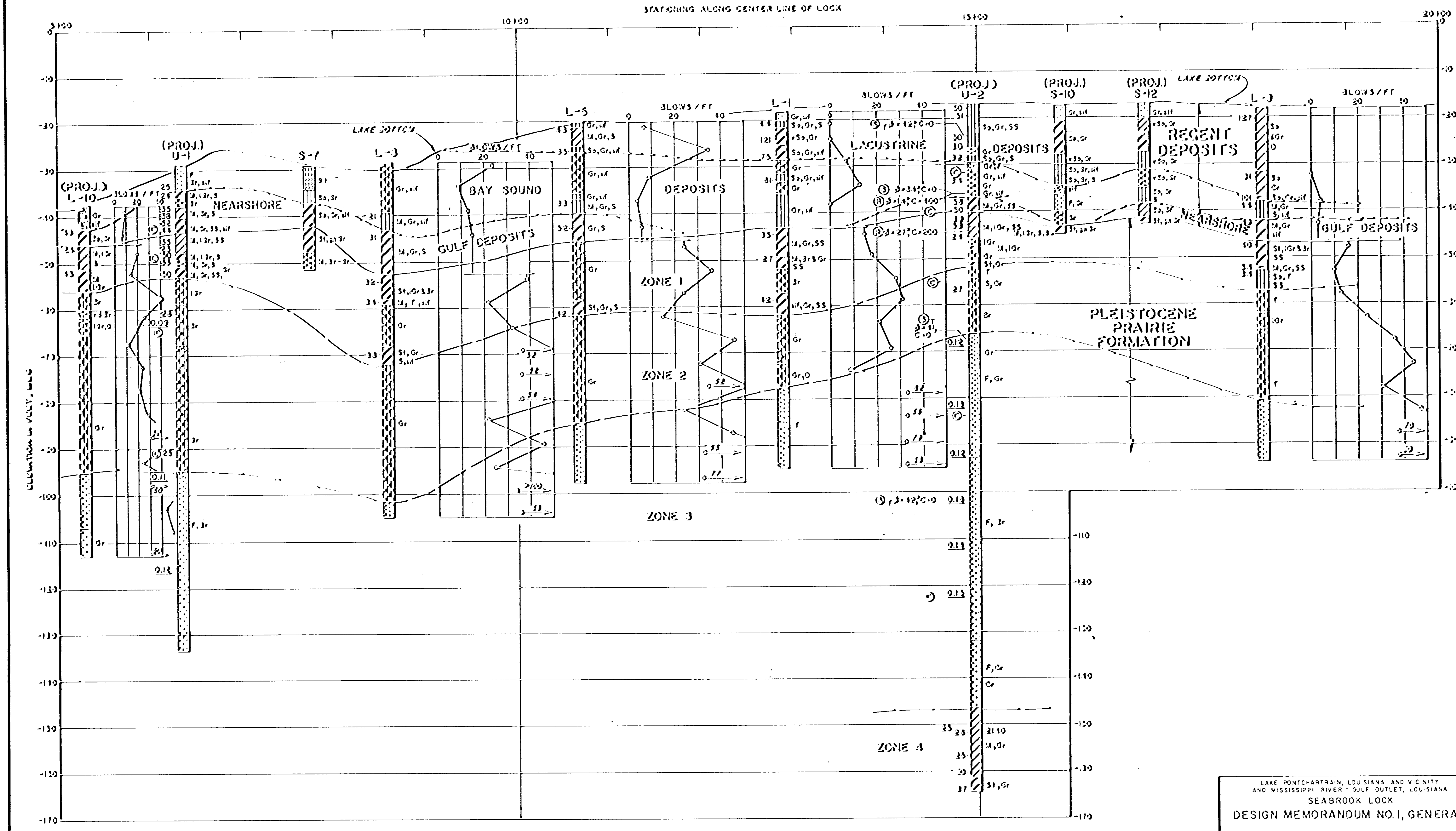
DESIGN MEMORANDUM NO. 1, GENERAL

LOGS OF BORINGS - SECTION A-A

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U.S. ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683

PLATE D2



LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK

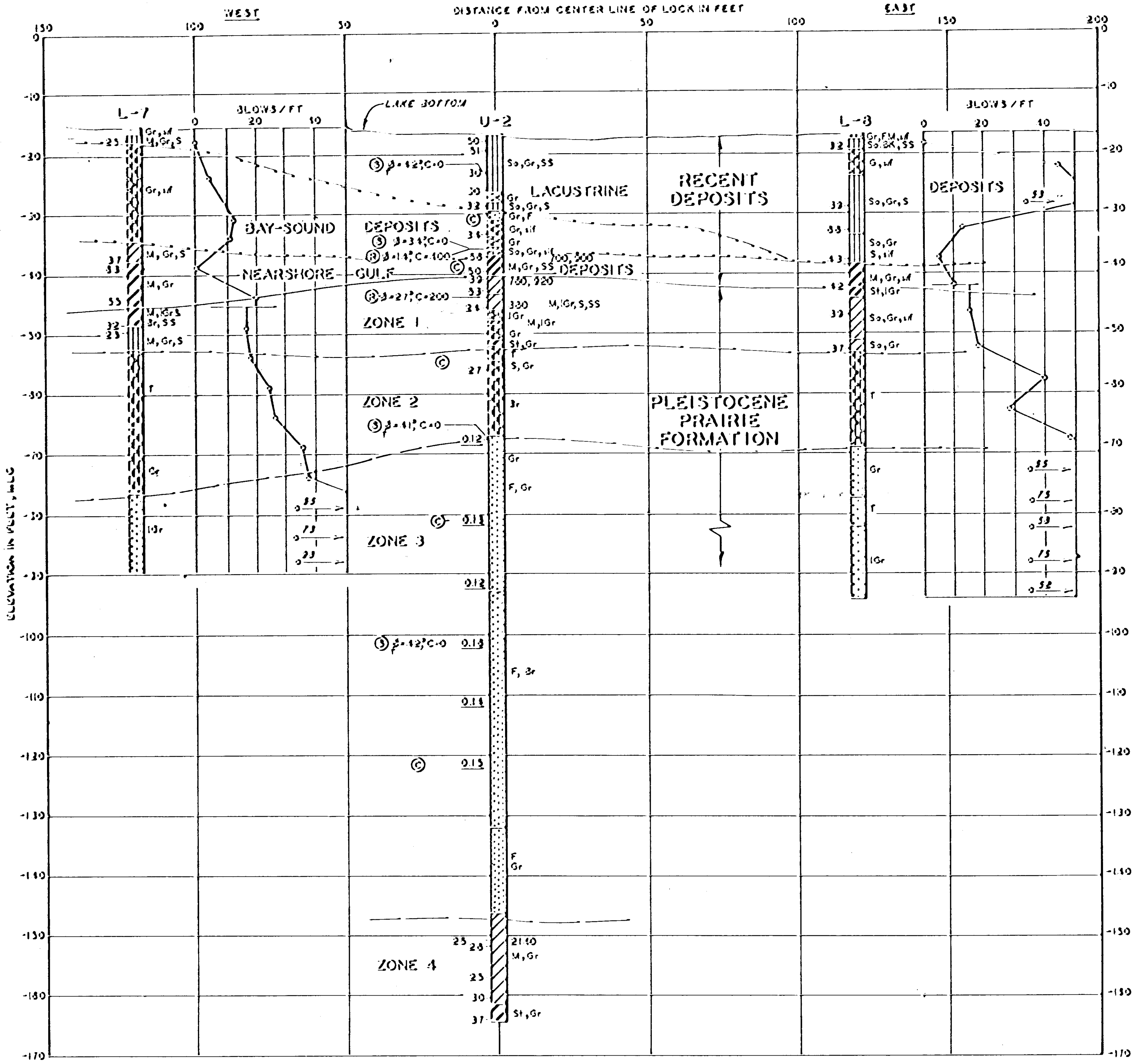
DESIGN MEMORANDUM NO. I, GENERAL

LOGS OF BORINGS-SECTION B-B

SCALES AS SHOWN

U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N Y	U S ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683

PLATE D3



LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

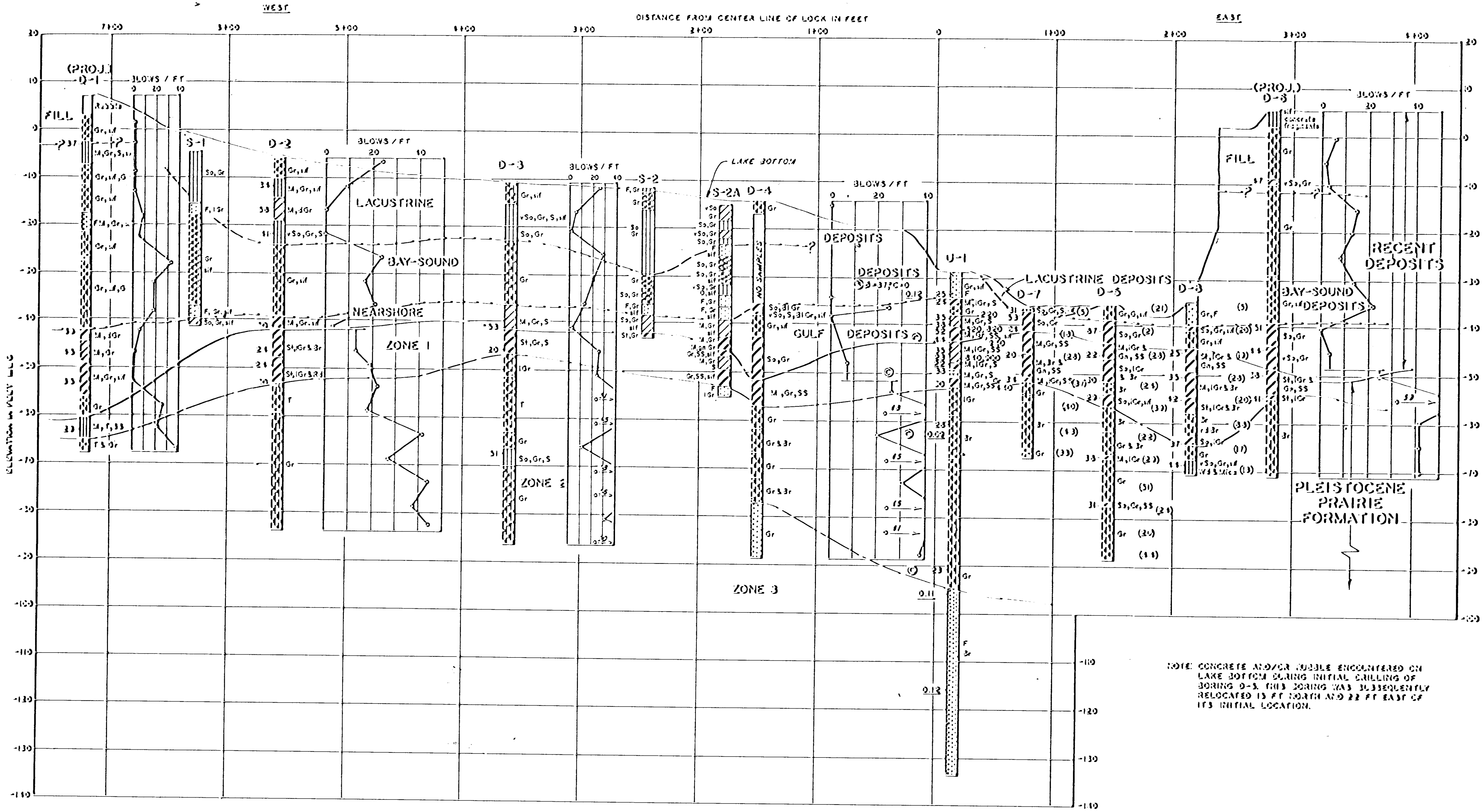
DESIGN MEMORANDUM NO. I, GENERAL

LOGS OF BORINGS-SECTION C-C

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U.S. ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683

PLATE D4



NOTE: CONCRETE AND/OR RUBBLE ENCOUNTERED ON LAKE BOTTOM DURING INITIAL DRILLING OF BORING D-5. THIS BORING WAS SUBSEQUENTLY RELOCATED 13 FT NORTH AND 22 FT EAST OF ITS INITIAL LOCATION.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

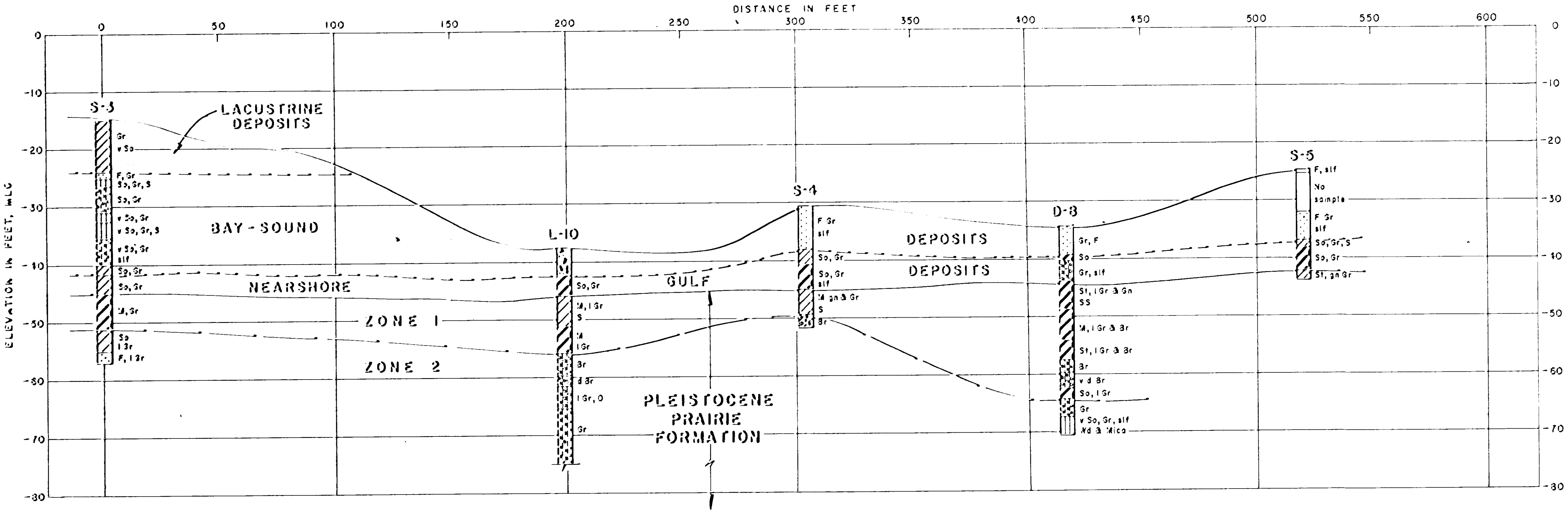
DESIGN MEMORANDUM NO. 1, GENERAL

LOGS OF BORINGS-SECTION D-D

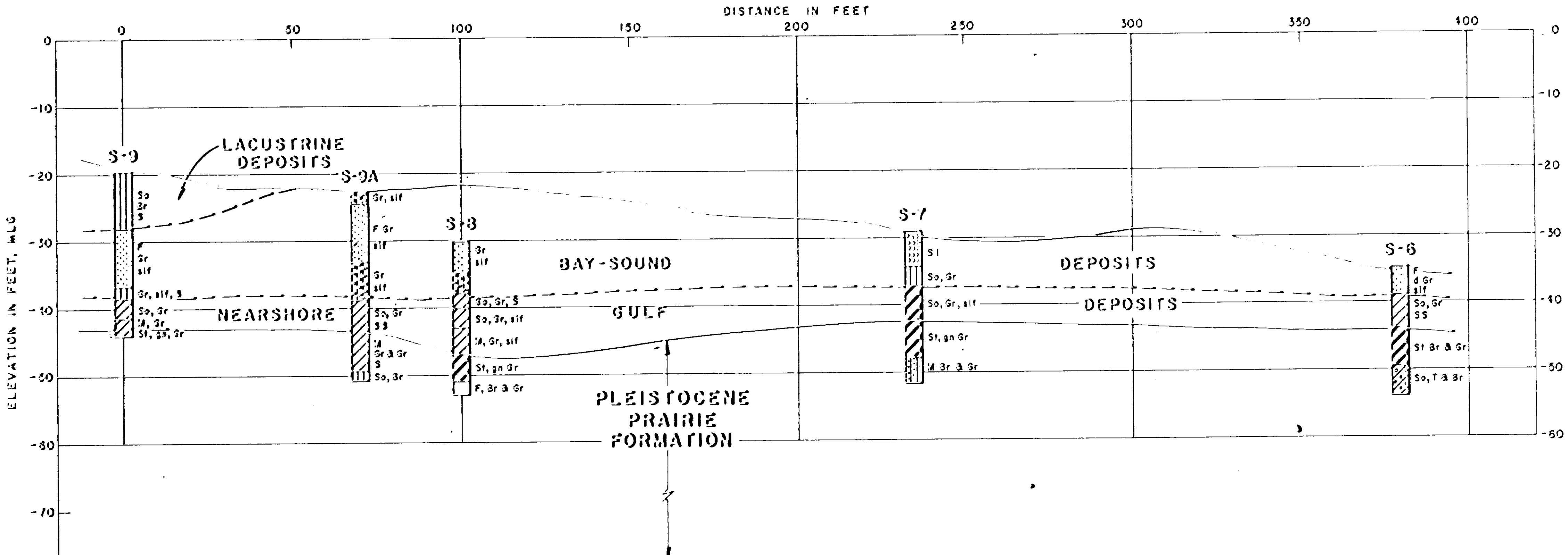
SCALES AS SHOWN

U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N.Y.	U S ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683

PLATE D5



SECTION E-E



SECTION F-F

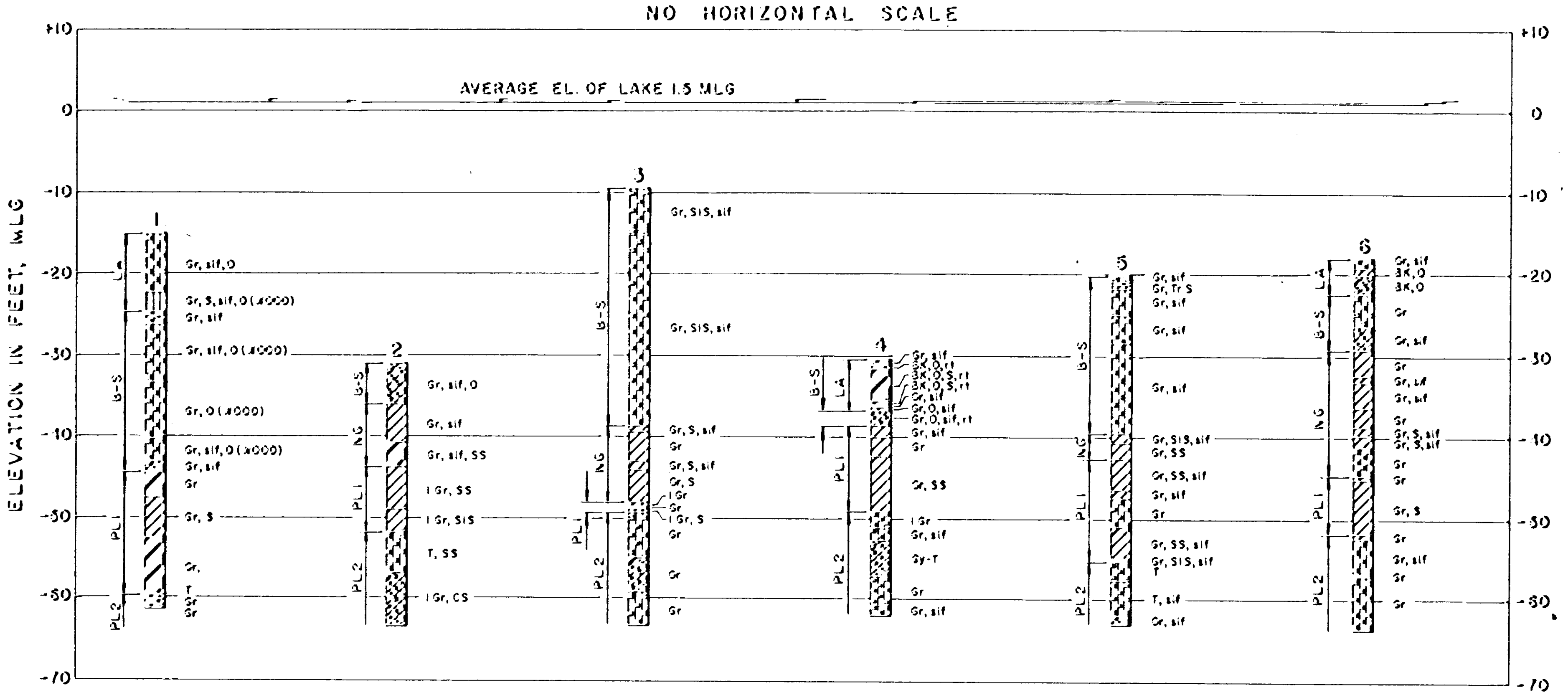
LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

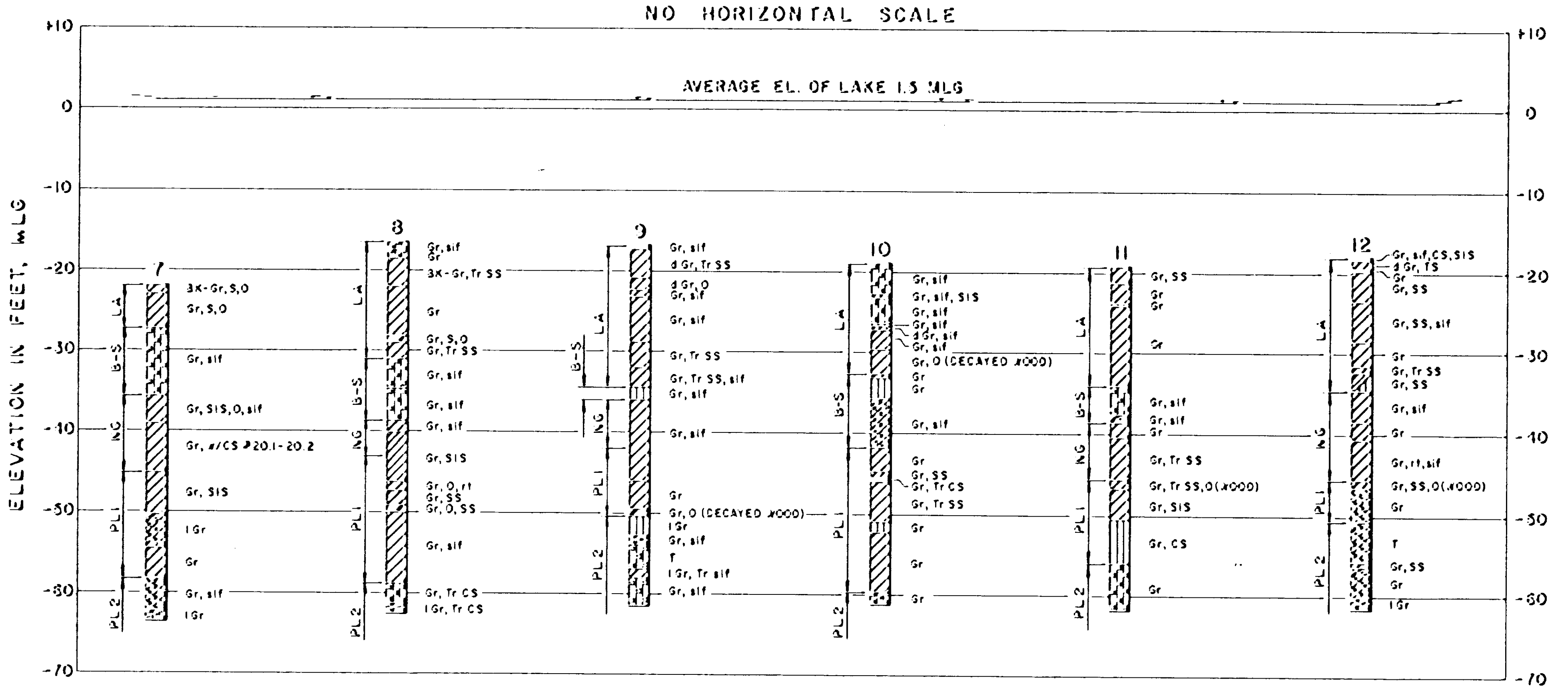
DESIGN MEMORANDUM NO. I, GENERAL
LOGS OF BORINGS
SECTIONS E-E AND F-F

SCALES AS SHOWN

U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N Y	U S ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO H-2-23683

PLATE D6





SOIL LEGEND

▥	SILT	◸	FAT CLAY
▨	SILTY SAND	▨	LEAN CLAY
▩	CLAYEY SAND		

STRATIFICATION LEGEND

(SHOWN LEFT OF BORING)

LA - LACUSTRINE DEPOSITS } RECENT
B-S - BAY SOUND DEPOSITS } RECENT
NG - NEAR SHORE GULF DEPOSITS } RECENT
PL 1 - PLEISTOCENE PRAIRIE FORMATION, ZONE 1
PL 2 - PLEISTOCENE PRAIRIE FORMATION, ZONE 2

DESCRIPTIVE SYMBOLS

(SHOWN RIGHT OF BORING)

SYMBOL	COLOR	MODIFICATION	SYMBOL
Gr	GRAY	TRACE	Tr
T	TAN	ROOTLETS	rt
l Gr	LIGHT GRAY	SHELL FRAGMENTS	slf
BK	BLACK	ORGANIC MATTER	O
Gyl-T	GRAYISH-TAN	CLAY STRATA/LENSES	CS
BK-Gr	BLACKISH-GRAY	SILT STRATA/LENSES	SIS
d Gr	DARK GRAY	SAND STRATA/LENSES	SS
		SANDY	S

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL
LOGS OF BORINGS (1 THROUGH 12)
MADE JAN. - MAR. 1970
SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U.S. ARMY NEW ORLEANS, LA.
DATE:	FILE NO. H-2-23683

PLATE D7

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION		TYPE	LETTER SYMBOL	SYMBOL	TYPICAL NAMES
COARSE - GRAINED SOILS More than half of material is larger than No 200 sieve size	GRAVELS More than half of coarse fraction is larger than No 4 sieve size	CLEAN GRAVEL (Little or No Fines)	GW	▨	GRAVEL, Well Graded, gravel - sand mixtures, little or no fines
			GP	▨	GRAVEL, Poorly Graded, gravel - sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable Amount of Fines)	GM	▨	SILTY GRAVEL, gravel - sand - silt mixtures
			GC	▨	CLAYEY GRAVEL, gravel - sand - clay mixtures
	SANDS More than half of coarse fraction is smaller than No 4 sieve size	CLEAN SAND (Little or No Fines)	SW	▨	SAND, Well - Graded, gravelly sands
			SP	▨	SAND, Poorly - Graded, gravelly sands
		SANDS WITH FINES (Appreciable Amount of Fines)	SM	▨	SILTY SAND, sand - silt mixtures
			SC	▨	CLAYEY SAND, sand - clay mixtures
FINE - GRAINED SOILS More than half the material is smaller than No 200 sieve size		SILTS AND CLAYS (Liquid Limit < 50)	ML	▨	SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity
			CL	▨	LEAN CLAY; Sandy Clay; Silty Clay; of low to medium plasticity
			OL	▨	ORGANIC SILTS and organic silty clays of low plasticity
		SILTS AND CLAYS (Liquid Limit > 50)	MH	▨	SILT, fine sandy or silty soil with high plasticity
			CH	▨	FAT CLAY, inorganic clay of high plasticity
			OH	▨	ORGANIC CLAYS of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	▨	PEAT, and other highly organic soil
WOOD			Wd	▨	WOOD
SHELLS			Sl	▨	SHELLS
NO SAMPLE					

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols

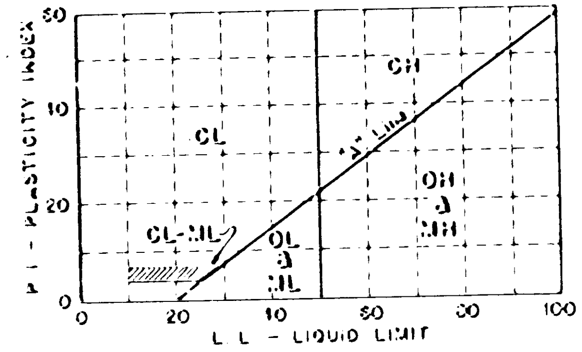
DESCRIPTIVE SYMBOLS

COLOR

COLOR	SYMBOL
TAN	T
YELLOW	Y
RED	R
BLACK	BK
GRAY	Gr
LIGHT GRAY	lGr
DARK GRAY	dGr
BROWN	Br
LIGHT BROWN	lBr
DARK BROWN	dBr
BROWNISH - GRAY	br Gr
GRAYISH - BROWN	gy Br
GREENISH - GRAY	gn Gr
GRAYISH - GREEN	gy Gn
GREEN	Gn
BLUE	Bl
BLUE - GREEN	Bl Gn
WHITE	Wh
MOTTLED	Mot

CONSISTENCY FOR COHESIVE SOILS

CONSISTENCY	COHESION IN LBS./SQ.FT. FROM UNCONFINED COMPRESSION TEST	SYMBOL
VERY SOFT	< 250	vSo
SOFT	250 - 500	So
MEDIUM	500 - 1000	M
STIFF	1000 - 2000	St
VERY STIFF	2000 - 4000	vSt
HARD	> 4000	H



PLASTICITY CHART

For classification of fine - grained soils

MODIFICATIONS

MODIFICATION	SYMBOL
Traces	Tr-
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata or lenses	CS
Silt strata or lenses	SiS
Sand strata or lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox
Crumbly	Cr
Loose	Lo
Vegetation	Veg

NOTES:

FIGURES TO LEFT OF BORING UNDER COLUMN "W OR D_{10}"

Are natural water contents in percent dry weight

When underlined denotes D_{10} size in mm *

FIGURES TO LEFT OF BORING UNDER COLUMNS "LL" AND "PL"

Are liquid and plastic limits, respectively

SYMBOLS TO LEFT OF BORING

▽	Ground - water surface and date observed
Ⓒ	Denotes location of consolidation test **
Ⓢ	Denotes location of consolidated - drained direct shear test **
Ⓡ	Denotes location of consolidated - undrained triaxial compression test **
Ⓠ	Denotes location of unconsolidated - undrained triaxial compression test **
Ⓣ	Denotes location of sample subjected to consolidation test and each of the above three types of shear tests **
FW	Denotes free water

FIGURES TO RIGHT OF BORING

Are values of cohesion in lbs./sq.ft. from unconfined compression tests

In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler (1 3/8" I.D., 2" O.D.) and a 140 lb. driving hammer with a 30" drop

Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample

Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

* The D_{10} size of a soil is the grain diameter in millimeters of which 10% of the soil is finer, and 90% coarser than size D_{10}

**Results of these tests are available for inspection in the U S Army Engineer District Office, if these symbols appear beside the boring logs on the drawings

Ⓢ$_T$ denotes triaxial test

GENERAL NOTES:

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of clause 4 of the contract.

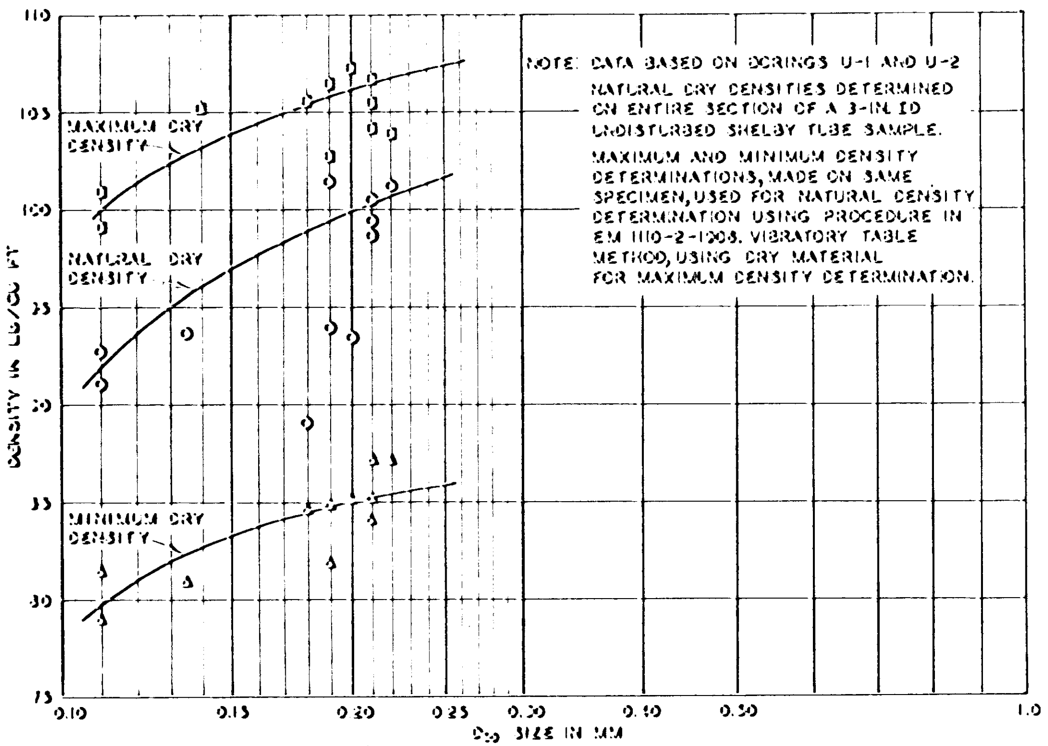
Ground - water elevations shown on the boring logs represent ground - water surfaces encountered on the dates shown. Absence of water surface data on certain borings implies that no ground - water data is available, but does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of these borings.

Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1, GENERAL
SOIL SYMBOLS

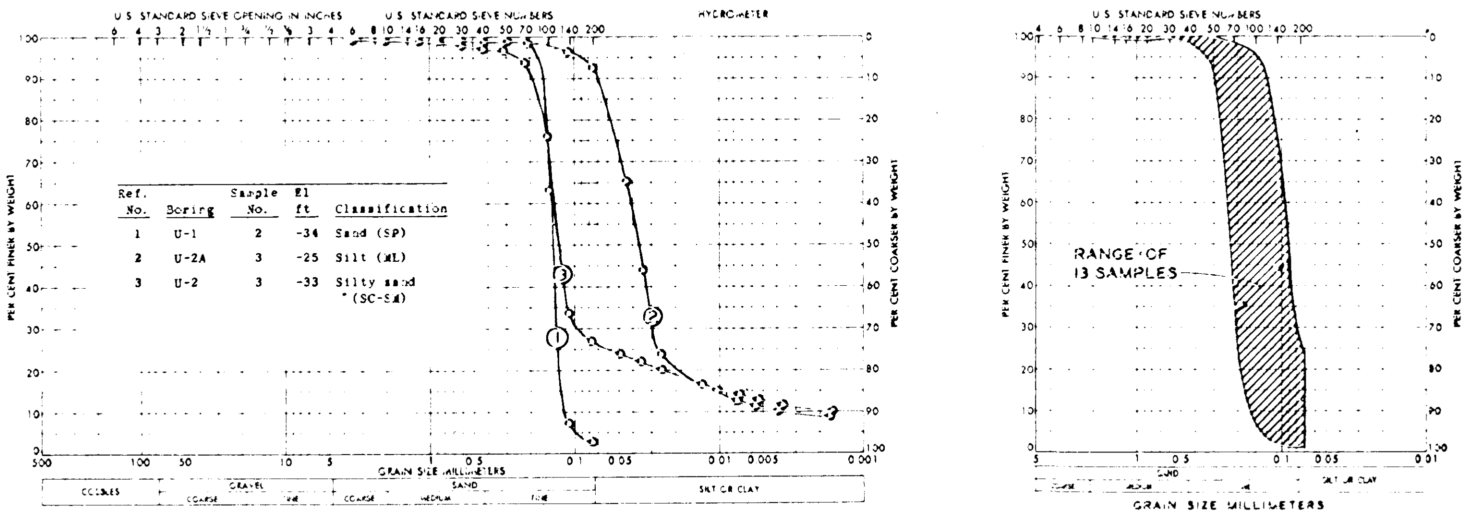
SCALES AS SHOWN

U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N Y	U S ARMY ENGINEER DISTRICT NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO H-2-23683



NOTE: DATA BASED ON BORINGS U-1 AND U-2
NATURAL DRY DENSITIES DETERMINED ON ENTIRE SECTION OF A 3-IN ID UNDISTURBED SHELBY TUBE SAMPLE.
MAXIMUM AND MINIMUM DENSITY DETERMINATIONS, MADE ON SAME SPECIMEN, USED FOR NATURAL DENSITY DETERMINATION USING PROCEDURE IN EM 1110-2-1908. VIBRATORY TABLE METHOD, USING DRY MATERIAL FOR MAXIMUM DENSITY DETERMINATION.

GRAIN SIZE VS DENSITY

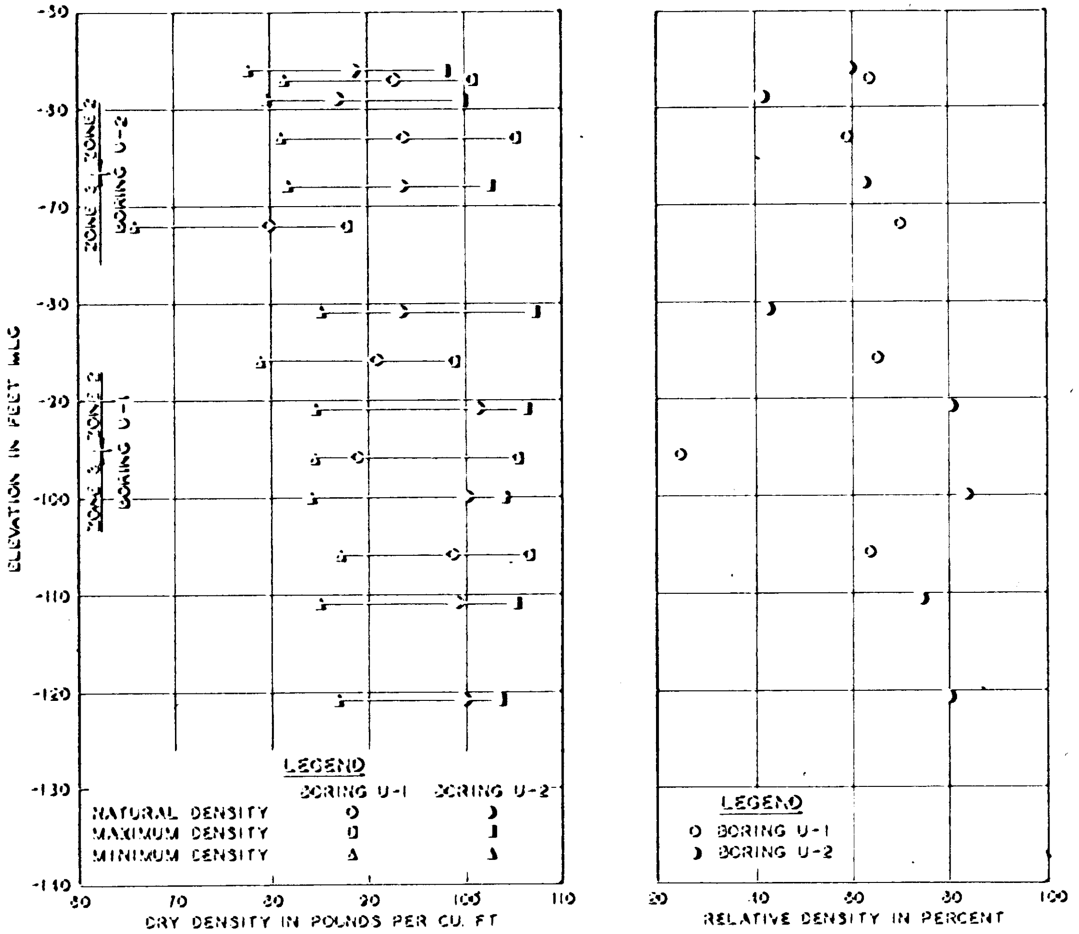


Ref. No.	Boring	Sample No.	El ft	Classification
1	U-1	2	-34	Sand (SP)
2	U-2A	3	-25	Silt (ML)
3	U-2	3	-33	Silty sand (SC-SM)

SILTS, SILTY SANDS, AND SANDS IN RECENT DEPOSITS

SILTY SANDS AND SANDS IN PLEISTOCENE ZONES 2 AND 3

GRAIN SIZE CURVES

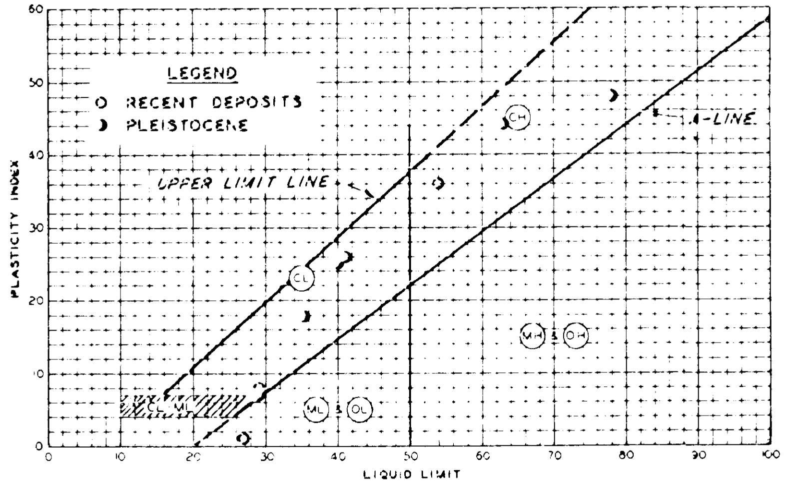


LEGEND

	BORING U-1	BORING U-2
NATURAL DENSITY	O	)
MAXIMUM DENSITY	0	1
MINIMUM DENSITY	Δ	Δ

LEGEND
O BORING U-1
) BORING U-2

DENSITY AND RELATIVE DENSITY VS DEPTH



LEGEND
O RECENT DEPOSITS
) PLEISTOCENE

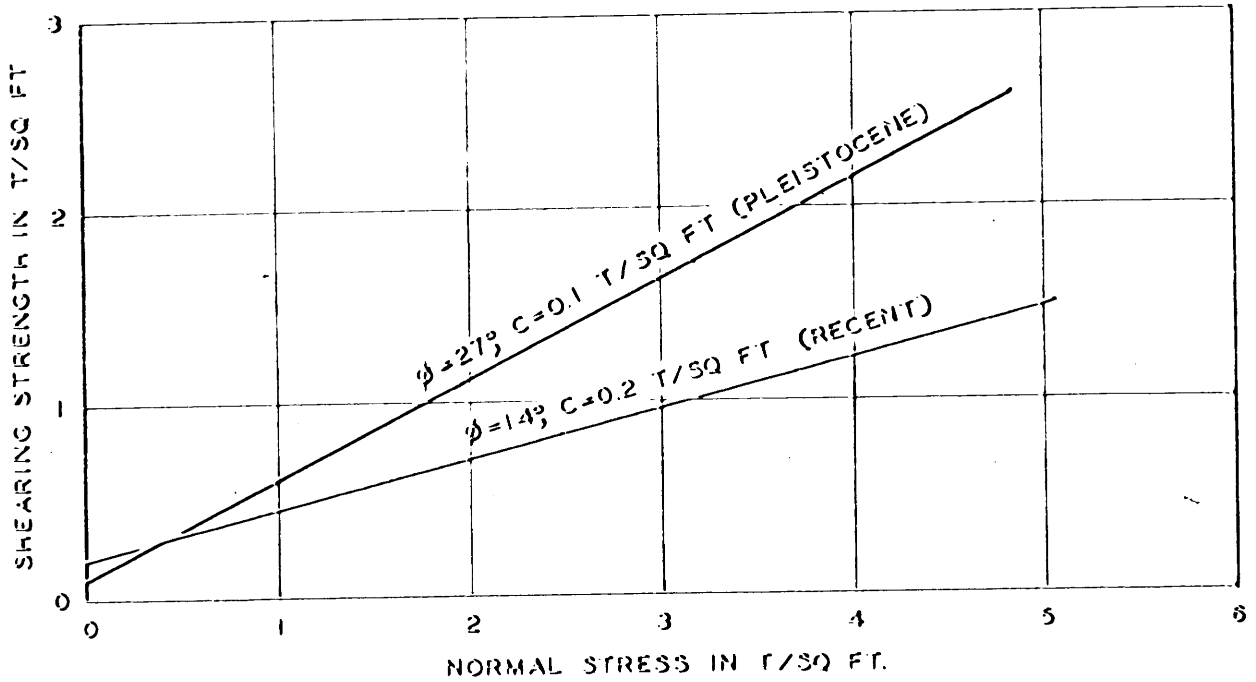
PLASTICITY CHART

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

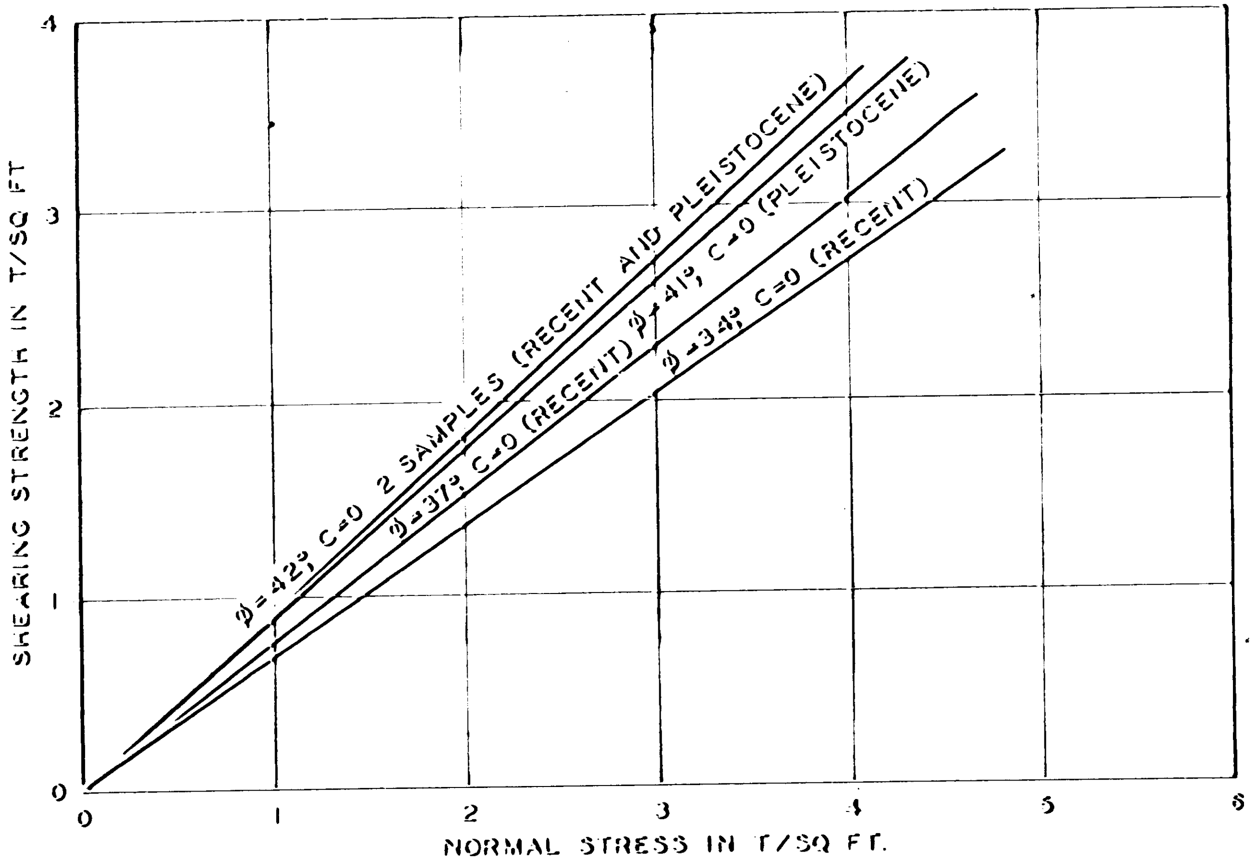
DESIGN MEMORANDUM NO. I, GENERAL CLASSIFICATION DATA AND DENSITY OF SANDS

SCALES AS SHOWN

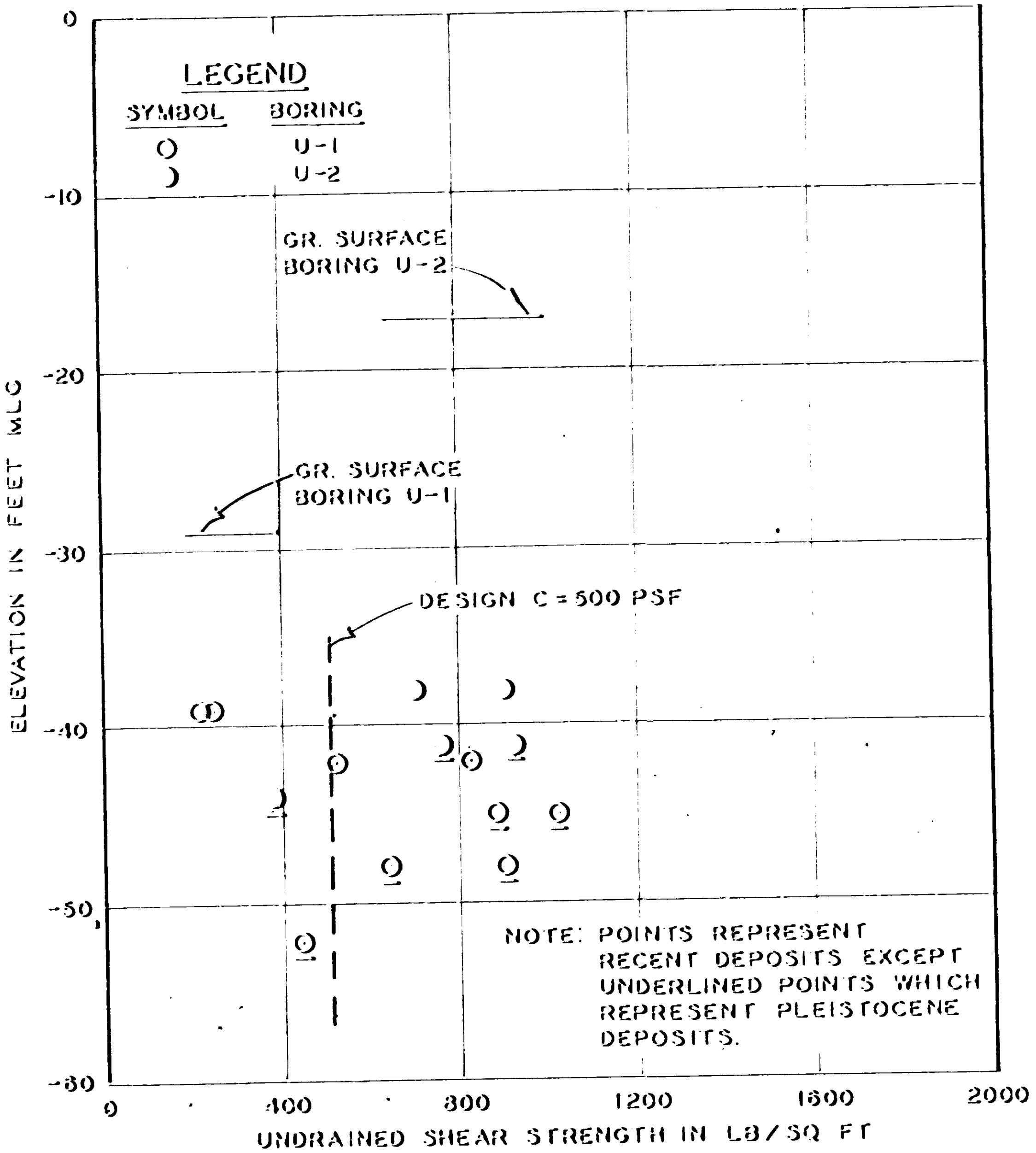
U S ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U S ARMY BUFFALO, N Y	U S ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U S ARMY NEW ORLEANS, LA
DATE	FILE NO H-2-23683



R TESTS-SILTY SANDS AND CLAYS



S TESTS-SILTS, SILTY SANDS AND SANDS



UNDRAINED SHEAR STRENGTH VS DEPTH

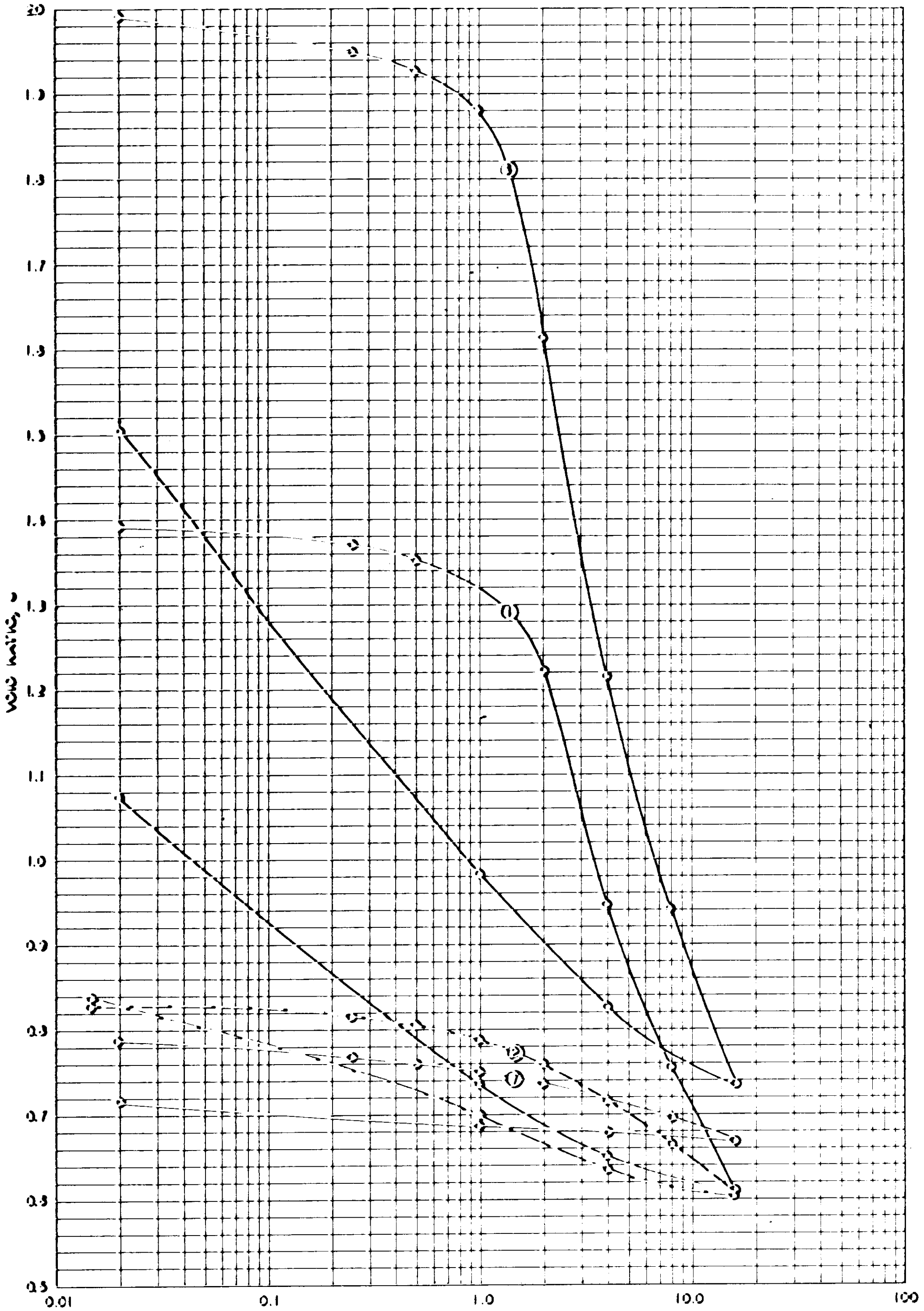
LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

DESIGN MEMORANDUM NO. 1, GENERAL

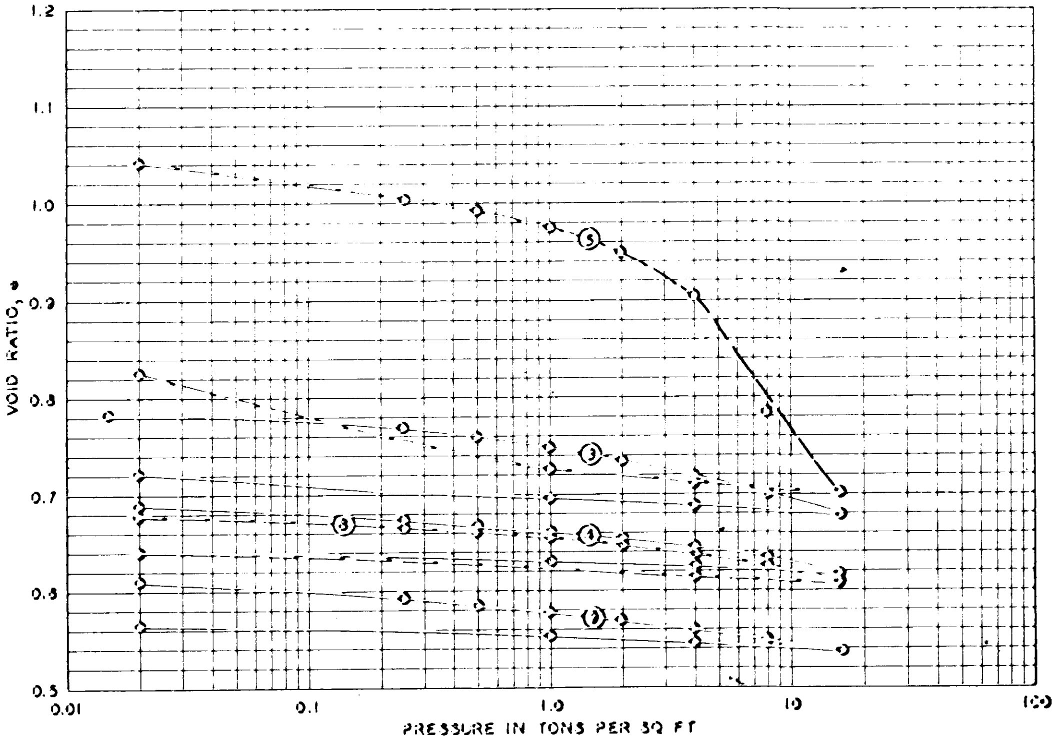
SHEAR STRENGTH DATA

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS U.S. ARMY NEW ORLEANS, LA
DATE	FILE NO. H-2-23683



CLAYS AND SILTS



SILTY SANDS AND SANDS

Ref. No.	Boring	Sample	Depth ft	El ft	Classification	Atterberg Limits LL	Atterberg Limits PL	Mechanical Analysis Sand	Mechanical Analysis Fines	w_o %	γ_d lb/cu ft	e_o	S_o %	Specific Gravity	P_o T/sq ft	P_c T/sq ft	C_c
1	U-1	6	13	-42	Clay (CH)	54	18	--	--	50.6	70.8	1.380	99.4	2.70	0.34	3.0	--
2	U-1	9	20	-49	Clay (CH)	78	20	--	--	30.0	92.7	0.828	98.2	2.71	0.53	3.8	0.20
3	U-1	15	34	-63	Silty sand (SP-SM)	--	--	93	7	28.6	92.9	0.781	97.1	2.65	0.84	--	0.08
4	U-1	22	62	-91	Silty sand (SM)	--	--	--	--	22.9	98.1	0.689	88.0	2.65	--	--	--
5	U-2	3	15	-32	Silty sand (SC-SM)	29	22	73	27	37.1	81.7	1.040	95.1	2.67	0.42	4.1	--
6	U-2	6	22	-39	Sandy clay (CH)	--	--	--	--	73.4	56.3	1.990	99.6	2.70	0.55	1.4	--
7	U-2	12	38	-55	Sandy silt (ML)	--	--	45	55	27.5	93.2	0.786	93.4	2.67	1.03	--	--
8	U-2	19	64	-81	Sand (SP)	--	--	97	3	25.1	98.8	0.678	98.5	2.66	--	--	--
9	U-2	27	104	-121	Sand (SP)	--	--	97	3	20.0	103.1	0.609	87.3	2.66	--	--	--

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA
SEABROOK LOCK

DESIGN MEMORANDUM NO. 1, GENERAL

CONSOLIDATION DATA

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, BUFFALO CORPS OF ENGINEERS, U.S. ARMY BUFFALO, N.Y.	U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS, U.S. ARMY NEW ORLEANS, LA.
DATE	FILE NO. H-2-23683

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1.GENERAL

APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 1 - UNCONFINED COMPRESSION TEST REPORTS, BORING U-1

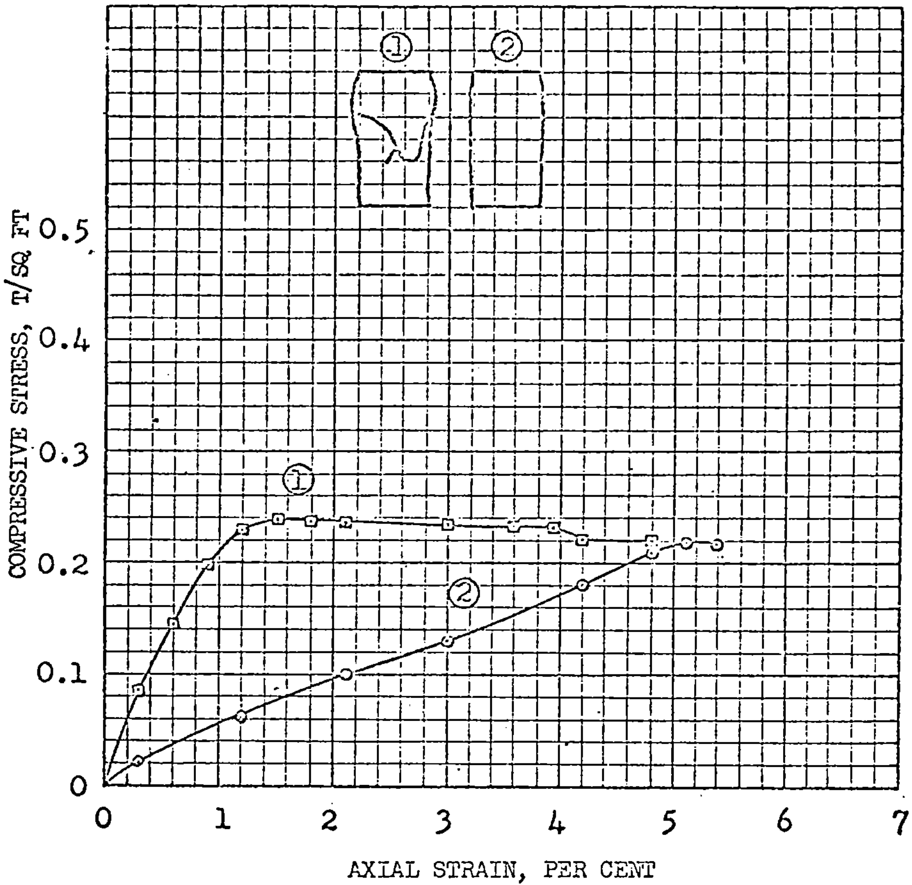
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.37 %/min.
2 0.39 %/min.



Test No.			1	2		
Initial	Water content	w_o	23.5 %	35.4 %	%	%
	Void ratio	e_o	0.638	0.968		
	Saturation	S_o	97.7 %	97.6 %	%	%
	Dry density, lb/cu ft	γ_d	101.1	84.8		
Time to failure, min		t_f	4	13		
Unconfined compressive strength, T/sq ft		q_u	0.24	0.22		
Undrained shear strength, T/sq ft		s_u	0.12	0.11		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.38 in. | Height 1 & 2 3.00 in.

Classification Specimen ①- SILTY SAND (SM), gray, contains several thin horizontal clayey seams. Specimen ②- SILTY CLAY (CL), gray, contains sandy pockets

LL 29* | PL 21* | PI 8*

Remarks ① G_s = 2.65 Est.
② G_s = 2.67 Est.
*conducted on trimmings from specimen 2.

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 5
Depth 8.9 - 11.3 Date 21 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 962
REV. JUL 62

FIG. A1

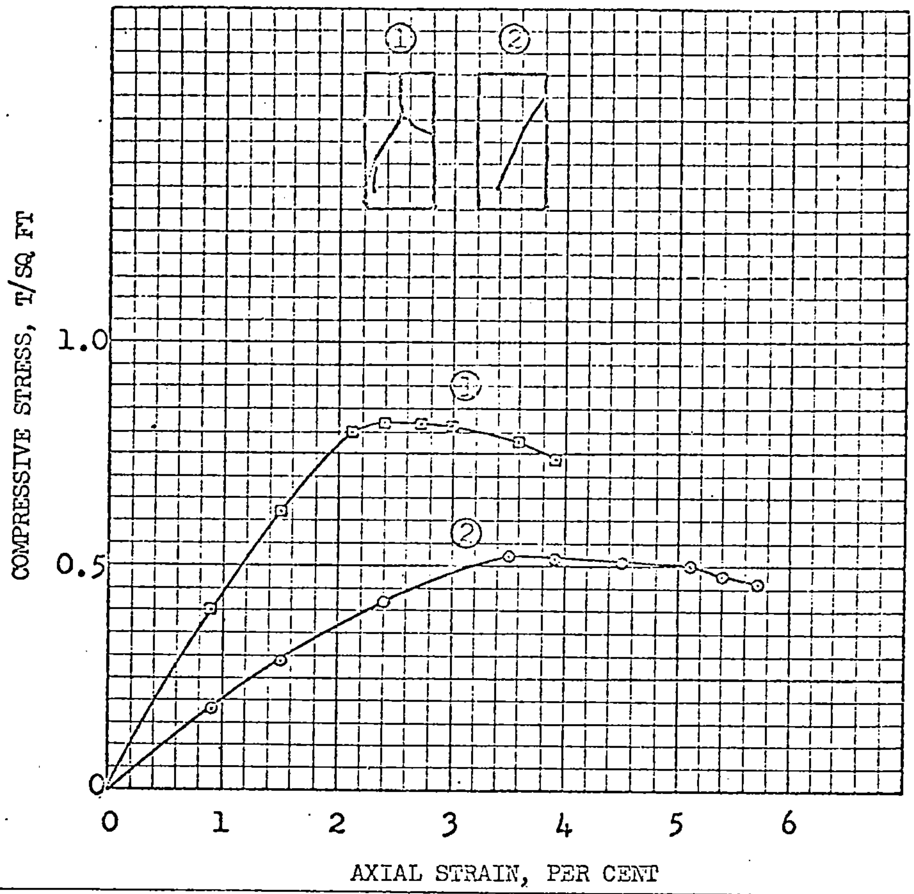
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.39 %/min.
2 0.38 %/min.



Test No.			1	2		
Initial	Water content	w_o	51.3 %	53.3 %	%	%
	Void ratio	e_o	1.38	1.44		
	Saturation	S_o	100 %	99.9 %	%	%
	Dry density, lb/cu ft	γ_d	70.8	69.1		
Time to failure, min		t_f	6	9		
Unconfined compressive strength, T/sq ft		q_u	0.82	0.52		
Undrained shear strength, T/sq ft		s_u	0.41 ([illegible])	0.26 ([illegible])		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.38 in. | Height 1 & 2 3.00 in.

Classification CLAY (CH), green, contains shells and small sandy pockets

LL ——— | PL ——— | PI ———

Remarks G_s = 2.70 From consol test

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 6

Depth 11.8 - 14.2 Date 21 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 962
REV. JUL 62

FIG. A2

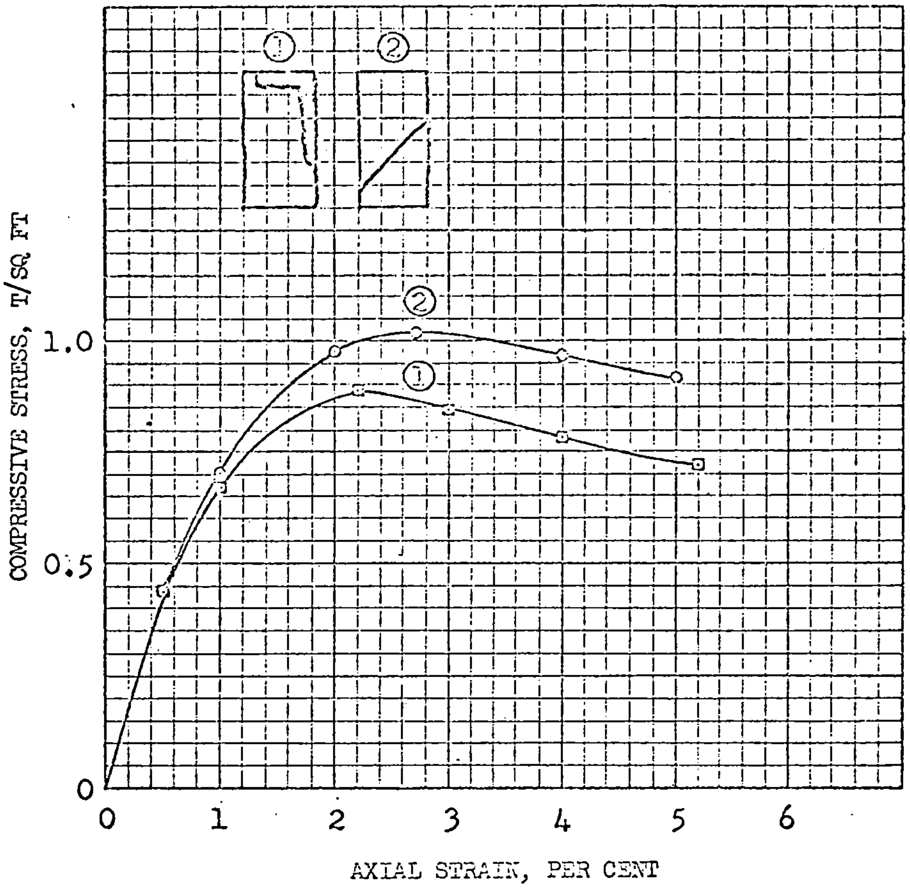
TEST TYPE
(Check one)

☐ Controlled-stress

☑ Controlled-strain

RATE OF STRAIN:

1 0.30 %/min.
2 1.00 %/min.



	Test No.		1	2		
Initial	Water content	w_o	36.1 %	33.6 %	%	%
	Void ratio	e_o	0.772	0.650		
	Saturation	S_o	98.5 %	100 %	%	%
	Dry density, lb/cu ft	γ_d	84.9	87.3		
	Time to failure, min	t_f	7	3		
	Unconfined compressive strength, T/sq ft	q_u	0.89	1.02		
	Undrained shear strength, T/sq ft	s_u	0.44	0.51		
	Sensitivity ratio	S_t				

Type of specimen UNDISTURBED Dia 1 & 2 1.38 in. Height 1 & 2 3.00 in.

Classification CLAY (CL), gray, contains shells and sandy pockets

LL 41 PL 15 PI 26

Remarks Gs = 2.69 Est.

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 7

Depth, 15.1 - 17.5 Date 7 FEB 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 962
REV. JUL 62

FIG. A3

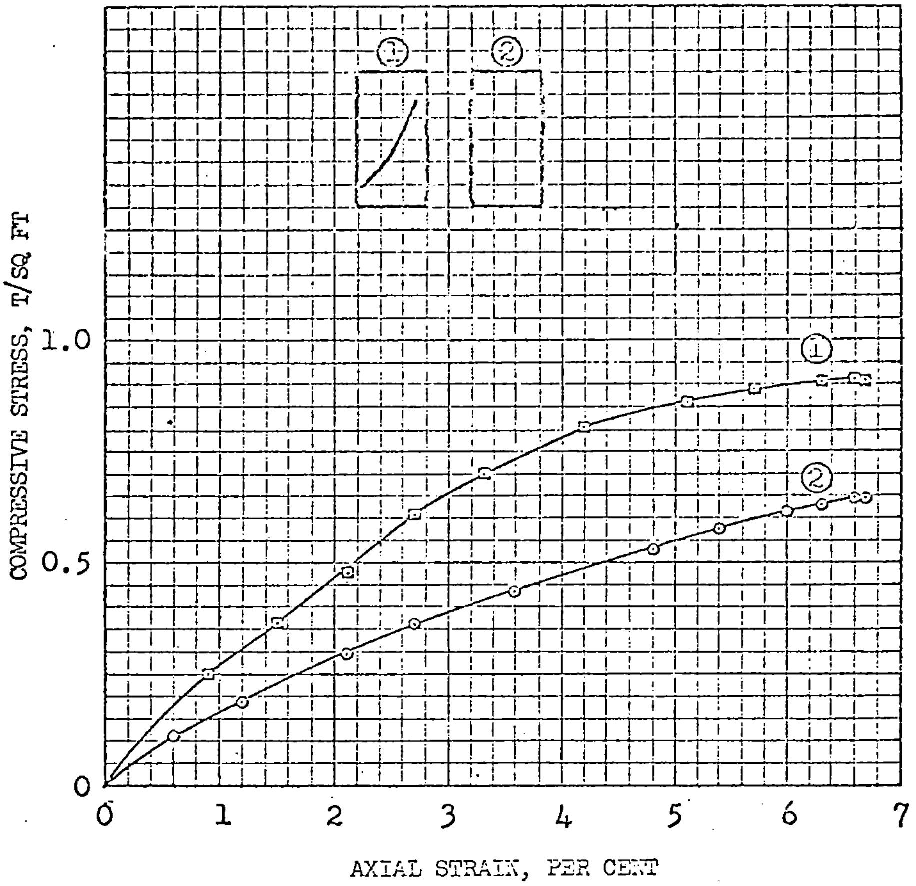
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.39 %/min.
2 0.45 %/min.



Test No.			1	2		
Initial	Water content	w_o	22.4 %	23.9 %	%	%
	Void ratio	e_o	0.588	0.660		
	Saturation	S_o	100 %	96.8 %	%	%
	Dry density, lb/cu ft	γ_d	105.0	100.4		
Time to failure, min		t_f	17	15		
Unconfined compressive strength, T/sq ft		q_u	0.91	0.65		
Undrained shear strength, T/sq ft		s_u	0.45	0.32		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.38 in. | Height 1 & 2 3.00 in.

Classification CLAY (CL), green, contains small sandy pockets

LL 35 | PL 17 | PI 18

Remarks G_s = 2.67 Est.

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 8

Depth 18.1 - 19.2 Date 19 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 962
REV. JUL 62

FIG. A4

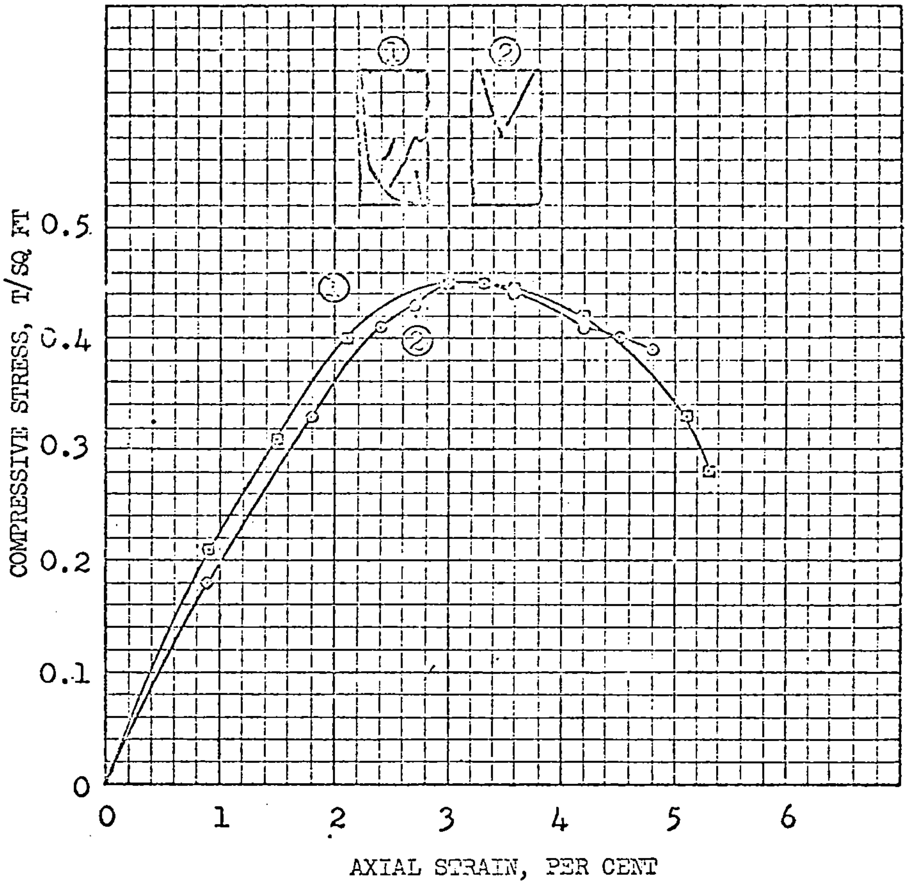
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.41 %/min.
2 0.37 %/min.



Test No.			1	2		
Initial	Water content	w_o	29.8 %	30.3 %	%	%
	Void ratio	e_o	0.810	0.838		
	Saturation	S_o	99.2 %	97.7 %	%	%
	Dry density, lb/cu ft	γ_d	93.1	91.7		
Time to failure, min		t_f	7	8		
Unconfined compressive strength, T/sq ft		q_u	0.45	0.45		
Undrained shear strength, T/sq ft		s_u	0.22	0.22		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.38 in. | Height 1 & 2 3.00 in.

Classification CLAY (CH), green, contains horizontal sandy seams

LL 63 | PL 19 | PI 44

Remarks G_s = 2.70 Est.

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 11

Depth 22.4 - 23.3 Date 19 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

RES FORM NO. 562
REV. JUL 62

FIG. A5

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

SEABROOK LOCK
DESIGN MEMORANDUM NO. 1.GENERAL

APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 2 - UNCONFINED COMPRESSION TEST REPORTS, BORING U-2

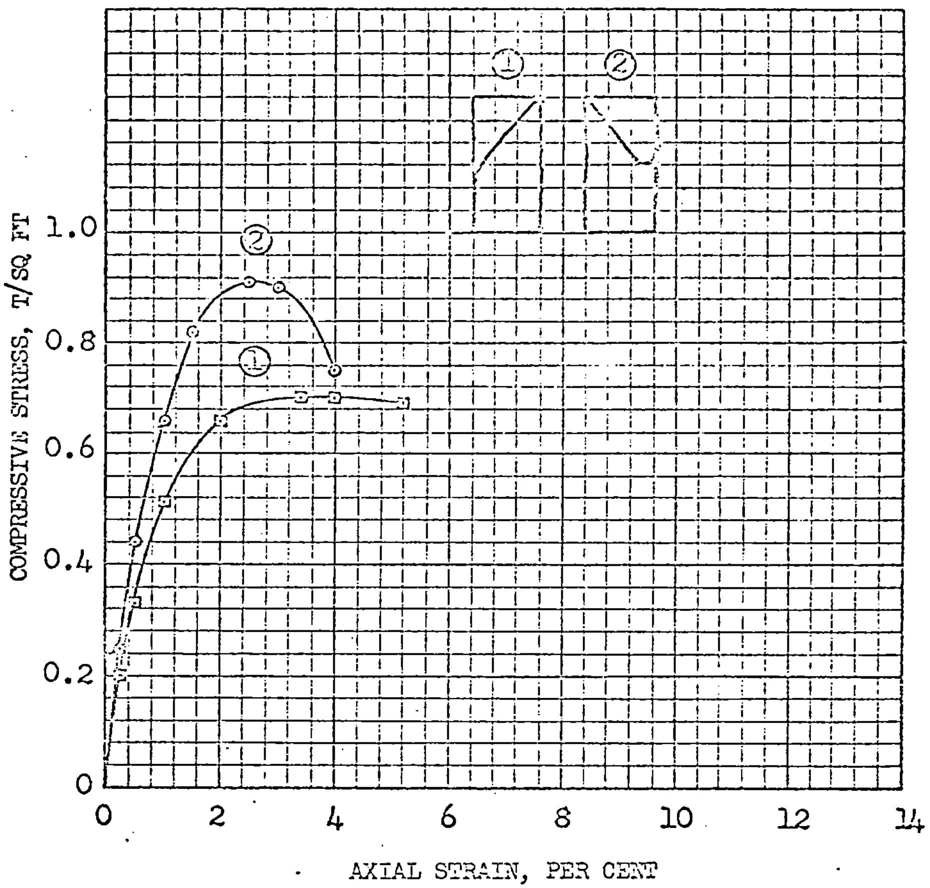
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.29 %/min.
2 0.33 %/min.



Test No.			1	2		
Initial	Water content	w_o	56.5 %	55.8 %	%	%
	Void ratio	e_o	1.58	1.55		
	Saturation	S_o	96.6 %	97.2 %	%	%
	Dry density, lb/cu ft	γ_d	65.2	66.0		
Time to failure, min		t_f	12	7		
Unconfined compressive strength, T/sq ft		q_u	0.70	0.91		
Undrained shear strength, T/sq ft		s_u	0.35	0.45		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED Dia 1 & 2 1.39 in. Height 1 & 2 3.00 in.

Classification SANDY CLAY (CH), gray

LL —— PL —— PI ——

Remarks

Gs = 2.70 From Consol. Test

Project SEABROOK LOCK

Area

Boring No. U-2 Sample No. 6
Depth, 20.5 - 22.9 Date 11 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

ENG FORM NO. 962
REV. JUL 62

FIG. A6

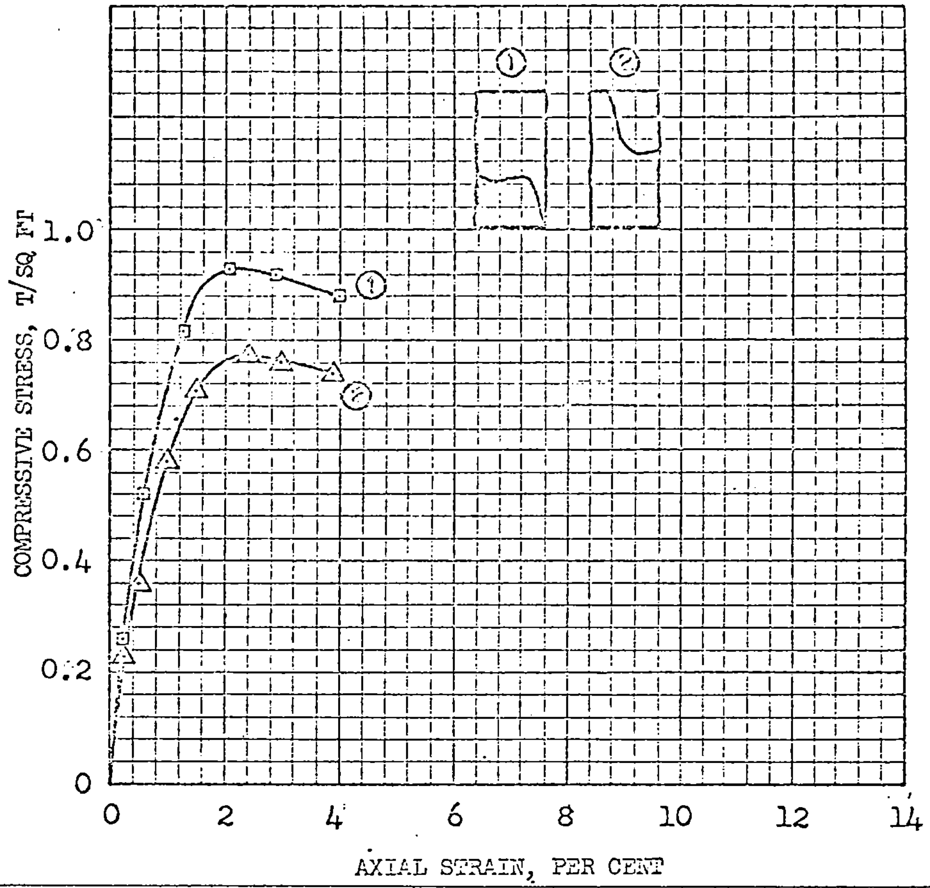
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.31 %/min.
2 0.24 %/min.



Test No.			1	2		
Initial	Water content	w_o	39.0 %	39.6 %	%	%
	Void ratio	e_o	1.06	1.07		
	Saturation	S_o	98.2 %	98.8 %	%	%
	Dry density, lb/cu ft	γ_d	81.0	80.6		
Time to failure, min		t_f	7	10		
Unconfined compressive strength, T/sq ft		q_u	0.93	0.77		
Undrained shear strength, T/sq ft		s_u	0.46 [illegible])	0.38 (7[illegible]psf)		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.38 in. | Height 1 & 2 3.00 in.

Classification SILTY CLAY(CL), gray, contains small sandy pockets

LL ——— | PL ——— | PI ———

Remarks Gs = 2.67 Est.

Project SEABROOK LOCK

Area

Boring No. U-2 Sample No. 7

Depth, 23.5 - 26.0 Date 12 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 562
REV. JUL 62

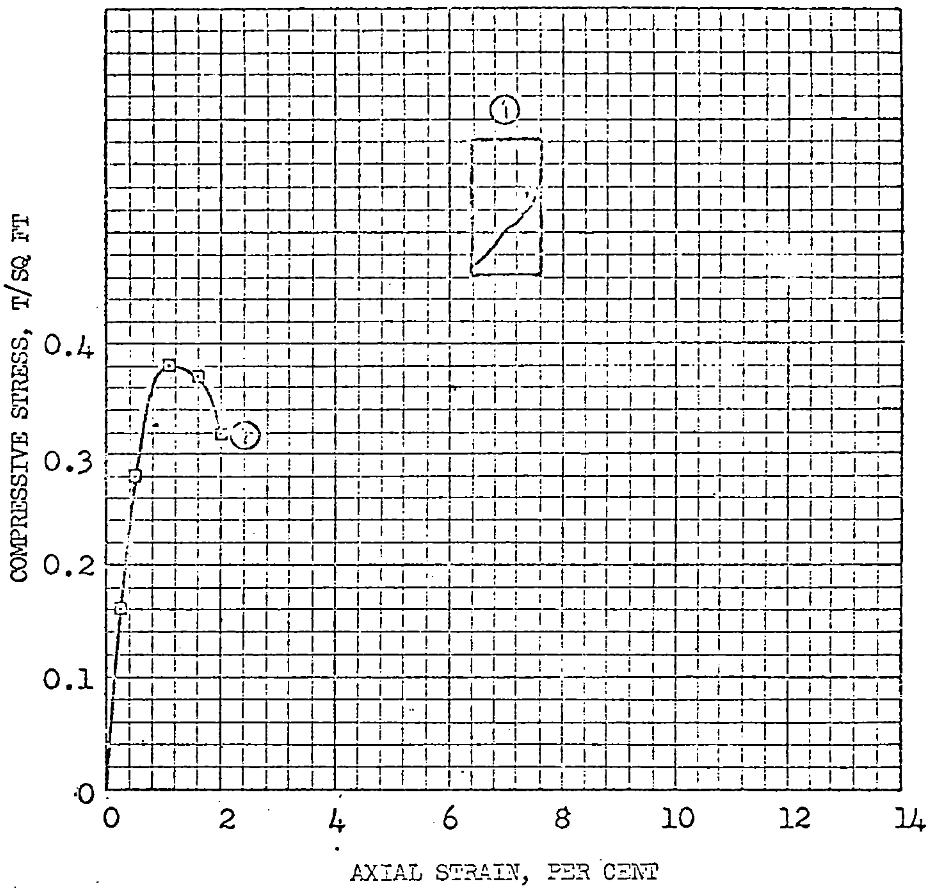
FIG. A7

TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN
1 0.22 %/min.



Test No.			1			
Initial	Water content	w_o	23.6 %	%	%	%
Initial	Void ratio	e_o	0.661			
Initial	Saturation	S_o	95.7 %	%	%	%
Initial	Dry density, lb/cu ft	γ_d	100.7			
Time to failure, min		t_f	5			
Unconfined compressive strength, T/sq ft		q_u	0.38			
Undrained shear strength, T/sq ft		s_u	0.19			
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED Dia 1.38 in. Height 3.00 in.

Classification Conglomerate of CLAY(CH), CLAYEY SAND(SC), and SILTY SAND(SM), gray

LL —— PL —— PI ——

Remarks Gs = 2.68 From R Test

Project SEABROOK LOCK

Area

Boring No. U-2 Sample No. 8

Depth, 26.5 - 28.9 Date 12 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

FIG A8

WES FORM NO. 962
REV. JUL 62

FIG. A8

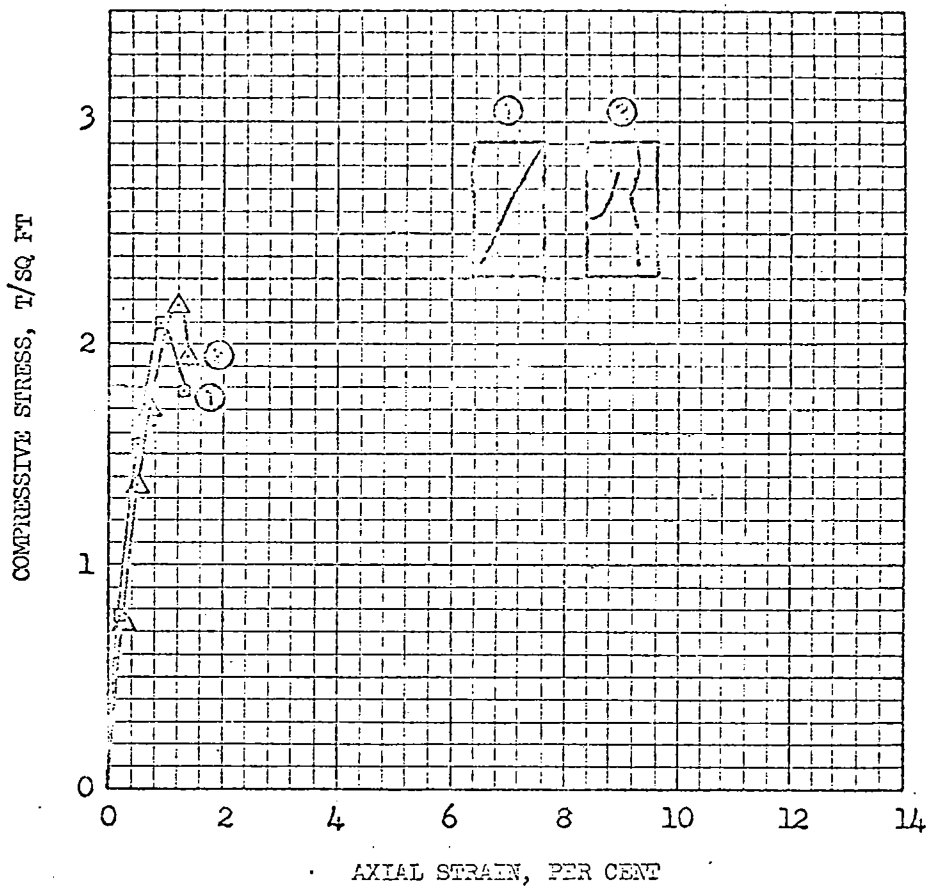
TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

RATE OF STRAIN:

1 0.26 %/min.
2 0.20 %/min.



Test No.			1	2		
Initial	Water content	w_o	25.7 %	25.4 %	%	%
	Void ratio	e_o	0.704	0.699		
	Saturation	S_o	97.5 %	97.0 %	%	%
	Dry density, lb/cu ft	γ_d	97.8	98.1		
Time to failure, min		t_f	4	6		
Unconfined compressive strength, T/sq ft		q_u	2.09	2.18		
Undrained shear strength, T/sq ft		s_u	1.04	1.09		
Sensitivity ratio		S_t				

Type of specimen UNDISTURBED | Dia 1 & 2 1.39 in. | Height 1 & 2 3.00 in.

Classification SILTY CLAY (CL), gray

LL —— PL —— PI ——

Remarks Gs = 2.67 Est.

Project SEABROOK LOCK

Area

Boring No. U-2 Sample No. 33

Depth, 133.0-135.4 Date 12 JAN 1966

RTS UNCONFINED COMPRESSION TEST REPORT

WES FORM NO. 952
REV. JUL 62

FIG. A9

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

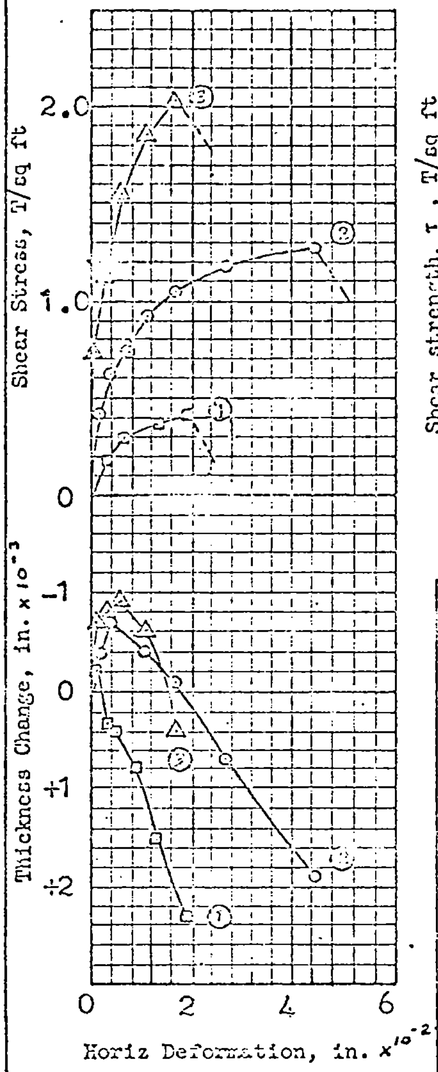
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

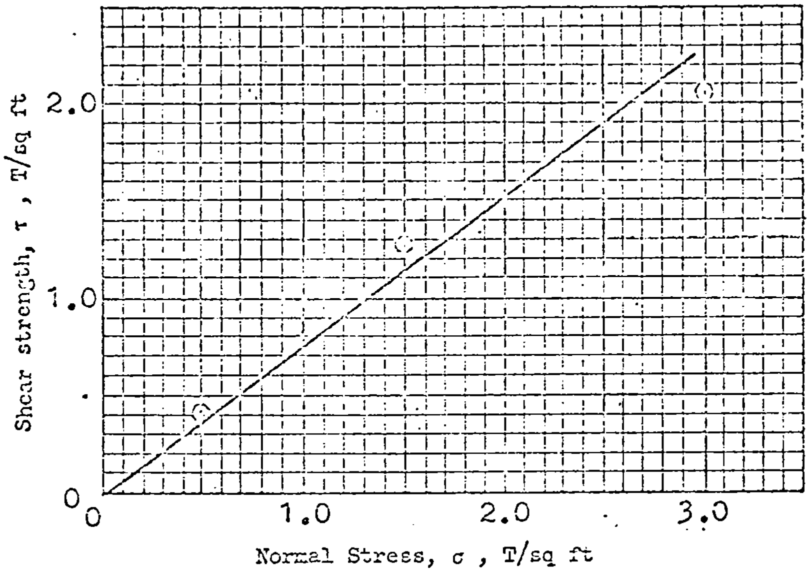
APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 3 - DIRECT SHEAR TEST REPORT, BORING U-1





Test No.			1	2	3	
Initial	Water content	w_o	24.9 %	24.4 %	24.8 %	%
	Void ratio	e_o	0.737	0.741	0.726	
	Saturation	S_o	89.9 %	87.6 %	90.9 %	%
	Dry density lb/cu ft	γ_d	95.6	95.4	96.2	
Void ratio after consolidation		e_c				
Time for 50% consolidation, min		t_{50}	<1	<1	<1	
Final	Water content	w_f	26.6 %	27.5 %	26.1 %	%
	Void ratio	e_f				
	Saturation	S_f	%	%	%	%
Actual time to failure, min.		t_f	420	420	420	
Normal stress T/sq ft		σ	0.50	1.50	3.00	
Maximum shear strength, T/sq ft		τ	0.41	1.27	2.05	

Shear Values

ϕ' = 37°

tan ϕ' = 0.840

c' = 0

Test Type (Check One) S ☒ Controlled, stress ☐ Controlled, strain

Type of Specimen Undisturbed 3.00 in. Square 0.502 in. Thickness

Classification SAND (SP), gray, contains shell fragments

LL ——— PL ——— PI ——— G_s = 2.66 Est.

Remarks

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 2

Depth 4.3-5.2 Date 24 MAR. '66

JMS DIRECT SHEAR TEST REPORT

ENG FORM 1 MAY 63 2092 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

c 3437

FIG. A10

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

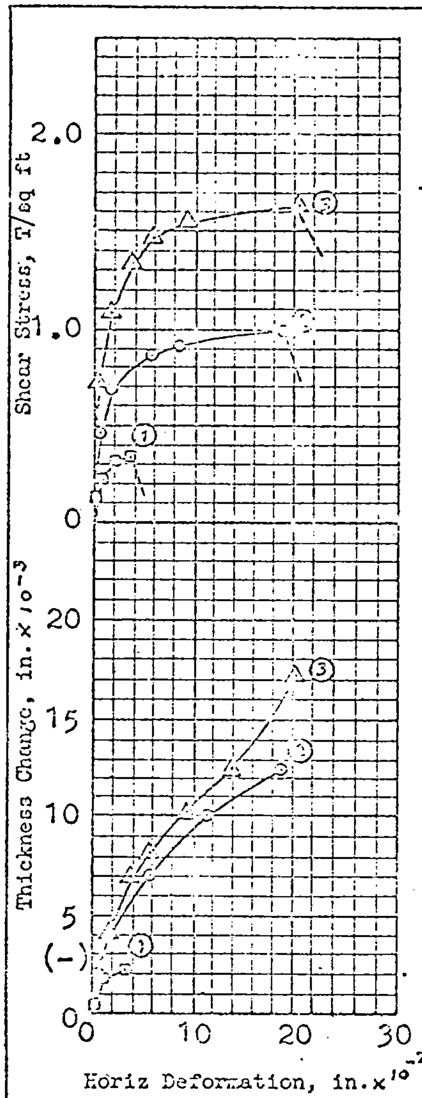
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

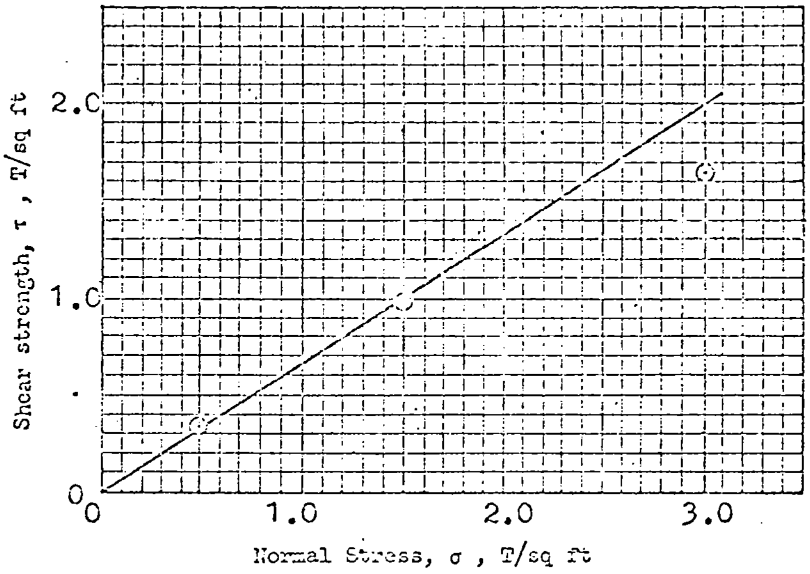
APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 4 - DIRECT SHEAR TEST REPORT, BORING U-2





Test No.			1	2	3	
Initial	Water content	w_o	28.5 %	28.3 %	29.2 %	%
	Void ratio	e_o	0.845	0.811	0.853	
	Saturation	S_o	89.7 %	82.1 %	91.1 %	%
	Dry density lb/cu ft	γ_d	90.0	91.7	89.6	
Void ratio after consolidation		e_c				
Time for 50% consolidation, min		t_{50}	<1	<1	<1	
Final	Water content	w_f	32.2 %	30.3 %	29.1 %	%
	Void ratio	e_f				
	Saturation	S_f	%	%	%	%
Actual time to failure, min.		t_f	340	420	400	
Normal stress T/sq ft		σ	0.50	1.50	3.00	
Maximum shear strength, T/sq ft		τ	0.34	0.99	1.62	

Shear Values

φ' = 34

tan φ' = 0.674

c' = 0

Test Type (Check One) S ☒ Controlled, stress ☐ Controlled, strain

Type of Specimen Undisturbed 3.00 in. Square 0.502 in. Thickness

Classification SILTY SAND (SM), gray, contains shell fragments

LL ——— PL ——— PI ——— Gs = 2.66 From R Test

Remarks

Project SEABROOK LOCK

Area

Boring No. U-2 Sample No. 5

Depth 18.6-19.5 Date 22 MAR. '66

JMS DIRECT SHEAR TEST REPORT

ENG FORM 1 MAY 63 2092 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

C 3437

FIG. A11

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

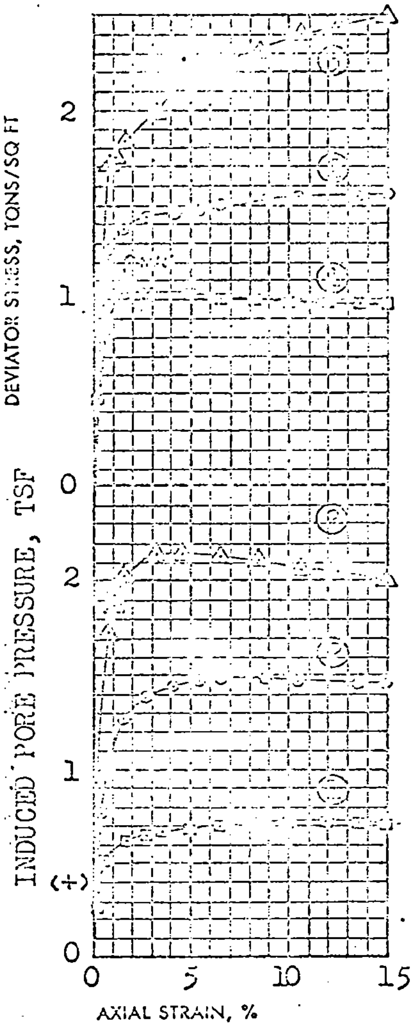
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

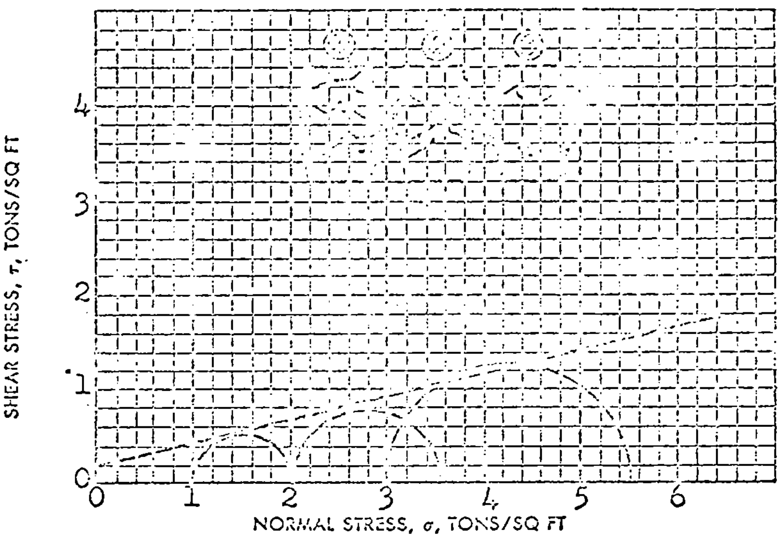
APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 5 - TRIAXIAL COMPRESSION TEST REPORT, BORING U-2





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	34.9 %	33.9 %	36.0 %	%
	VOID RATIO	e_o	0.929	0.887	0.960	
	SATURATION	S_o	99.9 %	100 %	99.8 %	%
	DRY DENSITY LB/CU FT	γ_d	86.1	88.0	84.7	
BEFORE TEST	WATER CONTENT		32.8 %	29.2 %	30.6 %	%
	SATURATION	S_c	97.9* %	95.8* %	96.6* %	%
	CONSOLIDATION PRESS., TONS/SQ FT	σ_c	1.0	2.0	3.0	
	VOID RATIO	e_c	0.891	0.811	0.843	
	Dry density lb/cu ft	γ_d	87.8	91.7	90.1	
	VOID RATIO	e_f				
MAJOR PRINCIPAL STRESS, TONS/SQ FT		σ_1	2.03	3.56	5.51	
MINOR PRINCIPAL STRESS, TONS/SQ FT		σ_3	1.0	2.0	3.0	
TIME TO FAILURE, MIN			8	70	78	
INITIAL DIAMETER, in.			1.39	1.38	1.38	
INITIAL HEIGHT, H_o, in.			3.00	3.00	3.00	
Back pressure, psi			70	60	70	

SHEAR VALUES+

φ = 14°

TAN φ = 0.250

c = 0.20 TONS/SQ FT

+See attached plate for effective values.

TYPE TEST R

METHOD OF SATURATION BP

☐ CONTROLLED STRESS ☑ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED	RATE OF STRAIN	0.20	0.21	0.19	%/MIN

CLASSIFICATION SILTY SAND(SM), gray, contains shell fragments and clayey seams

LL — PL — PI — Gs = 2.64

REMARKS

*Pore pressure response indicated 100% saturation

PROJECT SEABROOK LOCK

AREA

BORING NO. U-2 SAMPLE NO. 5

DEPTH 18.6-19.5 DATE 17 JAN 1966

FH TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)
1 MAY 63

GPO : 1964 OF—714—729

FIG. A12

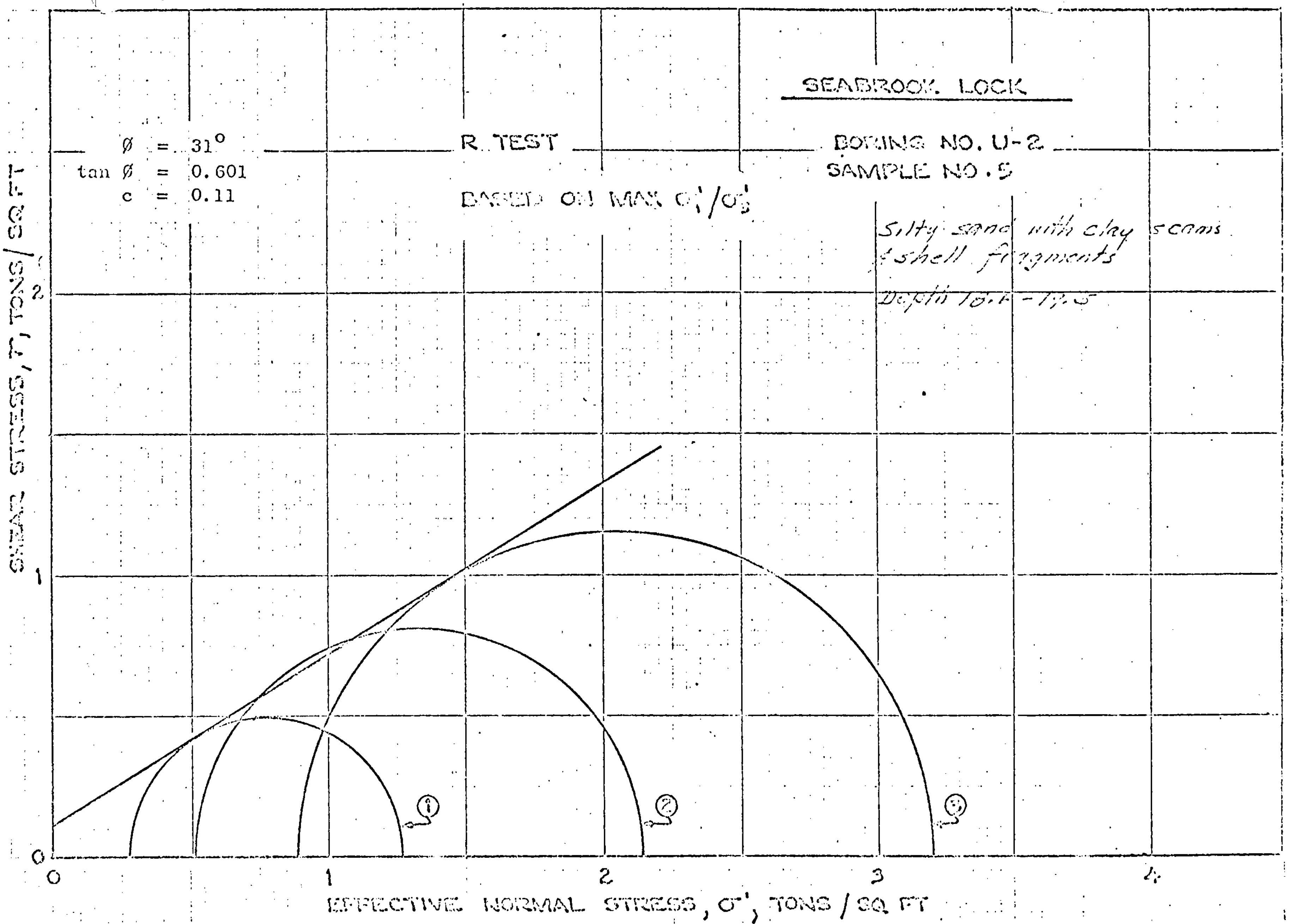
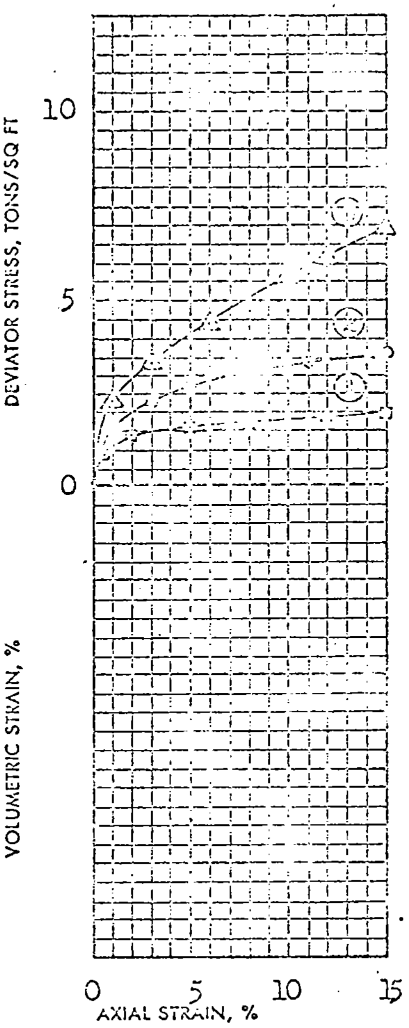
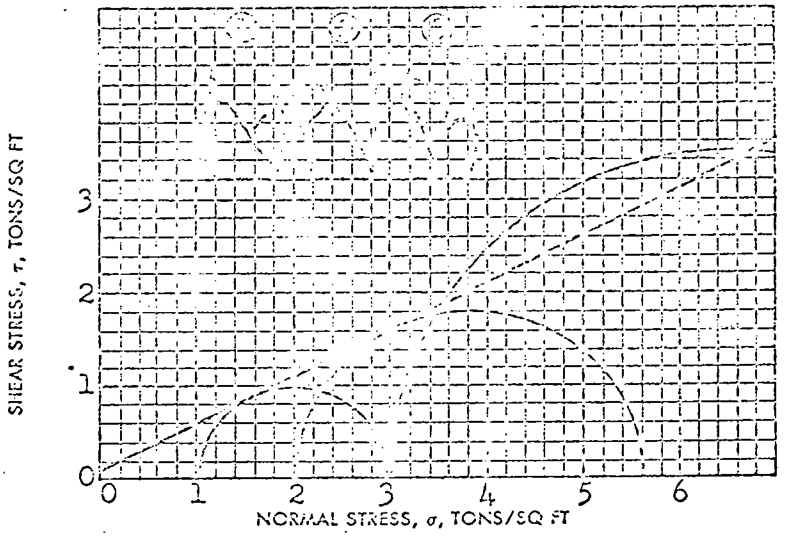

SEABROOK LOCK
BORING NO. U-2
SAMPLE NO. 5
Silty sand with clay seams & shell fragments
Depth 10.6 - 11.5
R TEST
BASED ON MAX σ_1'/σ_3'
ϕ = 31°
tan ϕ = 0.601
c = 0.11
①
②
③
0
1
2
3
4
0
1
2
EFFECTIVE NORMAL STRESS, σ', TONS / SQ FT
SHEAR STRESS, τ, TONS / SQ FT

FIG. A13





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	19.8 %	19.9 %	17.2%	%
	VOID RATIO	e_o	0.629	0.575	0.542	
	SATURATION	S_o	84.4 %	92.8 %	85.0%	%
	DRY DENSITY LB/CU FT	γ_d	102.7	106.2	108.5	
BEFORE TEST	WATER CONTENT	w_c	21.5 %	20.0 %	19.0%	%
	SATURATION	S_c	96.4* %	100+ %	100 %	%
	CONSOLIDATION PRESS., TONS/SQ FT	σ_c	1.0	2.0	3.0	
	VOID RATIO	e_c	0.598	0.513	0.479	
	Dry density lb/cu ft	γ_d	104.7	110.6	113.1	
	VOID RATIO	e_f				
MAJOR PRINCIPAL STRESS, TONS/SQ FT		σ_1	2.99	5.62	9.96	
MINOR PRINCIPAL STRESS, TONS/SQ FT		σ_3	1.0	2.0	3.0	
TIME TO FAILURE, MIN			93	104	95	
INITIAL DIAMETER, in.			1.39	1.40	1.40	
INITIAL HEIGHT, H_o, in.			3.00	3.00	3.00	
Back pressure, psi			70	50	80	

SHEAR VALUES

ϕ = 27°

TAN ϕ = 0.510

c = 0.10 TONS/SQ FT

TYPE TEST R

METHOD OF SATURATION BP

☐ CONTROLLED STRESS ☐ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	RATE OF STRAIN	0.16	0.14	0.16	%/MIN

CLASSIFICATION Conglomerate of CLAY(CH), CLAYEY SAND(SC), and SILTY SAND(SM), gray

LL 40	PL 15	PI 25	Gs = 2.68

REMARKS

*Pore pressure response indicated 100% saturation.

PROJECT SEABROOK LOCK

AREA

BORING NO. U-2

SAMPLE NO. 8

DEPTH 24.5 - 28.0

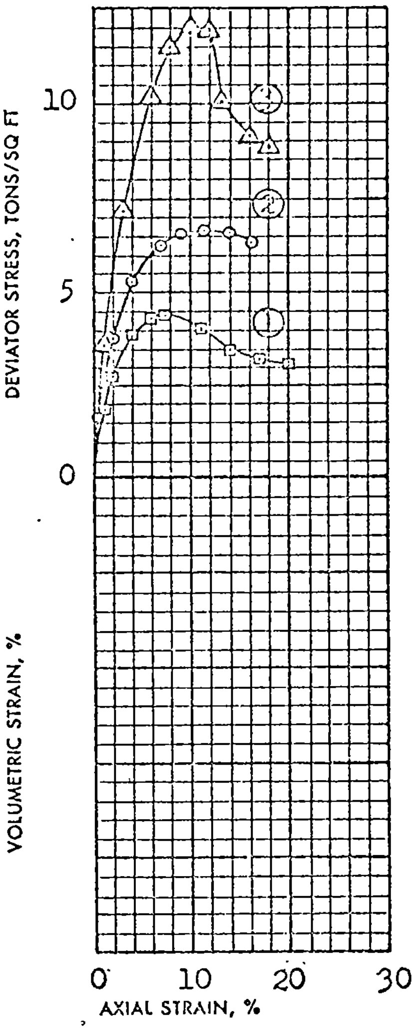
DATE 22 JAN 1966

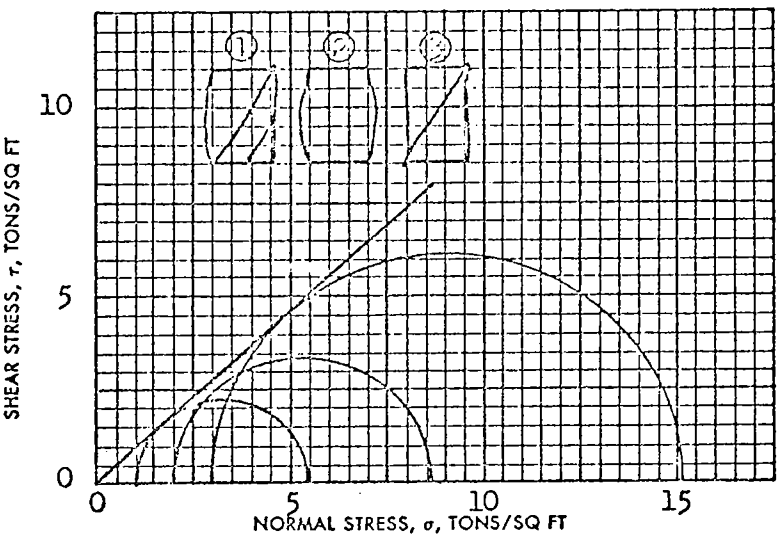
FH TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 MAY 63 2089 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO : 1964 OF—714-729

FIG. A14





SHEAR VALUES

φ = 42°

TAN φ = 0.900

c = 0 TONS/SQ FT

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	28.6 %	32.0 %	28.6%	%
	VOID RATIO	e_o	0.793	0.881	0.786	
	SATURATION	S_o	95.9 %	96.6 %	96.8%	%
	DRY DENSITY LB/CU FT	γ_d	92.6	88.3	93.0	
BEFORE TEST	WATER CONTENT	w_c	29.0 %	31.4 %	26.5%	%
	SATURATION	S_c	100+ %	100+ %	98.7%	%
	CONSOLIDATION PRESS., TONS/SQ FT	σ_c	1.0	2.0	3.0	
	VOID RATIO	e_c	0.728	0.813	0.714	
	Dry density lb/cu ft	γ_d	96.1	91.6	96.9	
	VOID RATIO	e_f				
MAJOR PRINCIPAL STRESS, TONS/SQ FT		σ_1	5.43	8.65	15.06	
MINOR PRINCIPAL STRESS, TONS/SQ FT		σ_3	1.0	2.0	3.0	
TIME TO FAILURE, MIN			20	34	25	
INITIAL DIAMETER, in.			1.40	1.40	1.39	
INITIAL HEIGHT, H_o, in.			3.00	3.00	3.00	

TYPE TEST S

METHOD OF SATURATION ☐ CONTROLLED STRESS ☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	RATE OF STRAIN	0.37	0.34	0.40	%/MIN

CLASSIFICATION SILT (ML), brown, contains a few thin horizontal sandy layers

LL 27 PL 26 PI 1 Gs = 2.66 Est.

REMARKS

PROJECT SEABROOK LOCK

AREA

BORING NO. U-2 SAMPLE NO. 3A

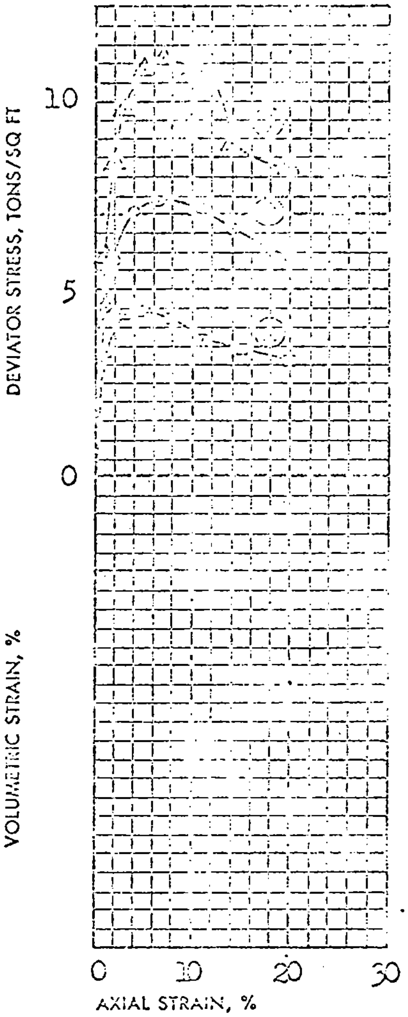
DEPTH 7.0 - 9.4 DATE 18 JAN 1966

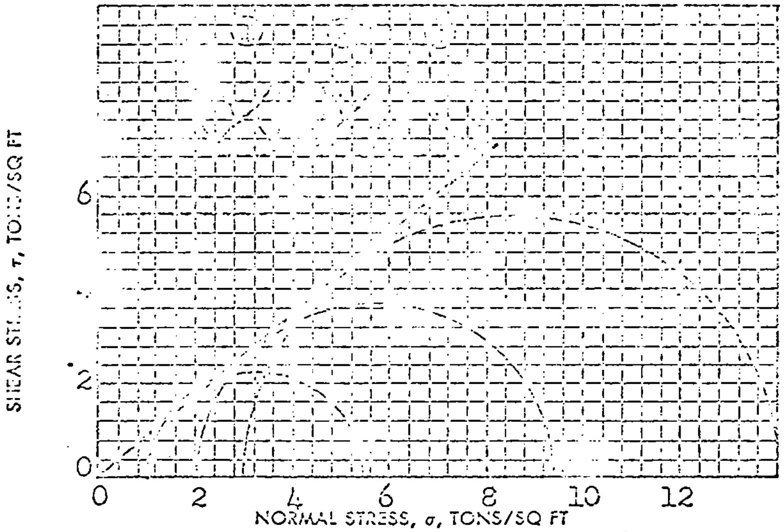
FH TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 MAY 63 2089 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO : 1964 OF—714-729

Revised August 19 FIG. A15





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	25.3 %	26.0 %	25.5%	%
	VOID RATIO	e_o	0.728	0.733	0.719	
	SATURATION	S_o	92.4 %	94.4 %	94.3%	%
	DRY DENSITY LB/CU FT	γ_d	96.1	95.8	96.6	
BEFORE TEST	WATER CONTENT	w_c	26.8 %	26.8 %	25.3%	%
	SATURATION	S_c	100+ %	100 %	98.1%	%
	CONSOLIDATION PRESS., TONS/SQ FT	σ_c	1.0	2.0	3.0	
	VOID RATIO	e_c	0.694	0.707	0.686	
	Dry density lb/cu ft	γ_d	98.0	97.3	98.5	
	VOID RATIO	e_f				
MAJOR PRINCIPAL STRESS, TONS/SQ FT		σ_1	5.49	9.40	14.12	
MINOR PRINCIPAL STRESS, TONS/SQ FT		σ_3	1.0	2.0	3.0	
TIME TO FAILURE, MIN			11	17	15	
INITIAL DIAMETER, in.			1.39	1.39	1.39	
INITIAL HEIGHT, H_o, in.			3.00	3.00	3.00	

SHEAR VALUES

φ = 41°

TAN φ = 0.870

c = 0 TONS/SQ FT

TYPE TEST S

METHOD OF SATURATION ______ ☐ CONTROLLED STRESS ☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	RATE OF STRAIN	0.42	0.38	0.48	%/MIN

CLASSIFICATION SAND (SP) gray

LL —— PL —— PI —— Gs= 2.66 Est.

REMARKS

PROJECT SEABROOK LOCK

AREA

BORING NO. U-2 SAMPLE NO. 16

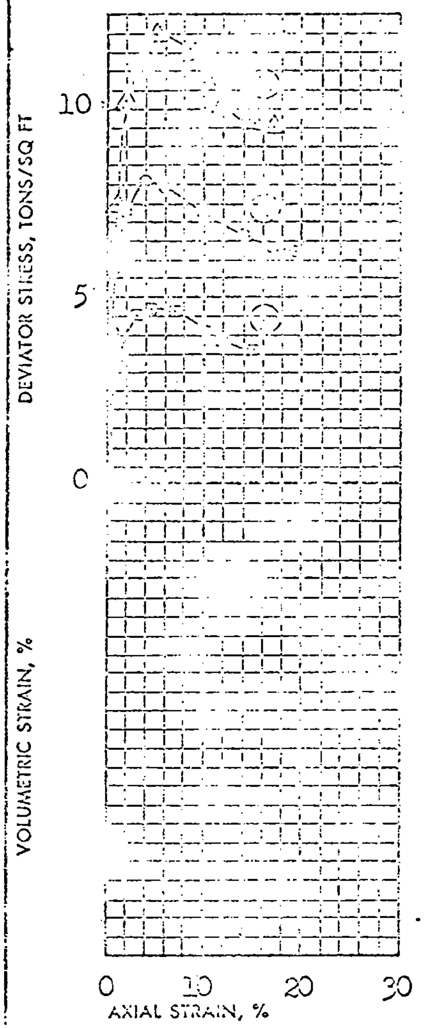
DEPTH 50.0 - 52.5 DATE 20 JAN 1966

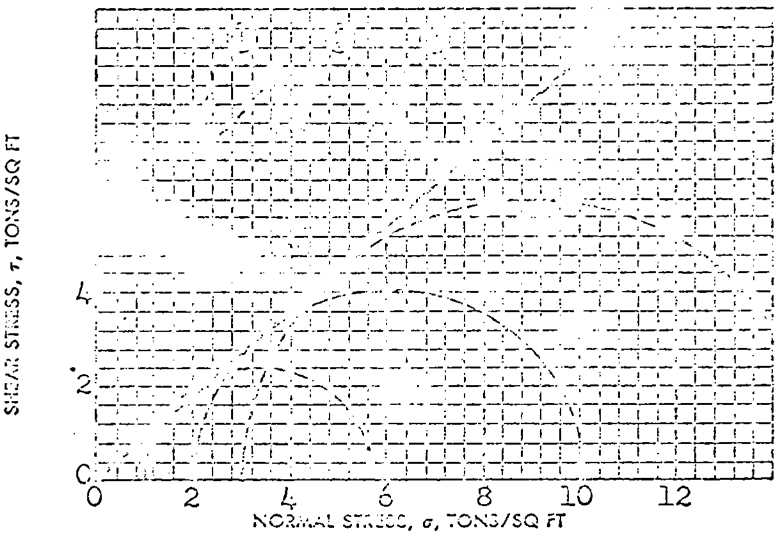
FH TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)
1 MAY 63

GPO : 1964 OF—714-729

FIG. A16





SHEAR VALUES

φ = 42 °

TAN φ = 0.900

c = 0 TONS/SQ FT

			1	2	3	
INITIAL	WATER CONTENT	w_o	22.9 %	22.8 %	22.8%	%
	VOID RATIO	e_o	0.656	0.666	0.666	
	SATURATION	S_o	92.8 %	91.1 %	91.1%	%
	DRY DENSITY LB/CU FT	γ_d	100.3	99.7	99.7	
BEFORE TEST	WATER CONTENT	w_c	26.6 %	26.4 %	26.8%	%
	SATURATION	S_c	100+ %	100+ %	100+%	%
	CONSOLIDATION PRESS., TONS/SQ FT	σ_c	1.0	2.0	3.0	
	VOID RATIO	e_c	0.652	0.639	0.633	
	Dry density lb/cu ft	γ_d	100.5	101.3	101.7	
	VOID RATIO	e_f				
MAJOR PRINCIPAL STRESS, TONS/SQ FT		σ_1	5.69	10.10	15.00	
MINOR PRINCIPAL STRESS, TONS/SQ FT		σ_3	1.0	2.0	3.0	
TIME TO FAILURE, MIN			15	11	14	
INITIAL DIAMETER, in.			1.39	1.40	1.41	
INITIAL HEIGHT, H_o, in.			3.00	3.00	3.00	

TYPE TEST S

METHOD OF SATURATION ______ ☐ CONTROLLED STRESS ☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	RATE OF STRAIN	0.30	0.35	0.36	%/MIN

CLASSIFICATION SAND (SP), brown

LL — PL — PI — Gs = 2.66 Est.

REMARKS ______

PROJECT SEABROOK LOCK

AREA

BORING NO. U-2 SAMPLE NO. 23

DEPTH 83.0 - 85.5 DATE 22 JAN 1966

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 MAY 63 2089 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO : 1964 OF—714-729

FIG. A17

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

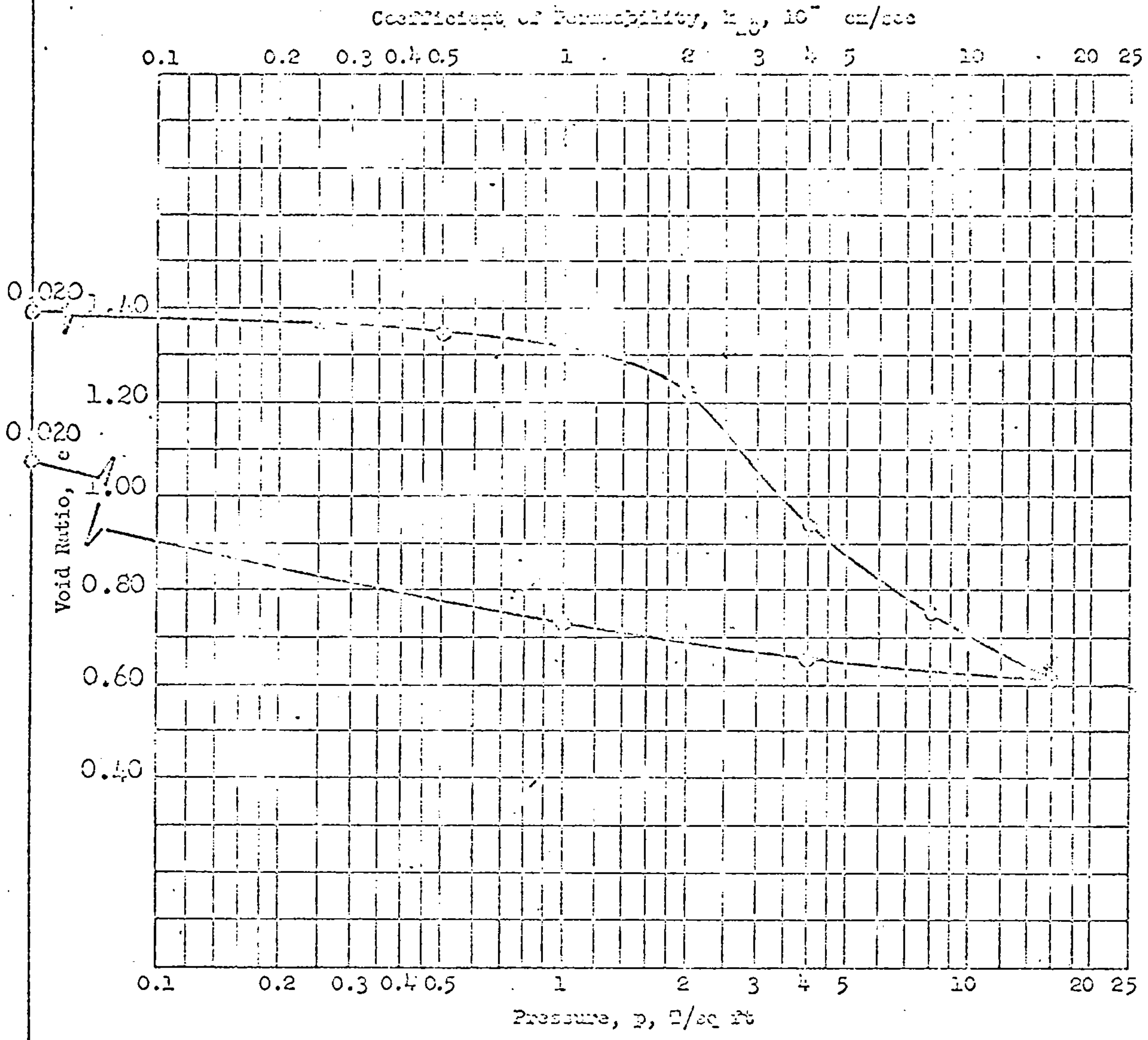
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 6 - CONSOLIDATION TEST REPORTS AND TIME CURVES, BORING U-1



Type of Specimen UNDISTURBED

Diam 2.50 in. Ht 0.752 in.

Overburden Pressure, p_o 0.34 T/sq ft

Precónsol. Pressure, p_c 2.0 T/sq ft

Compression Index, C_c

Classification CLAY (CH), green, *

LL 54 G_s = 2.70

PL 18 D_{10}

Remarks * contains shells and small sandy pockets

	Before Test		After Test
Water Content, w_o	50.6 %	w_f	38.7 %
Void Ratio, e_o	1.39	e_f	1.07
Saturation, S_o	98.4 %	S_f	97.8 %
Dry Density, γ_d	70.8 lb/ft^3		81.8

k_{20} at e_o = × 10^{-} cm/sec

Project SEABROOK LOCK

Area

Boring No. U-1 Sample No. 6

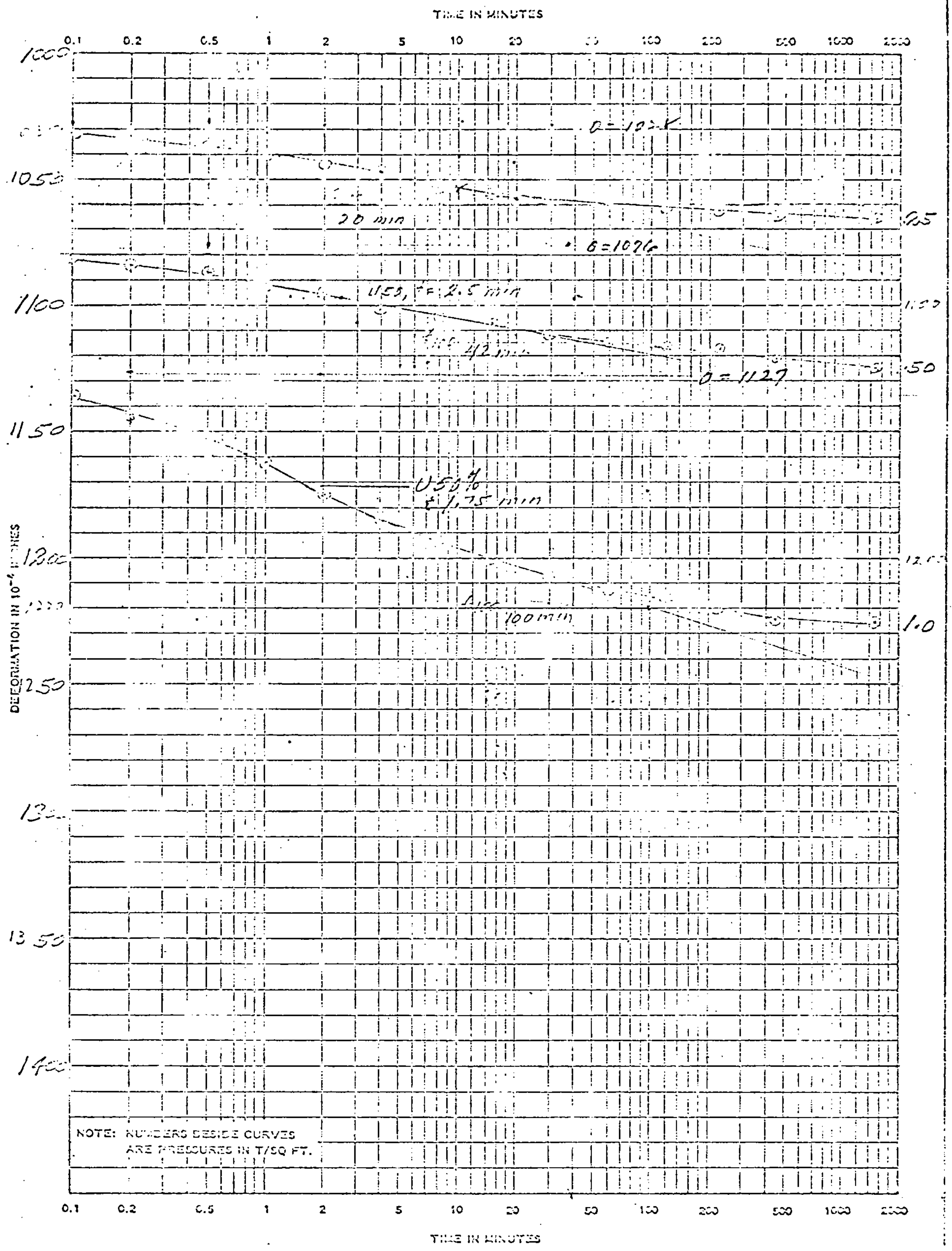
Depth 26 - 42 11.8-14.2 Date 27-Jan-66

JS CONSOLIDATION TEST REPORT

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

0 3424

FIG. A18



Project SEABROOK LOCK

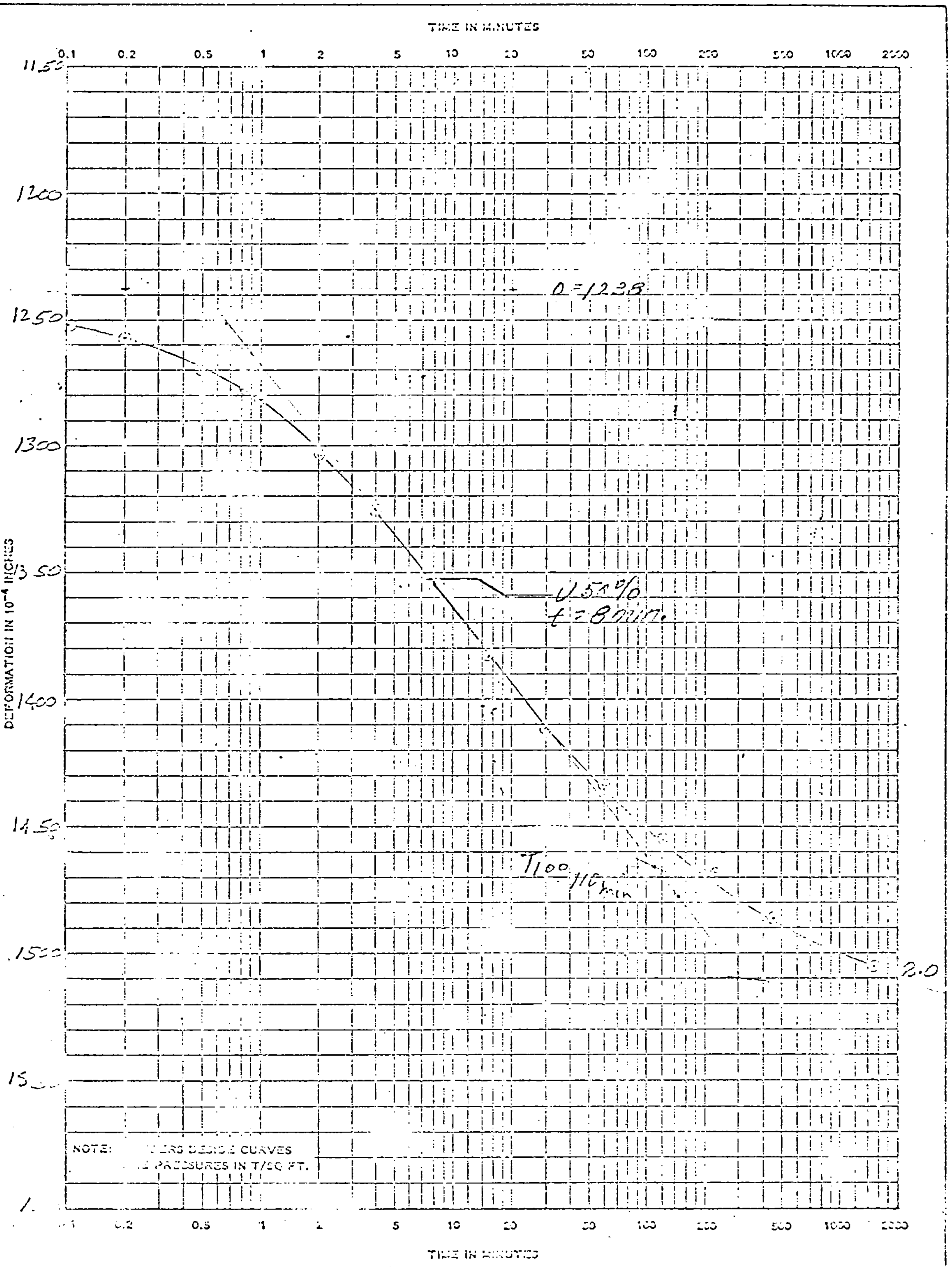
Area El. -42.0±

Boring No. U-1 Sample No. 6 Depth 11.8-14.2 Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST–TIME CURVES (TRANSLUCENT)

FIG. A18a



Project S[illegible]BROOK LOCK

Area

Boring No. U-1

Sample No. 6

Depth 11.8-12.2

Date FEB 66

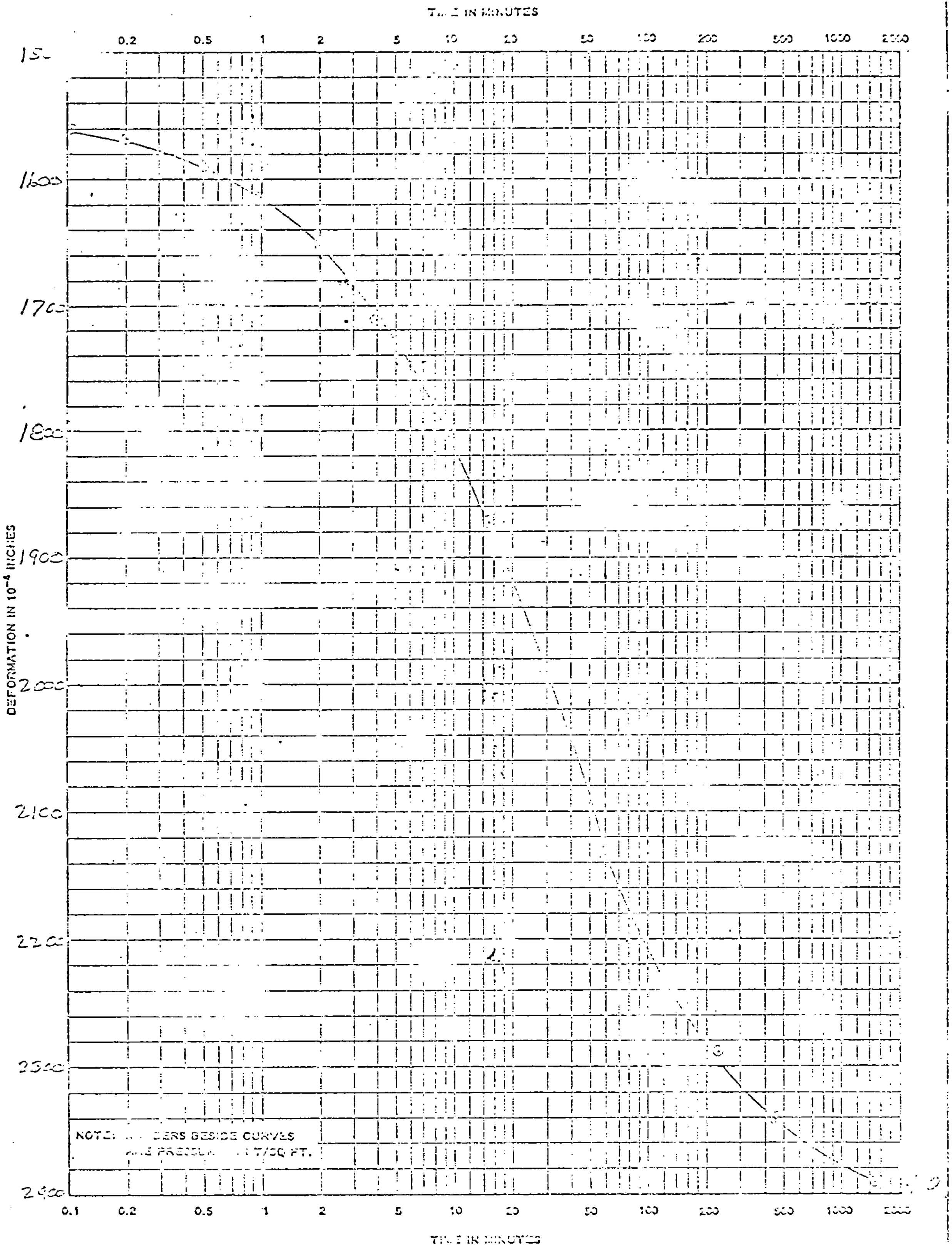
ENG FORM 2000
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

C 3/27

FIG. A15b



Project SEABROOK LOCK

Area

Boring No. U-1

Sample No. 6

Depth 11.8-14.2

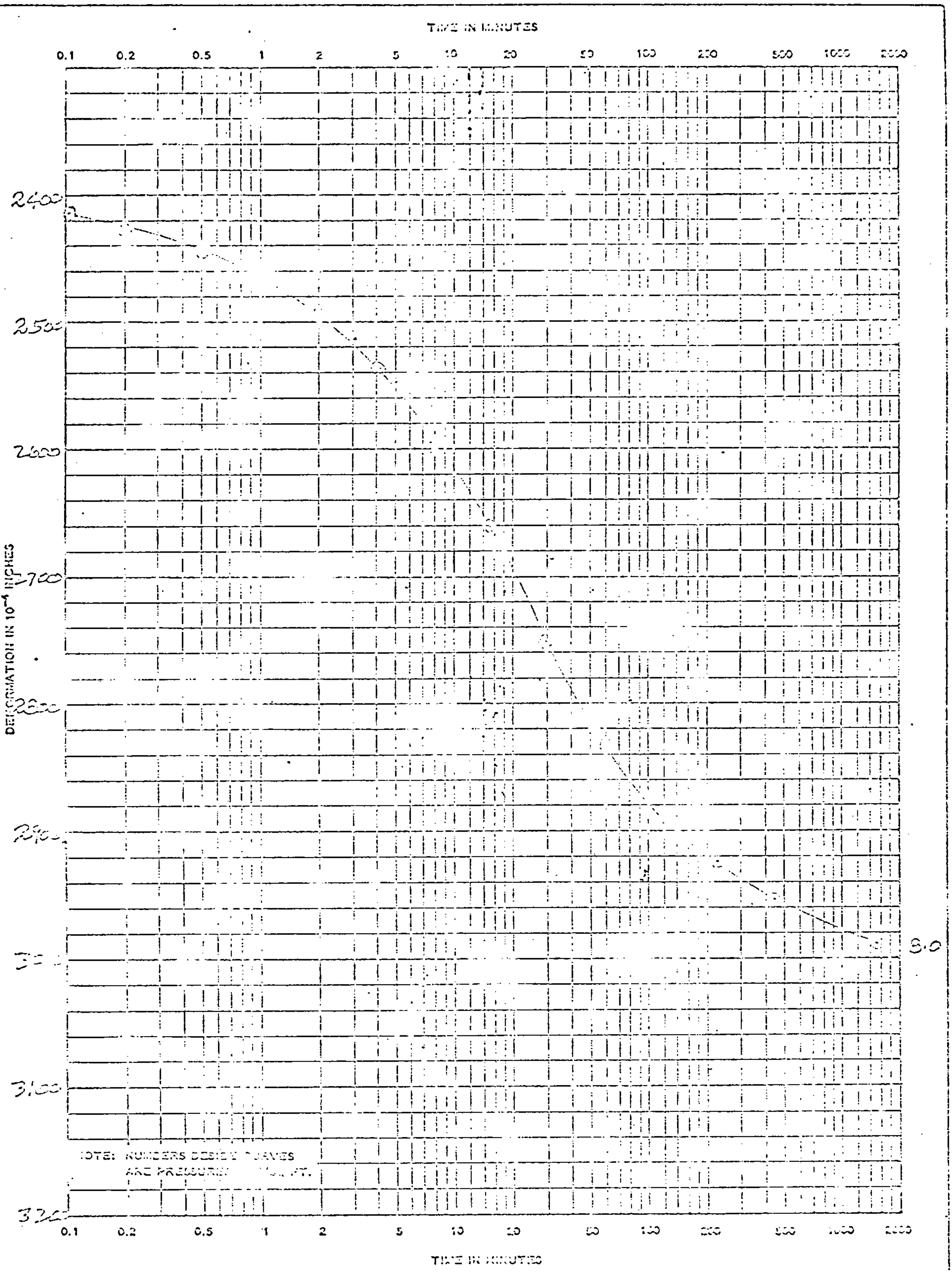
Date FEB 66

ENG FORM 2008
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

FIG. A18c



Project SEABROOK LOCK

Area

Boring No. U-1	Sample No. 6	Depth 11.8-14.2	Date FEB 66

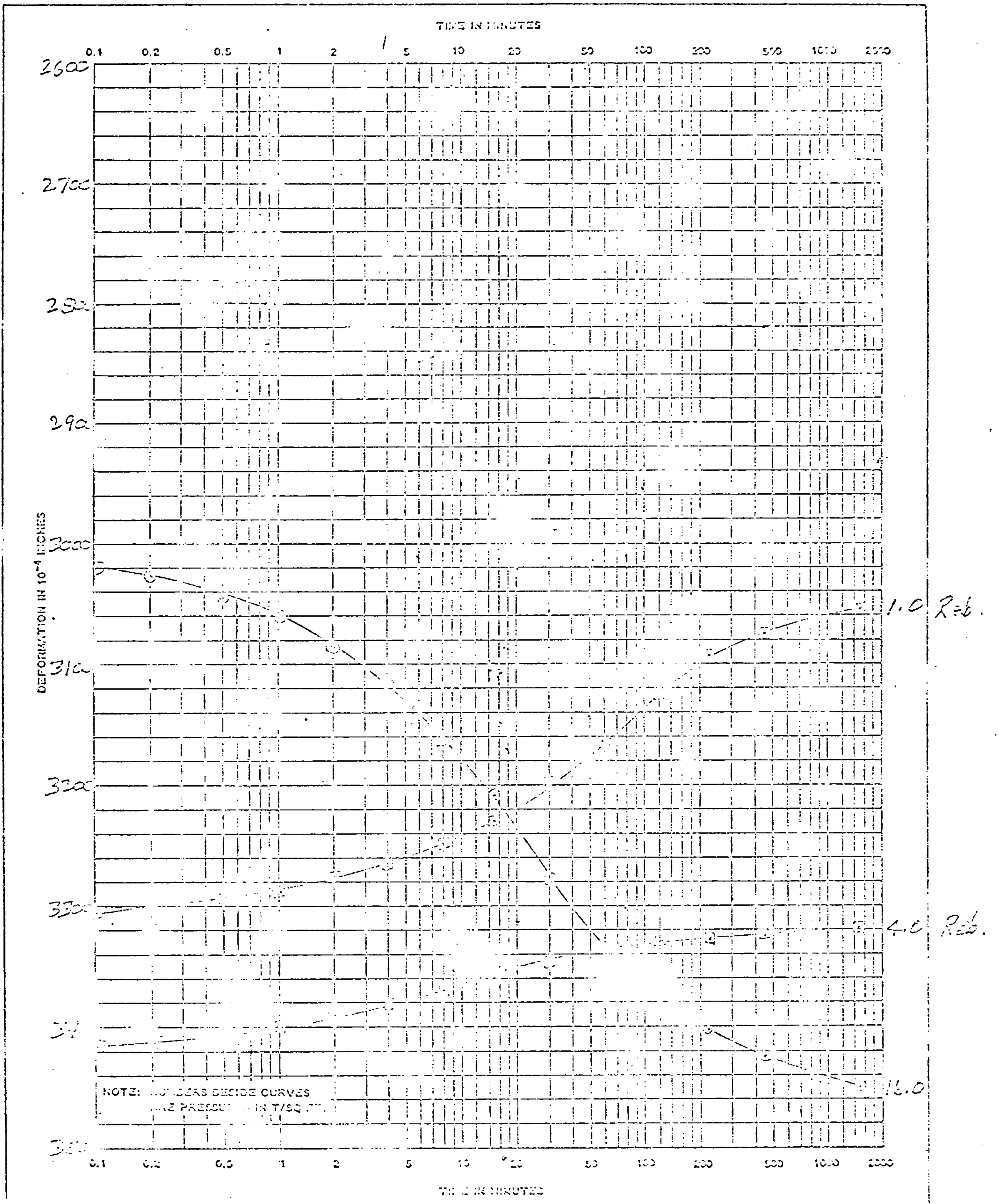
ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

C 3427

FIG. A18d



Project SEABROOK LOCK

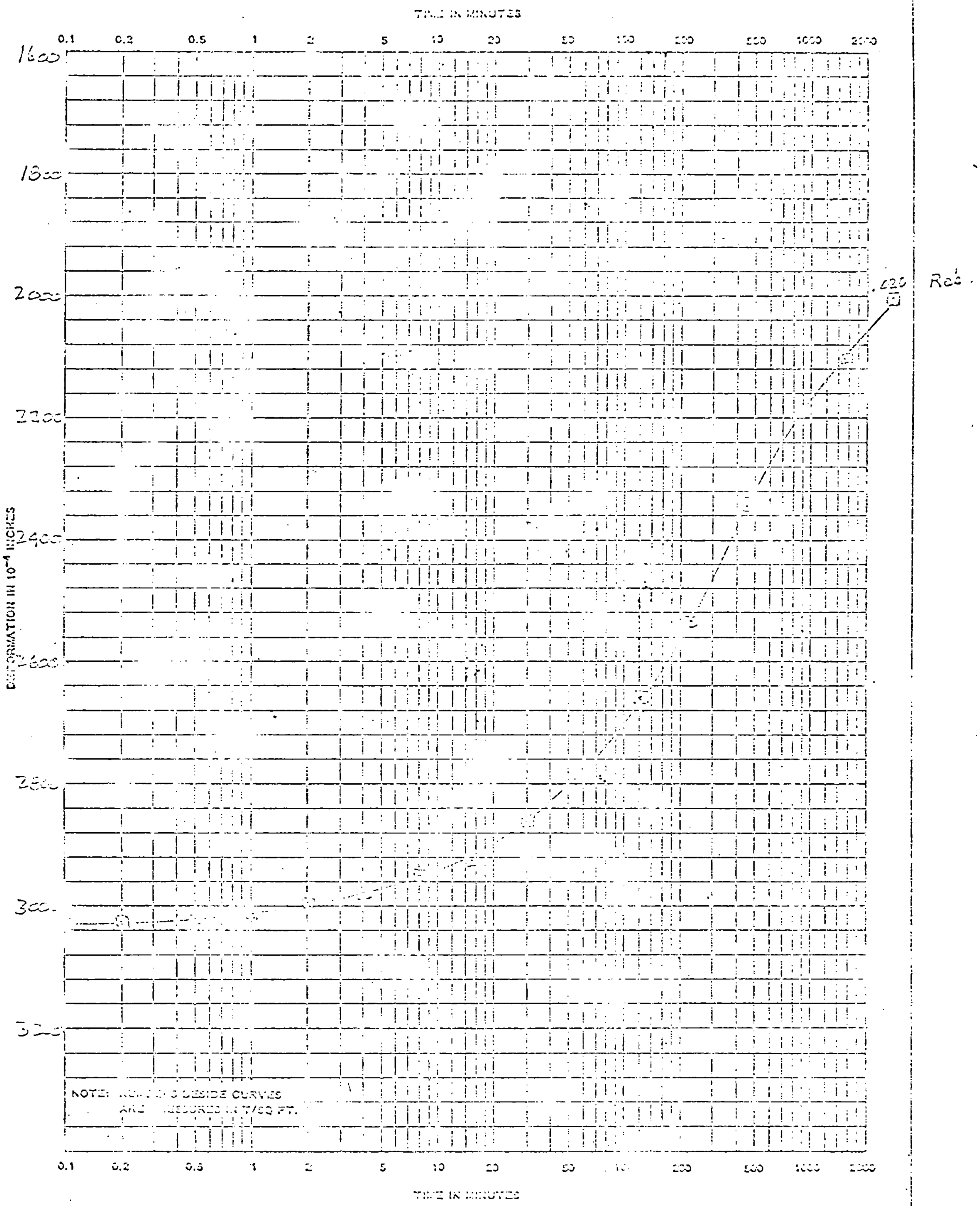
Area

Boring No. U-1 | Sample No. 6 | Depth 11.8-[illegible].2 | Date FEB 66

ENG FORM 2000 1 MAY 63 PREV 000 EDITIONS ARE OBSOLETE. [illegible] (TRANSLUCENT)

C 3/27

FIG. A18e



Project SEABROOK LOCK

Area

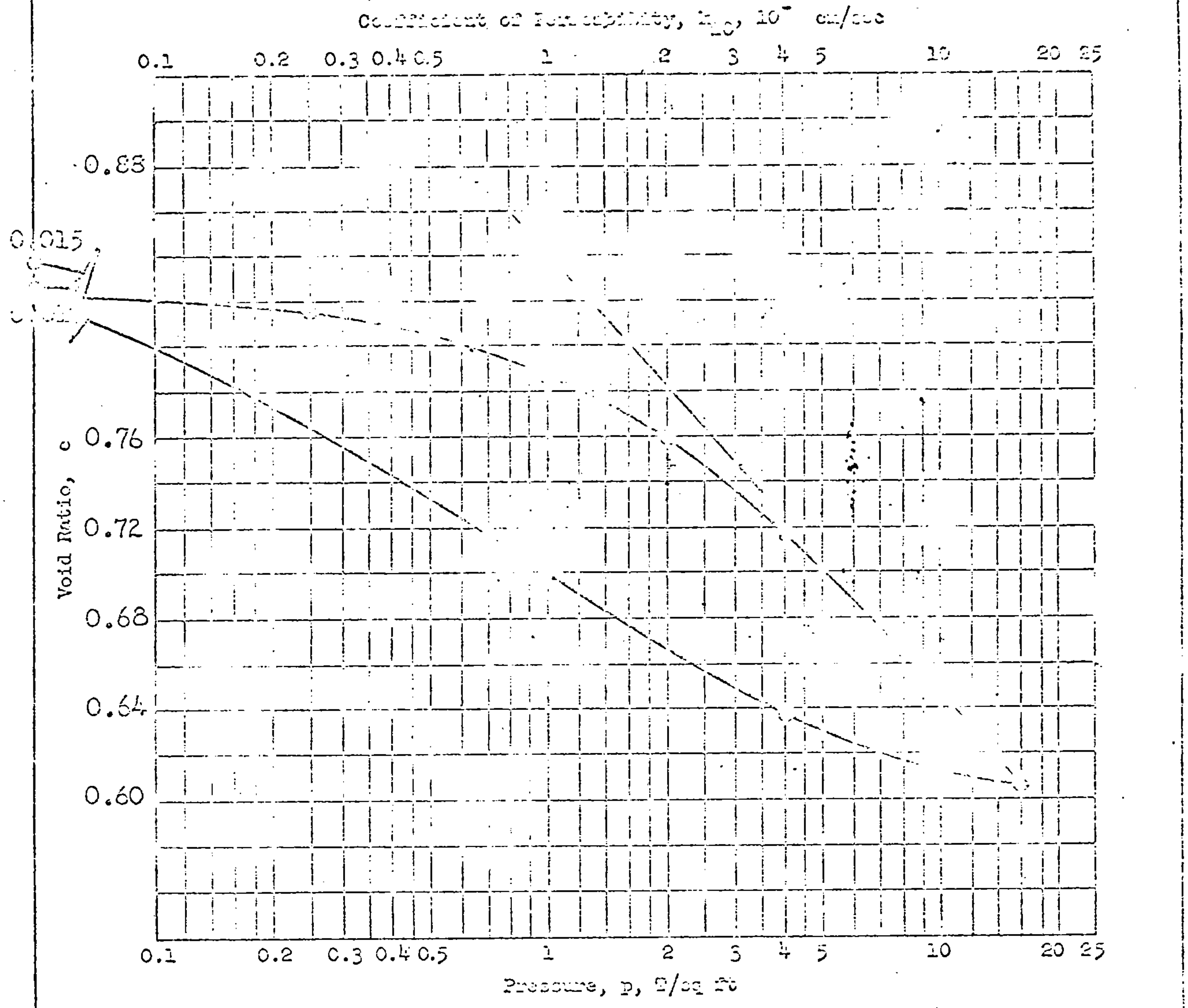
Boring No. U-1 Sample No. 6 Depth 11.8-14.2 Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

C[illegible]

(TRANSLUCENT)

FIG. A18?

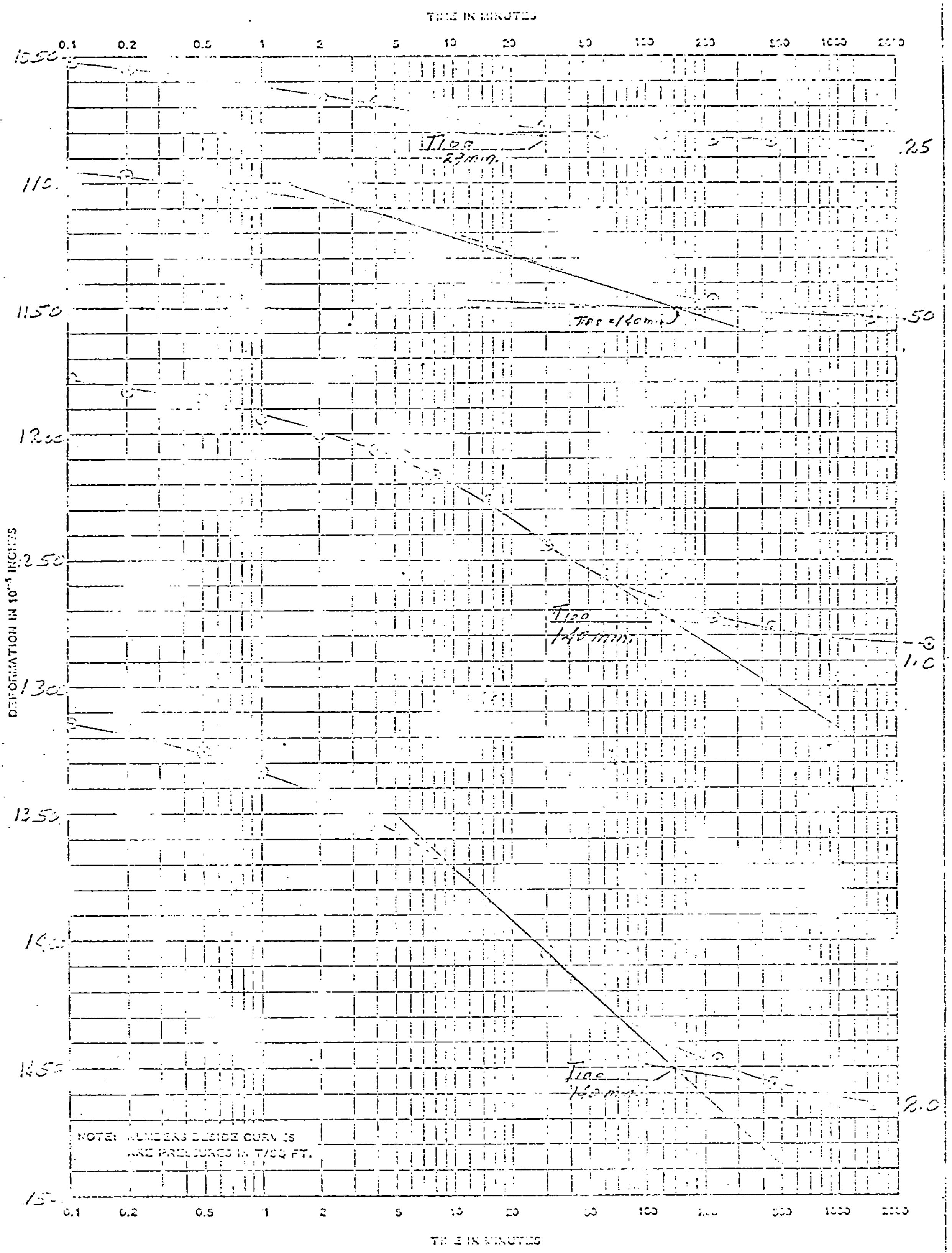


Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 4.25 in.	Ht 1.200 in.	Water Content, w_o	30.0 %	w_f	30.3 %
Overburden Pressure, p_o 0.52 T/sq ft		Void Ratio, e_o	0.828	e_f	0.633
Preconsol. Pressure, p_c 2.6 T/sq ft		Saturation, S_o	98.2 %	S_f	98.0 %
Compression Index, C_c = 0.20		Dry Density, γ_d	93.7 lb/ft³		92.1
Classification CLAY (CH), green, *		k_{20} at e_o = × 10^{-} cm/sec			
LL 78	G_s = 2.71	Project STARROCK LOCK			
PL 20	D_{10}				
Remarks * contains sandy pocket		Area			
		Boring No. U-3		Sample No. 9	
		Depth 19.3-20.4		Date 26-Jan-66	
		CONSOLIDATION TEST REPORT			

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

3 5424

FIG. A19



Project SEABROOK LOCK

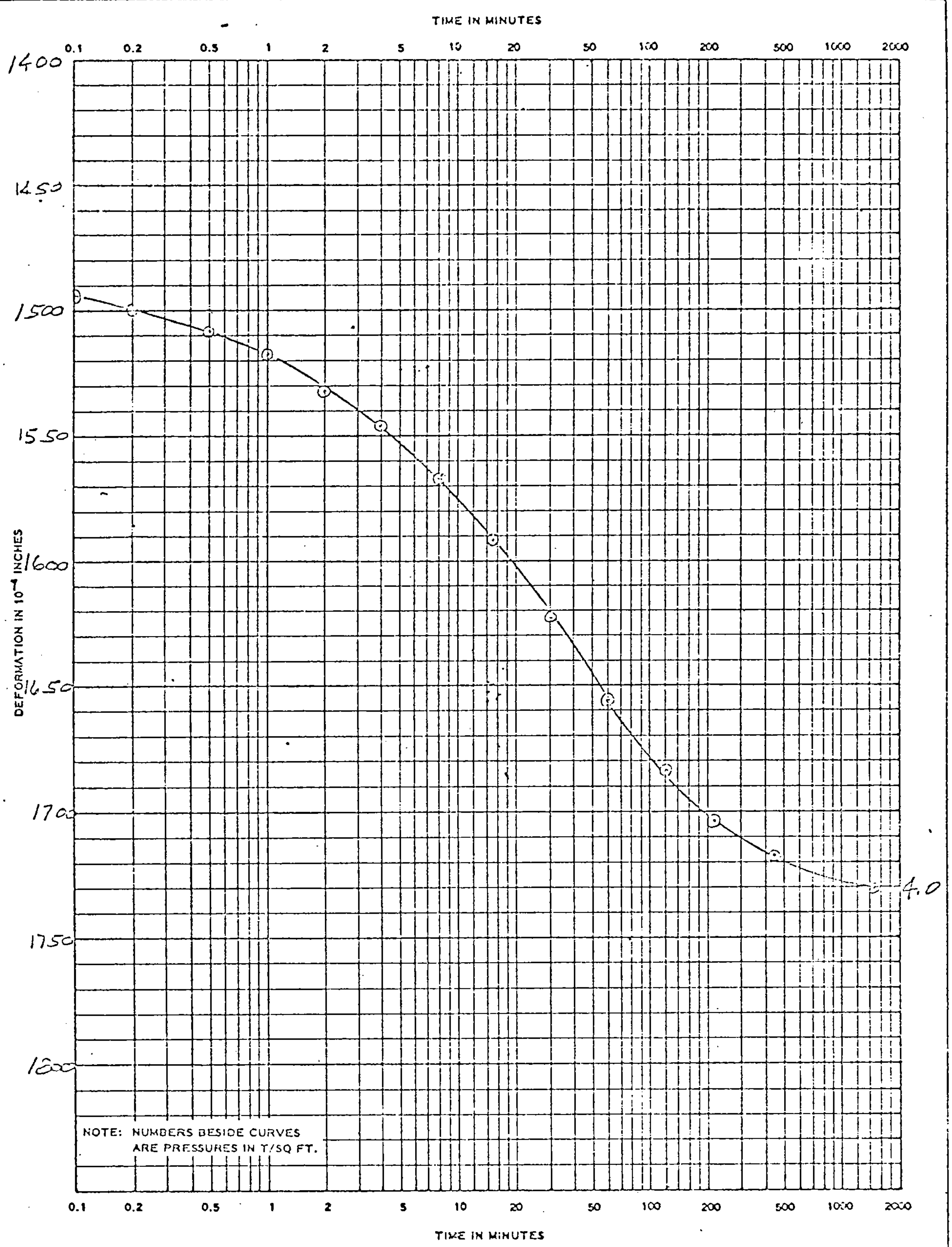
Area

Boring No. U-1 | Sample No. 9 | Depth 19.3-20.4 | Date JAN 66

ENG FORM 2003
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

(TRANSLUCENT)

FIG. A19a



NOTE: NUMBERS BESIDE CURVES ARE PRESSURES IN T/SQ FT.

Project SEABROCK LOCK

Area

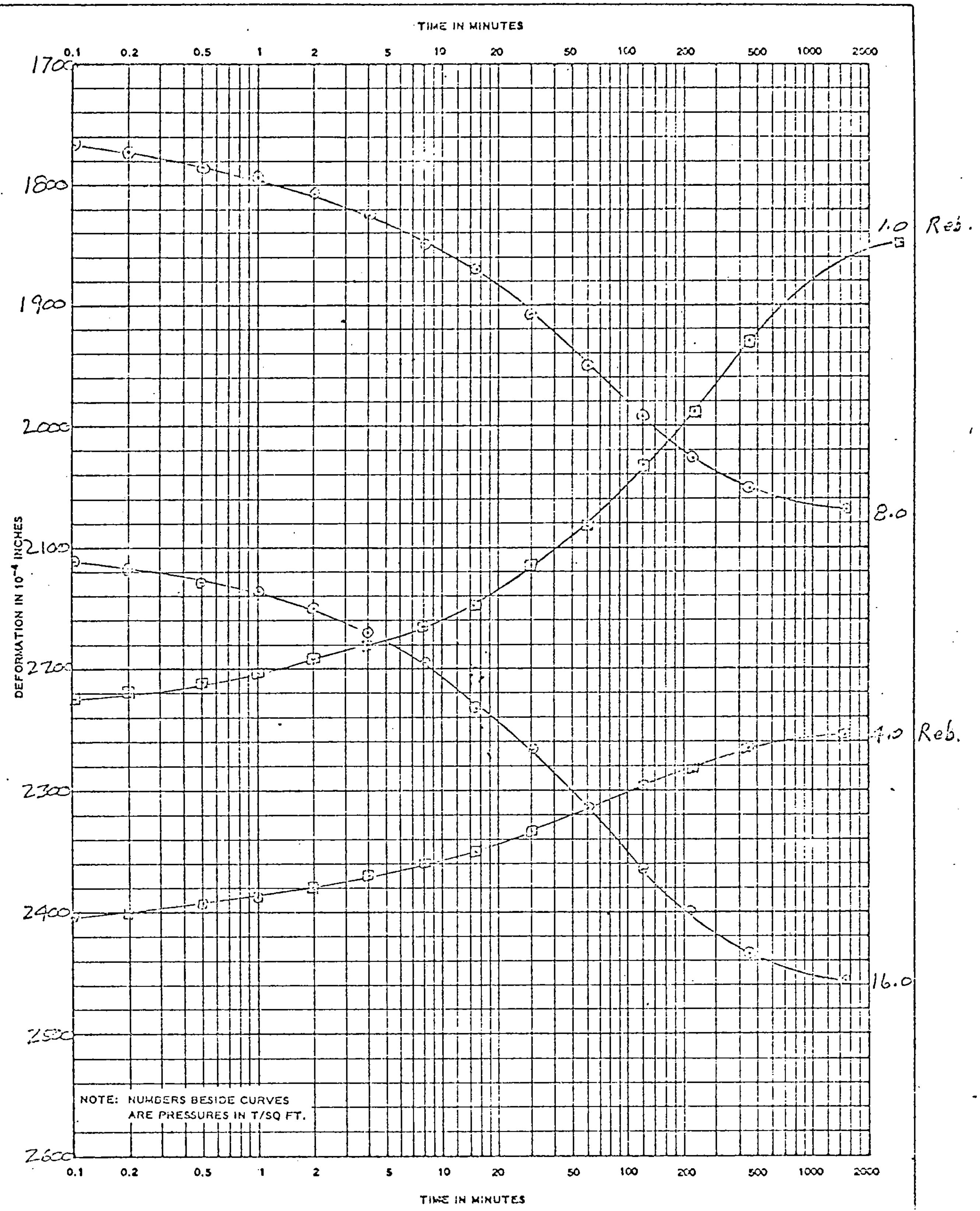
Boring No. U-1	Sample No. 9	Depth 19.3-20.4	Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A19b



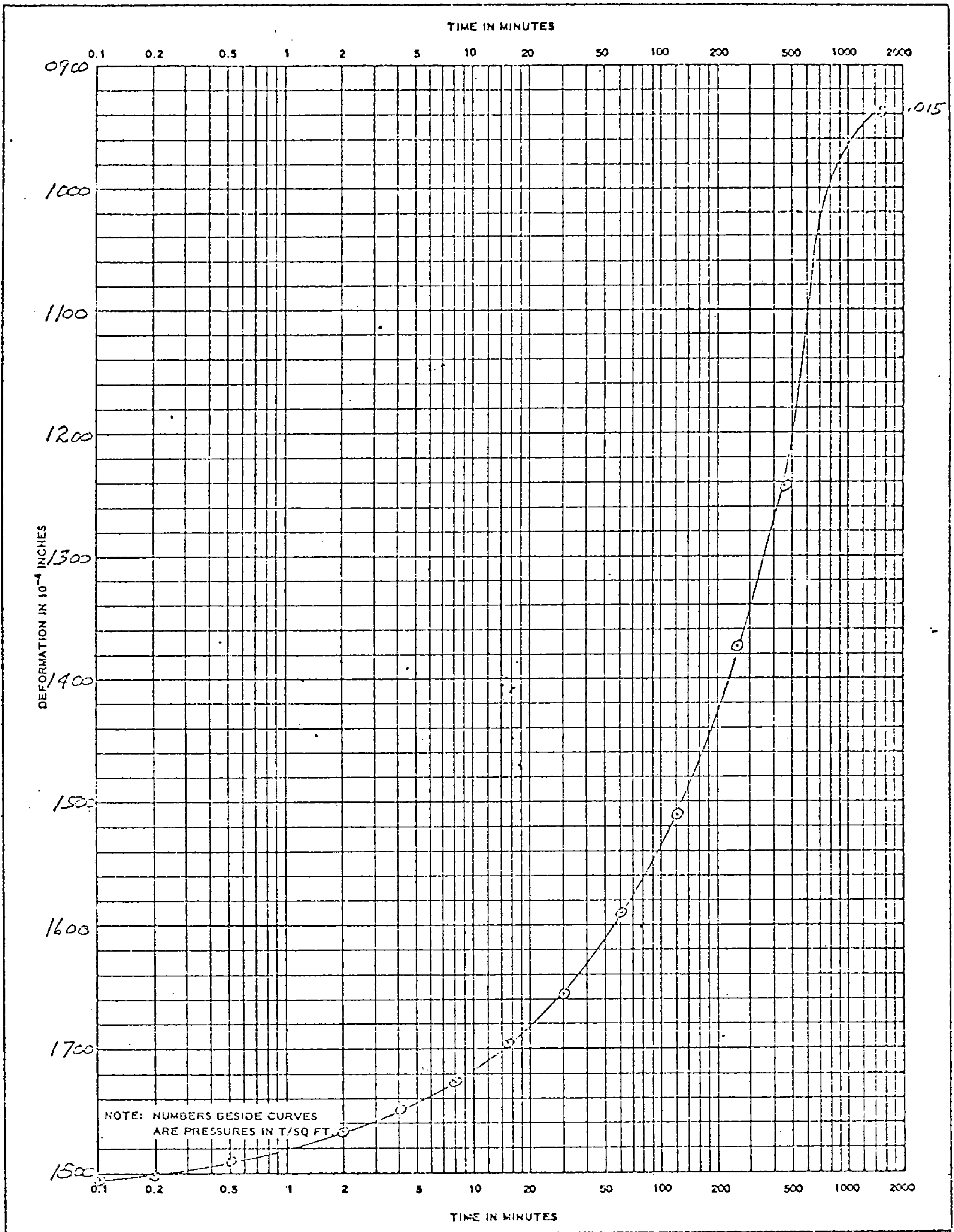
Project SEABROOK LOCK			
Area			
Boring No. U-1	Sample No. 9	Depth 19.3-20.4	Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A19c



Project SEABROOK LOCK

Area

Boring No. U-1	Sample No. 9	Depth 19.3-20.4	Date FEB 66

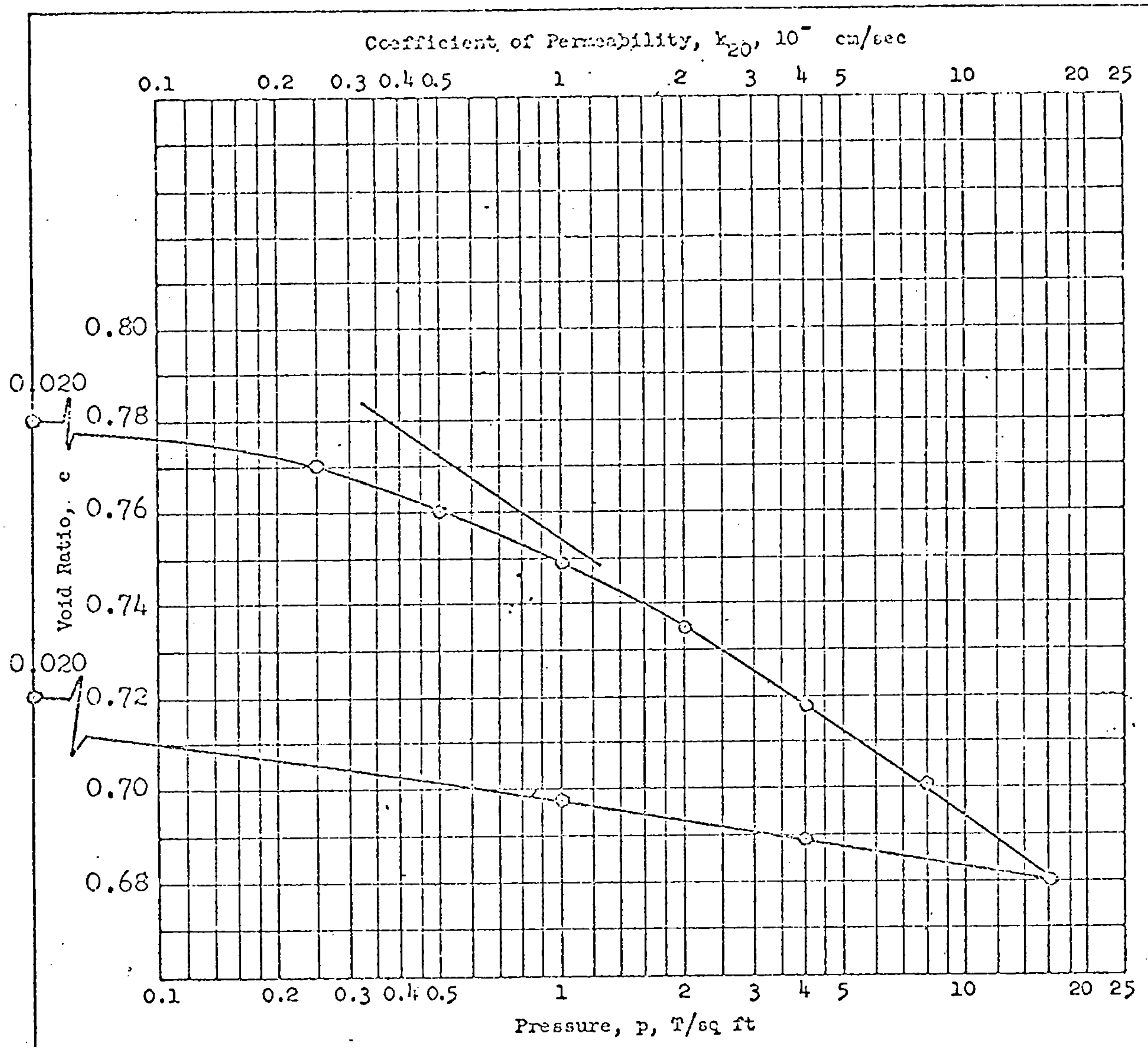
ENG FORM 2033
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427

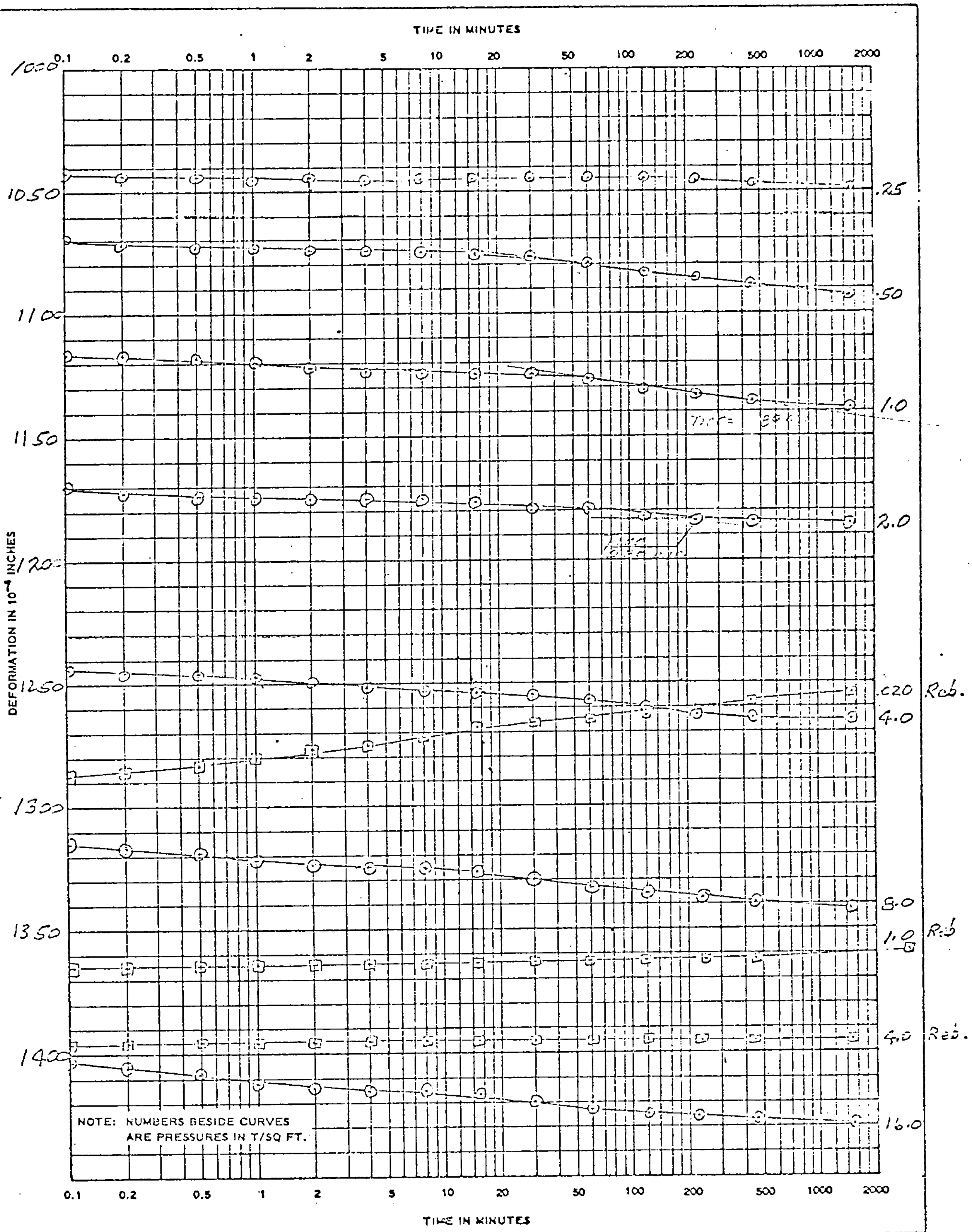
FIG. A19d



Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 2.50 in.	Ht 0.753 in.	Water Content, w_o	28.6 %	w_f	27.3 %
Overburden Pressure, p_o 0.94 T/sq ft		Void Ratio, e_o	0.781	e_f	0.721
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	97.1 %	S_f	100 %
Compression Index, C_c = 0.06		Dry Density, γ_d	92.9 lb/ft^3		96.2
Classification SILTY SAND (SP-SM)*		k_{20} at e_o = × 10⁻ cm/sec			

LL ——	G_s = 2.65	Project SEABROOK LOCK	
PL ——	D_{10} = 0.09 mm		
Remarks * brown		Area	
		Boring No. U-1	Sample No. 15
		Depth 33.1-35.6	Date 26-Jan-66
		JB CONSOLIDATION TEST REPORT	

FIG. A20



Project SEABROOK LOCK

Area

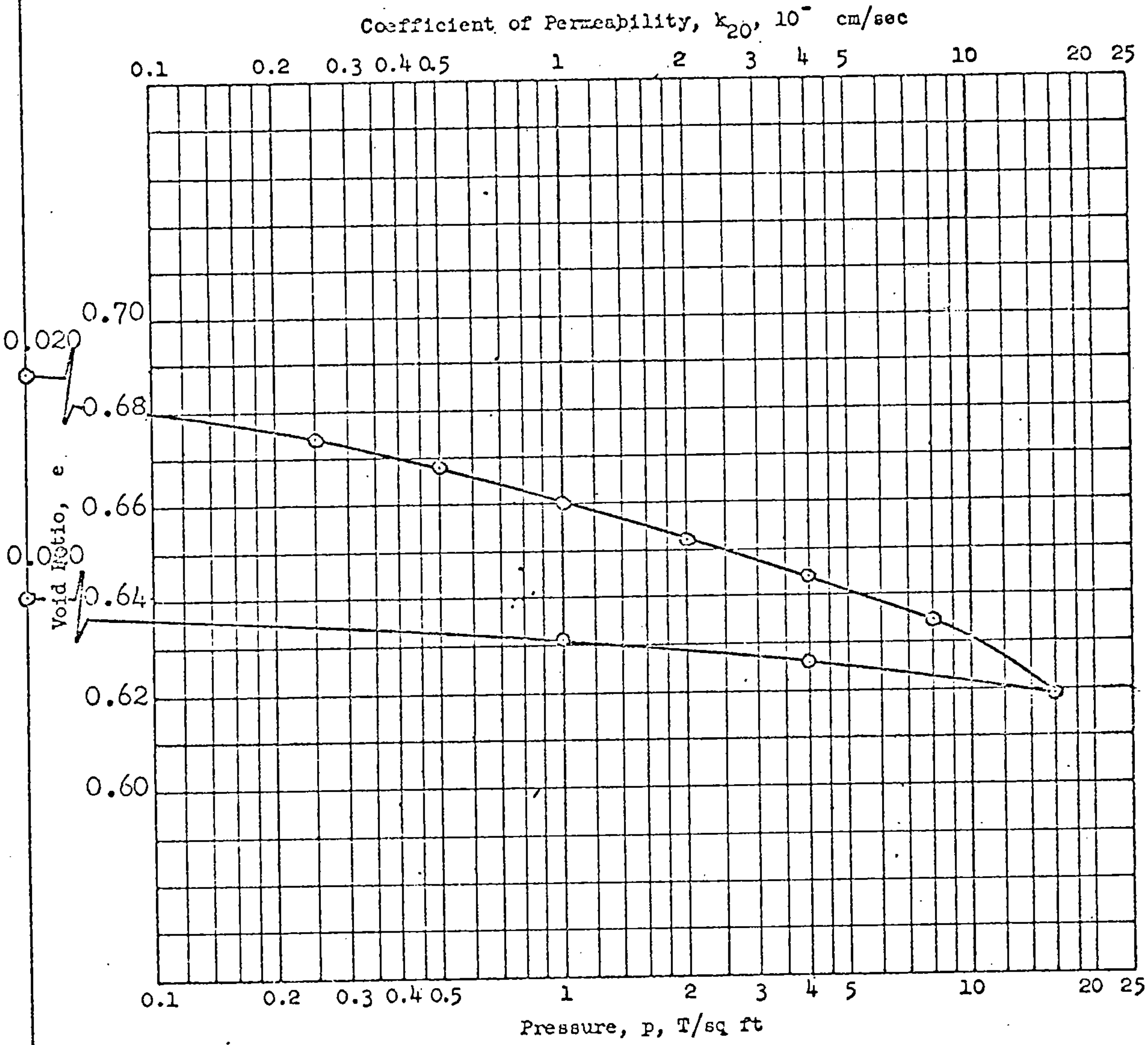
Boring No. U-1	Sample No. 15	Depth 33.1-35.6	Date FEB 66

ENG FORM 1 MAY 63 2088
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A20a



Type of Specimen UNDISTURBED		
Diam 2.50 in.	Ht 0.751 in.	
Overburden Pressure, Po		T/sq ft
Preconsol. Pressure, Pc		T/sq ft
Compression Index, C_c —		
Classification SILTY SAND (SM), gray		
LL ——	G_s = 2.65	
PL ——	D_{10}	

Remarks

Before Test		After Test	
Water Content, w_o	22.9 %	w_f	24.0 %
Void Ratio, e_o	0.689	e_f	0.641
Saturation, S_o	88.0 %	S_f	99.2 %
Dry Density, γ_d	98.1 lb/ft³		100.8

k_{20} at e_o = × 10⁻ cm/sec

Project SEABROOK LOCK

Area

Boring No. U-1	Sample No. 22
Depth 61.1-63.5	Date 28-Jan-66

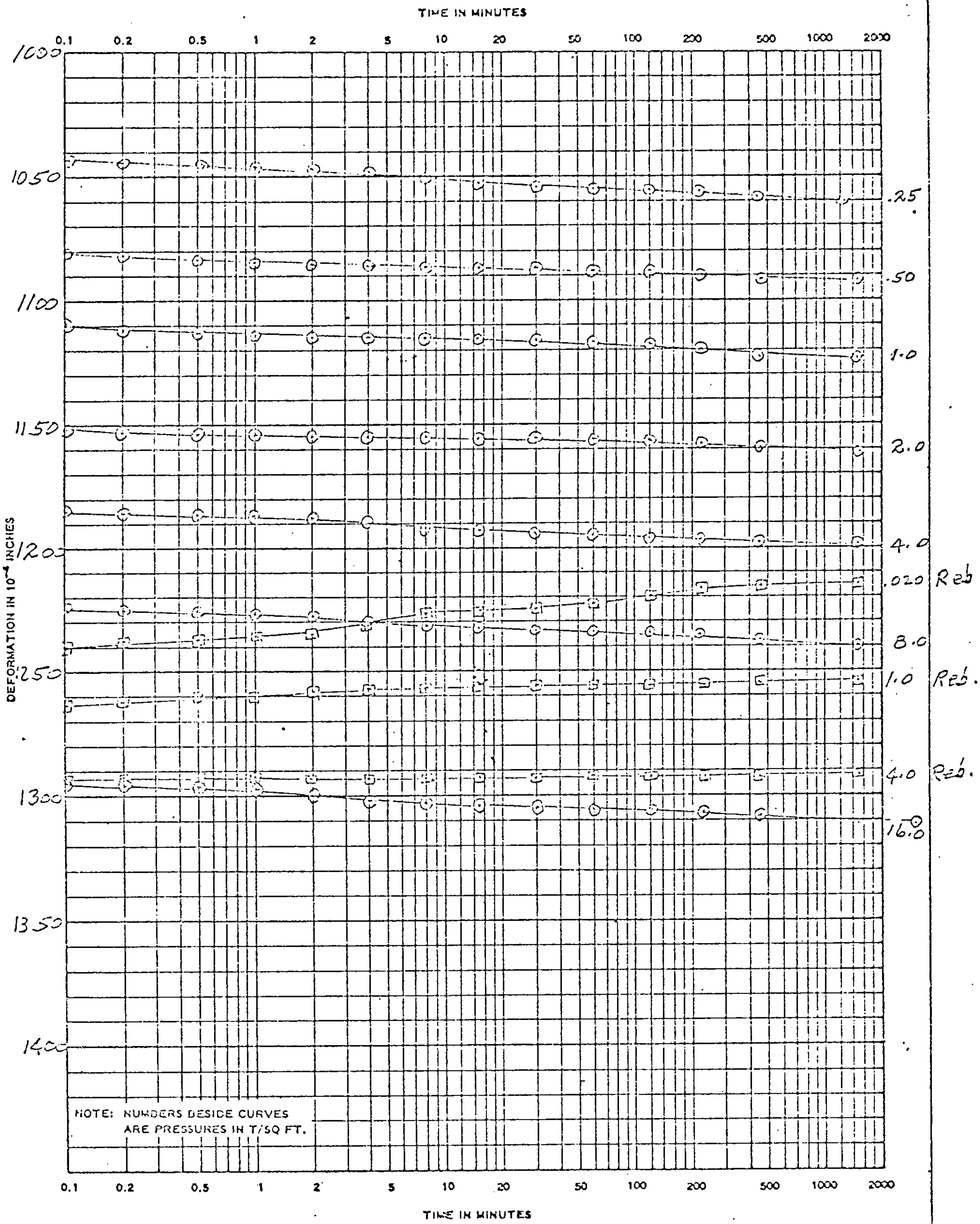
JB

CONSOLIDATION TEST REPORT

ENG FORM 2090
1 MAY 63

PREVIOUS EDITIONS ARE OBSOLETE. *(TRANSLUCENT)*

FIG. A21



NOTE: NUMBERS BESIDE CURVES ARE PRESSURES IN T/SQ FT.

Project SEABROOK LOCK

Area

Boring No. U-1 | Sample No. 22 | Depth 61.1-63.5 | Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A21a

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
AND
MISSISSIPPI RIVER - GULF OUTLET, LOUISIANA

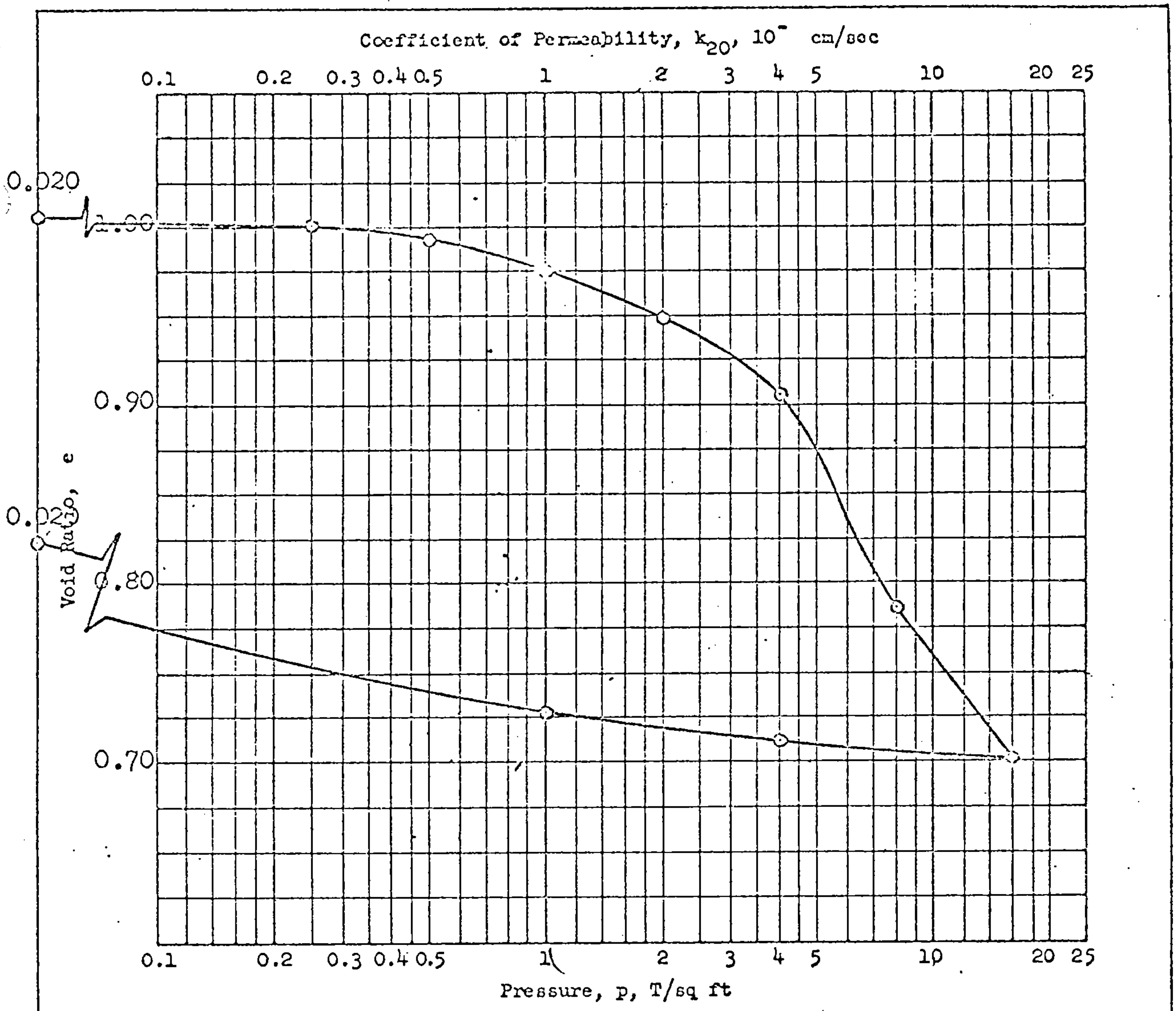
SEABROOK LOCK
DESIGN MEMORANDUM NO. 1 GENERAL

APPENDIX D

GEOLOGY AND SOILS

FIELD EXPLORATIONS AND GEOLOGY

EXHIBIT NO. 7 - CONSOLIDATION TEST REPORTS AND TIME CURVES, BORING U-2



Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 2.50 in.	Ht 0.751 in.	Water Content, w_o	37.1 %	w_f	30.5 %
Overburden Pressure, p_o 0.42 T/sq ft		Void Ratio, e_o	1.04	e_f	0.825
Preconsol. Pressure, p_c 4.1 T/sq ft		Saturation, S_o	95.1 %	S_f	98.5 %
Compression Index, C_c ——		Dry Density, γ_d	81.7 lb/ft^3		91.4
Classification SILTY SAND (SC-SM)*		k_{20} at e_o =	× 10^{-} cm/sec		

LL 29	G_s = 2.67
PL 22	D_{10}
Remarks *brown	

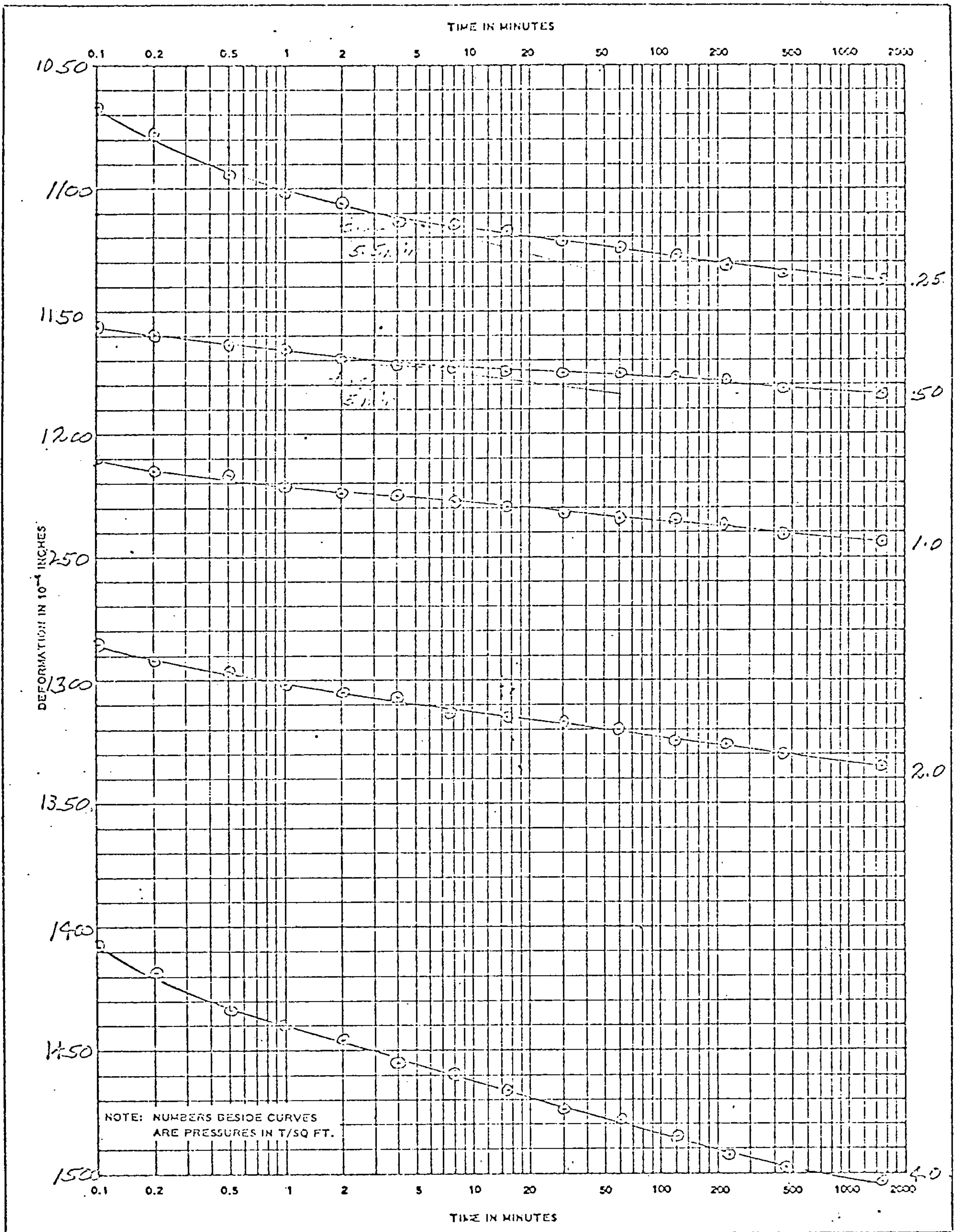
Project SEABROOK LOCK	
Area	
Boring No. U-2	Sample No. 3
Depth 14.5-16.8	Date 28-JAN-66

JB CONSOLIDATION TEST REPORT

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

0 3424

FIG. A22



NOTE: NUMBERS BESIDE CURVES ARE PRESSURES IN T/SQ FT.

Project SEABROOK LOCK

Area

Boring No. U-2

Sample No. 3

Depth 14.5-16.8

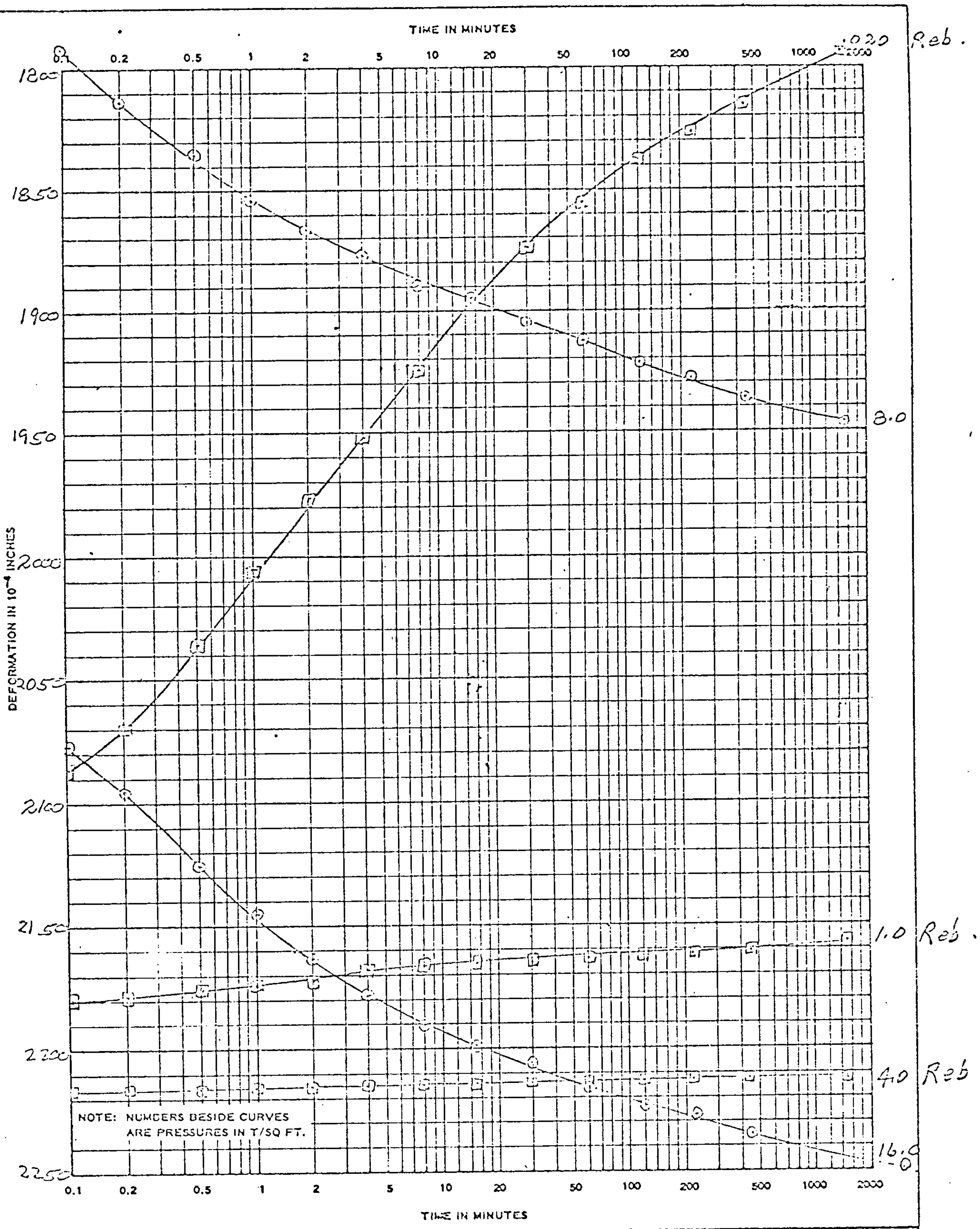
Date FEB 66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A22a



Project SEABROOK LOCK			
Area			
Boring No. U-2	Sample No. 3	Depth 14.5-16.8	Date FEB 66

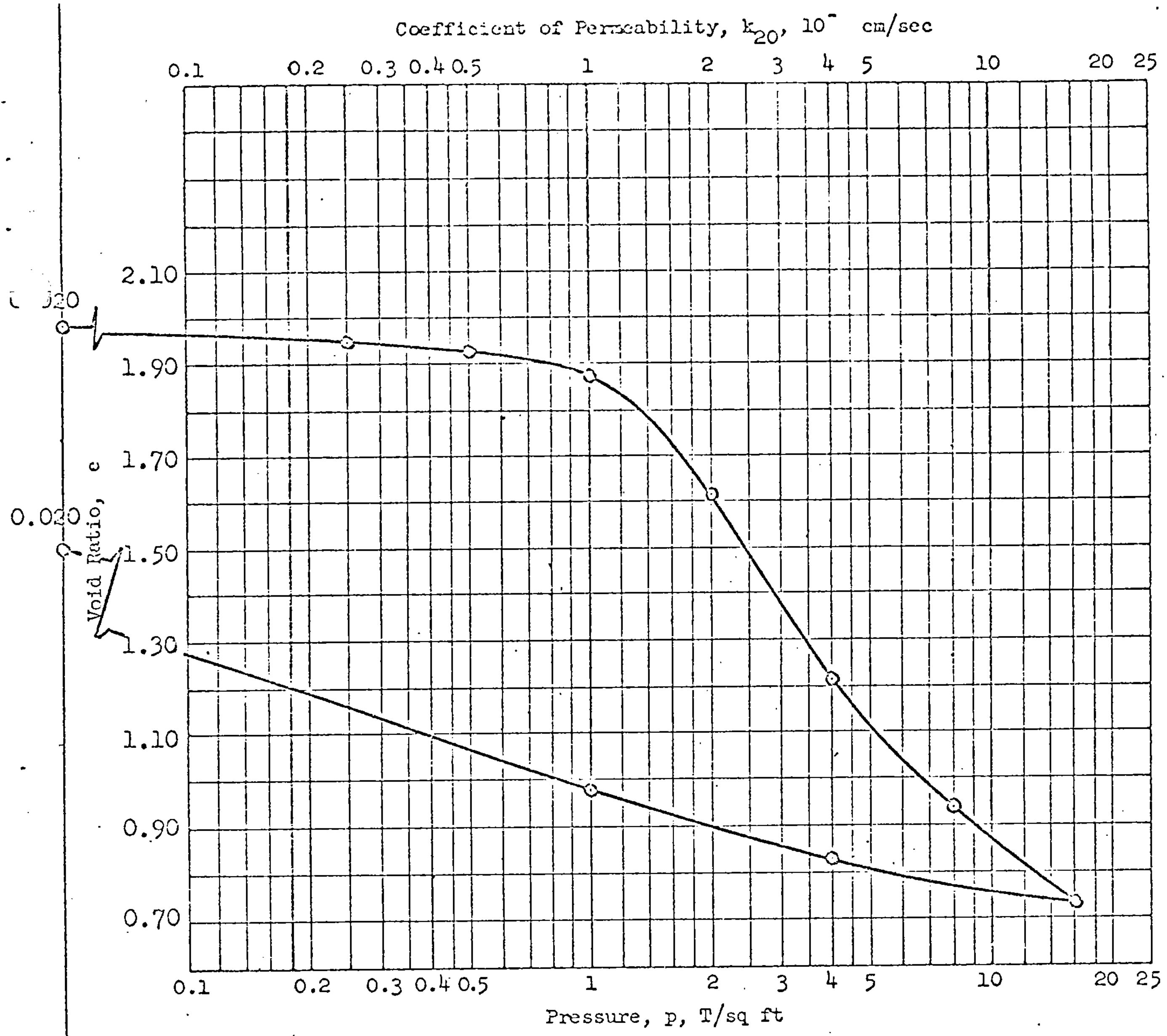
ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427

FIG. A22b

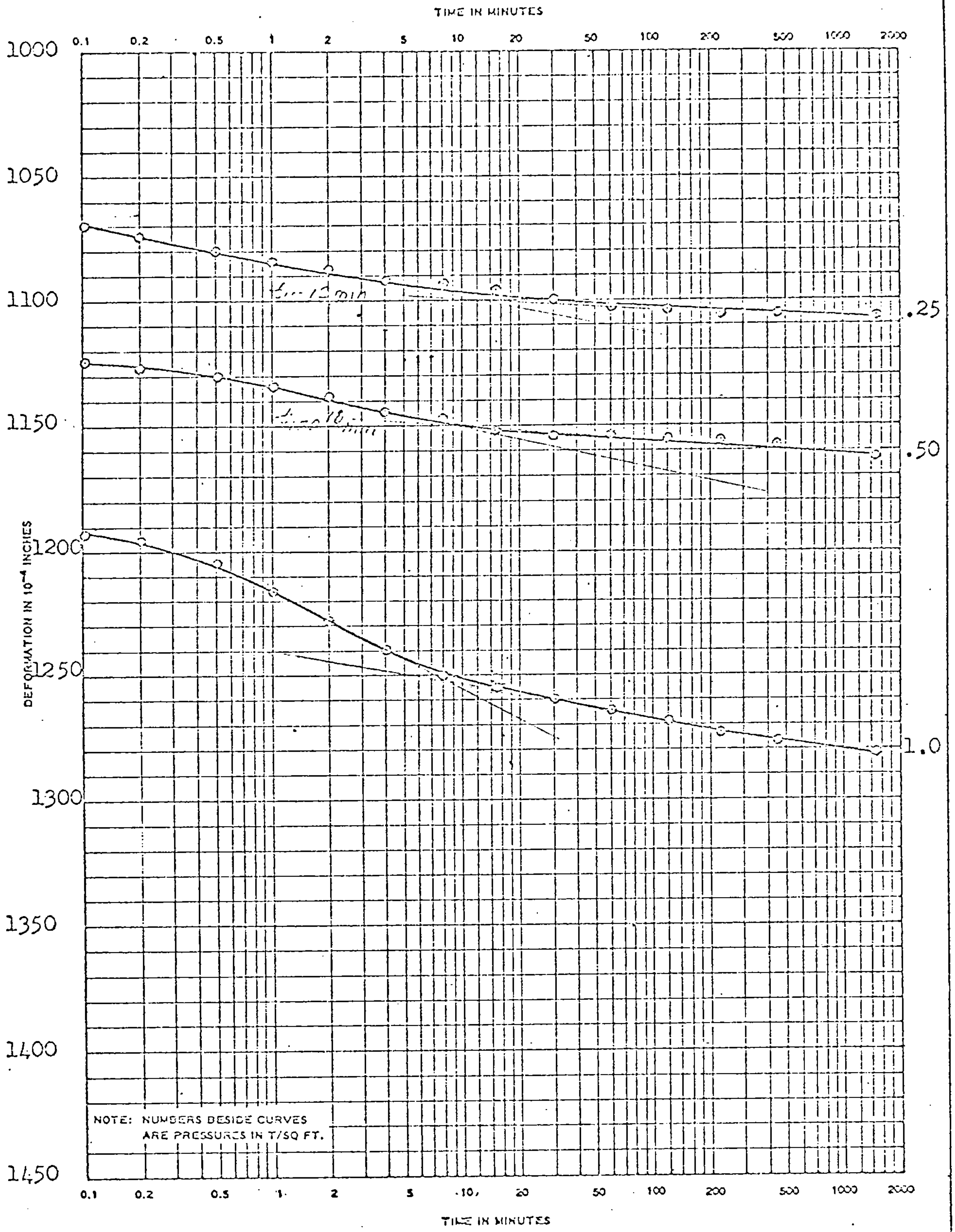


Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 2.50 in.	Ht 0.766 in.	Water Content, w_o	73.4 %	w_f	55.0 %
Overburden Pressure, p_o 0.55 T/sq ft		Void Ratio, e_o	1.99	e_f	1.50
Preconsol. Pressure, p_c 1.4 T/sq ft		Saturation, S_o	99.6 %	S_f	98.7 %
Compression Index, C_c —		Dry Density, γ_d	56.3 lb/ft^3		67.2
Classification CLAY(CH), gray, contains small sandy pockets		k_{20} at e_o = × 10^{-} cm/sec			
LL —	G_s = 2.70	Project SEABROOK LOCK			
PL —	D_{10}				
Remarks		Area			
		Boring No. U-2		Sample No. 6	
		Depth 20.5 - 22.9		Date 8 FEB 1966	

JB CONSOLIDATION TEST REPORT

FIG. A23

ENG FORM 2090 [illegible] (TRANSLUCENT)



Project SEABROOK LOCK

Area

Boring No. U-2

Sample No. 6

Depth 20.5-22.9

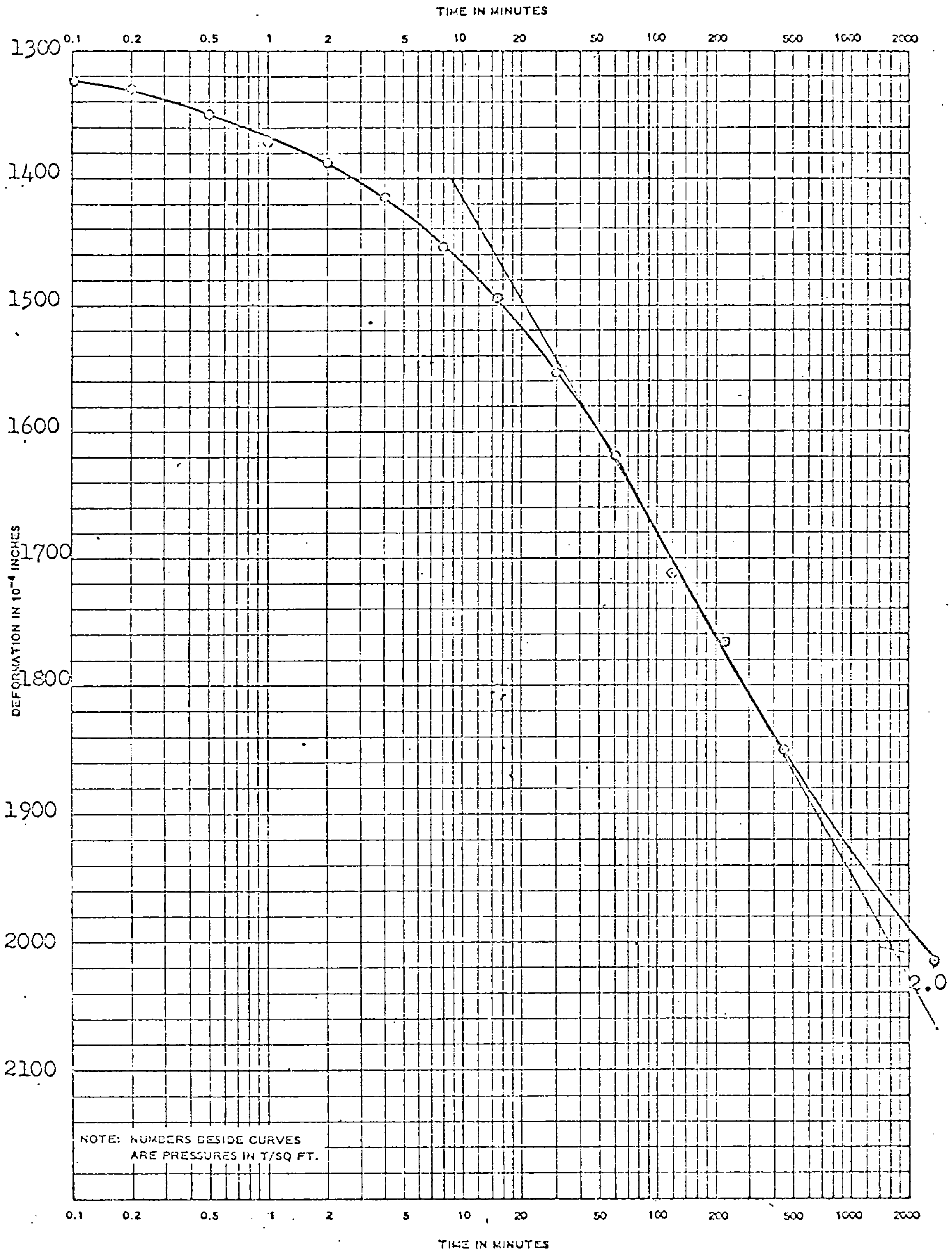
Date 8-FEB-66

ENG FORM 2083
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

FIG. A23a



Project SEABROOK LOCK

Area

Boring No. U-2

Sample No. 6

Depth 20.5-22.9

Date 8-FEB-66

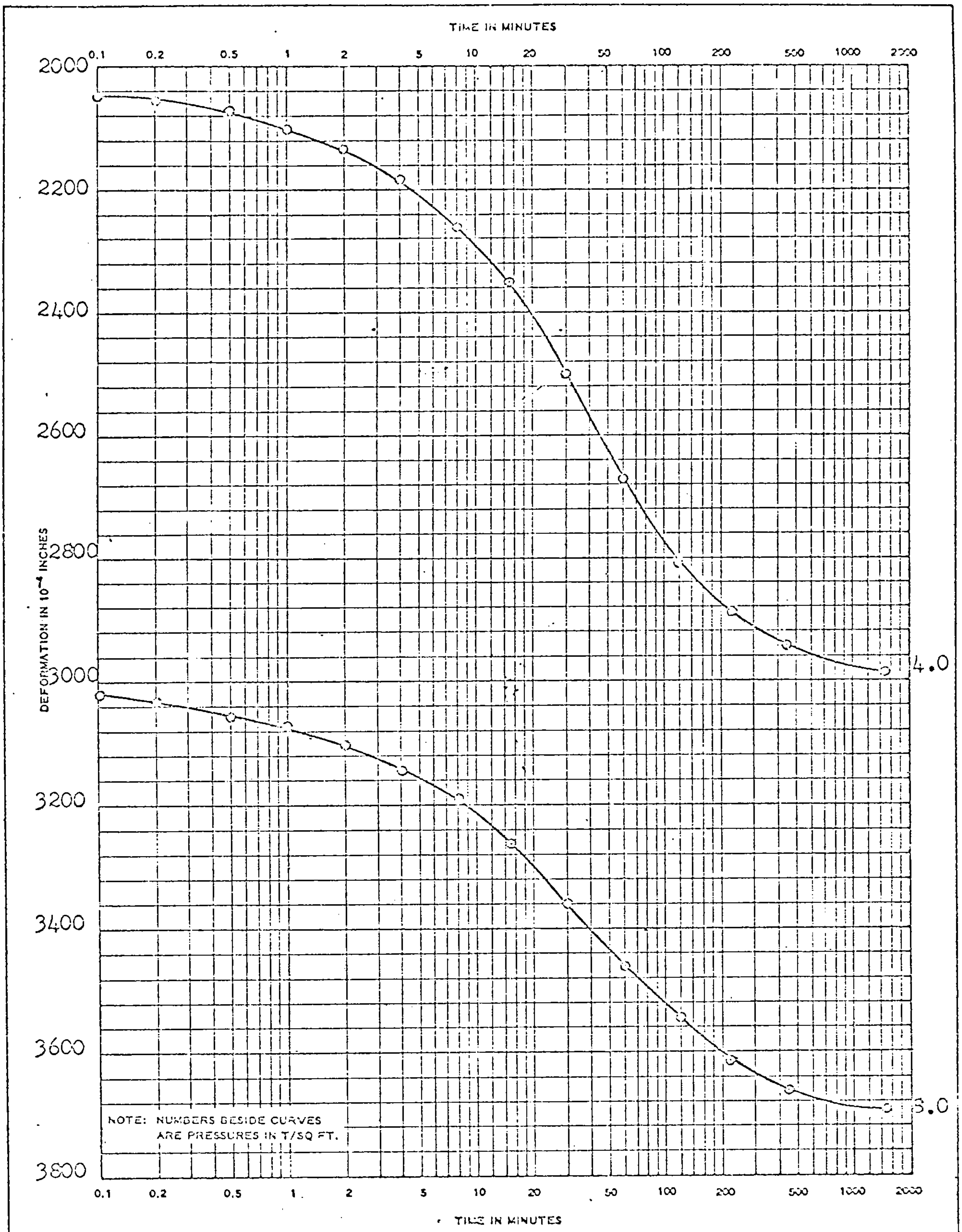
ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427

FIG. A23b



Project SEABROOK LOCK

Area

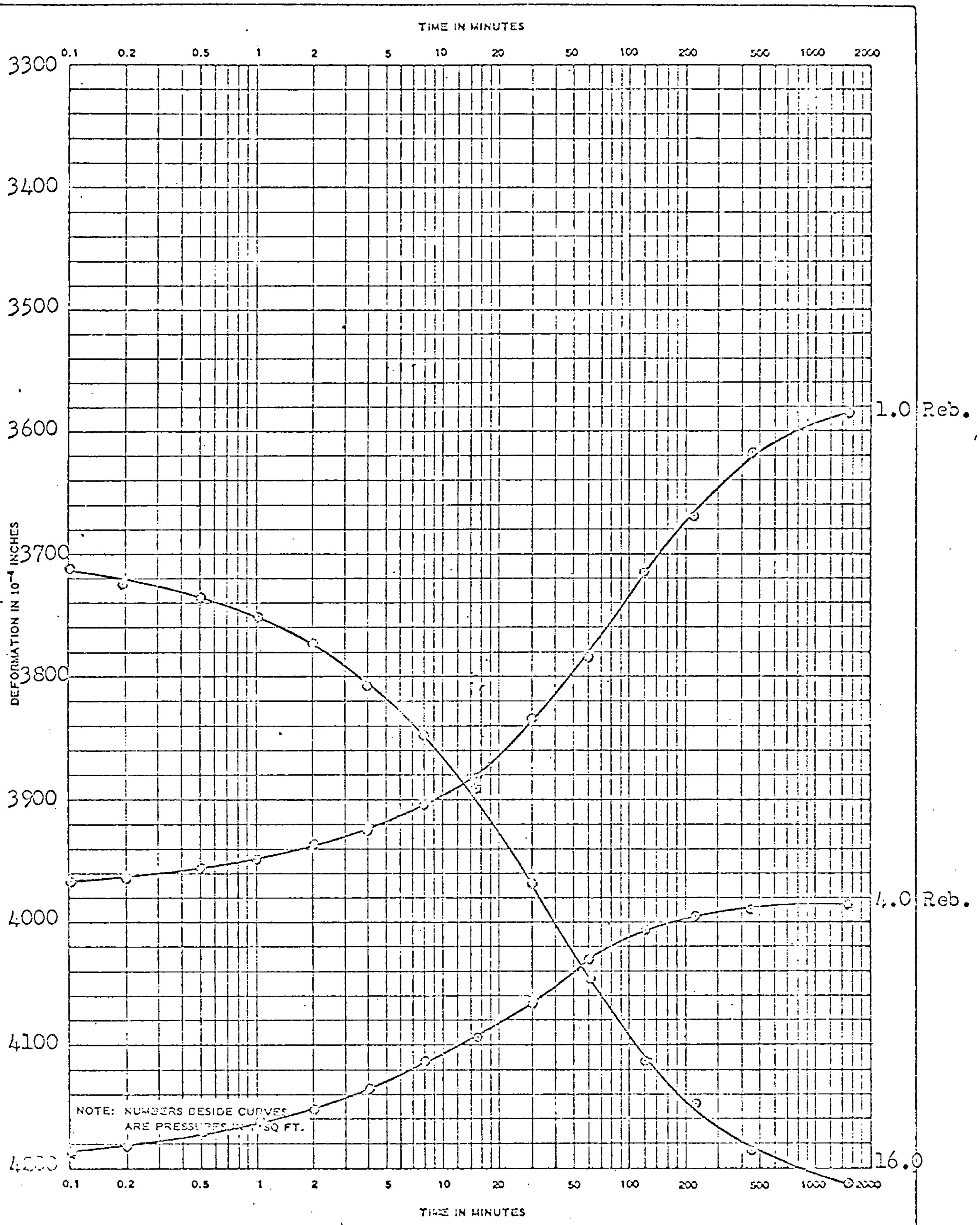
Boring No. U-2 | Sample No. 6 | Depth 20.5-22.9 | Date 8-FEB-66

ENG FORM 2088
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

FIG. A23c



Project SEABROOK LOCK			
Area			
Boring No. U-2	Sample No. 6	Depth 20.5-22.9	Date 8-FEB-66

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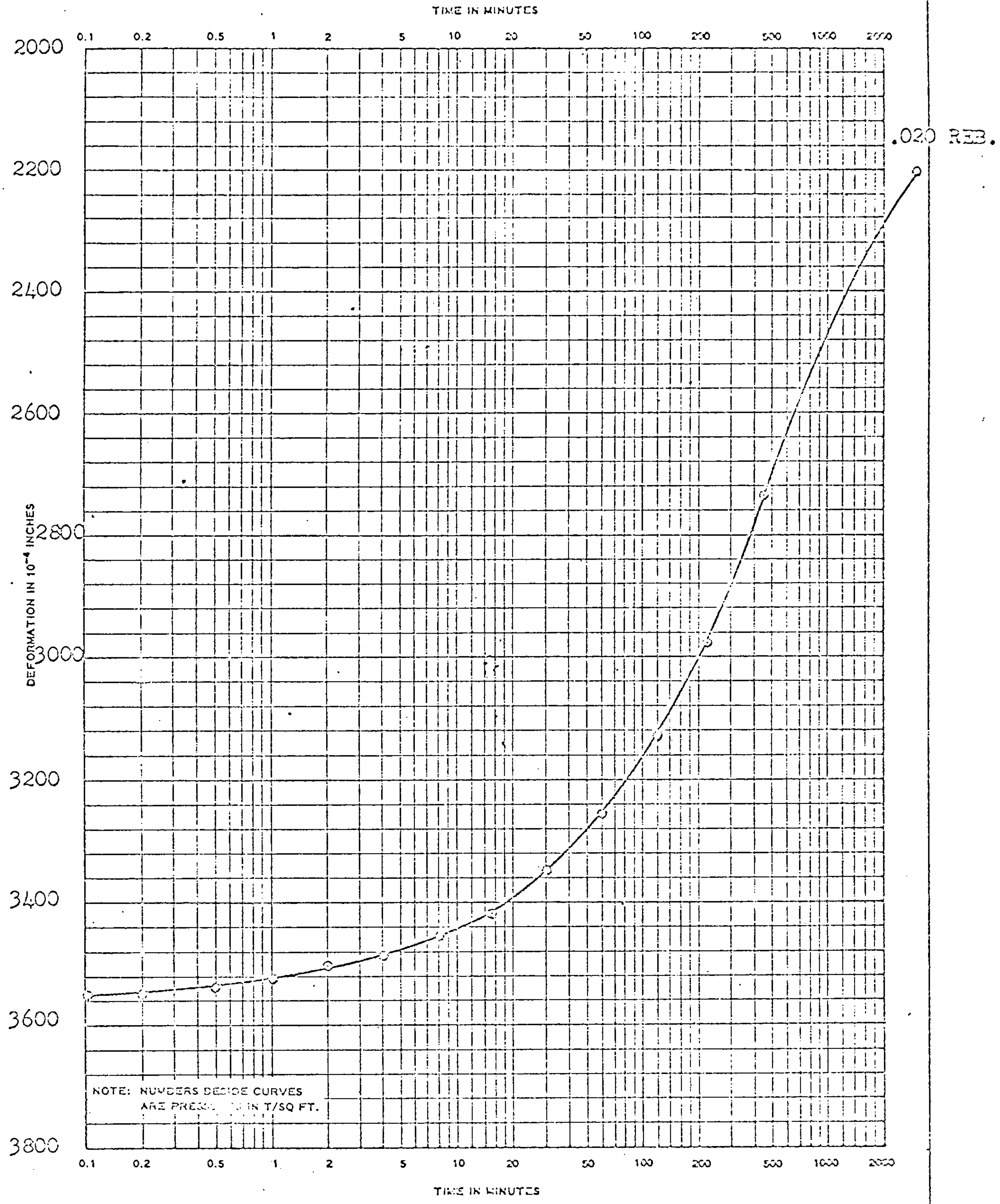
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427

FIG. A23d



Project SEABROOK LOCK

Area

Boring No. U-2 | Sample No. 6 | Depth 20.5-22.9 | Date 8-FEB-66

ENG FORM 2088
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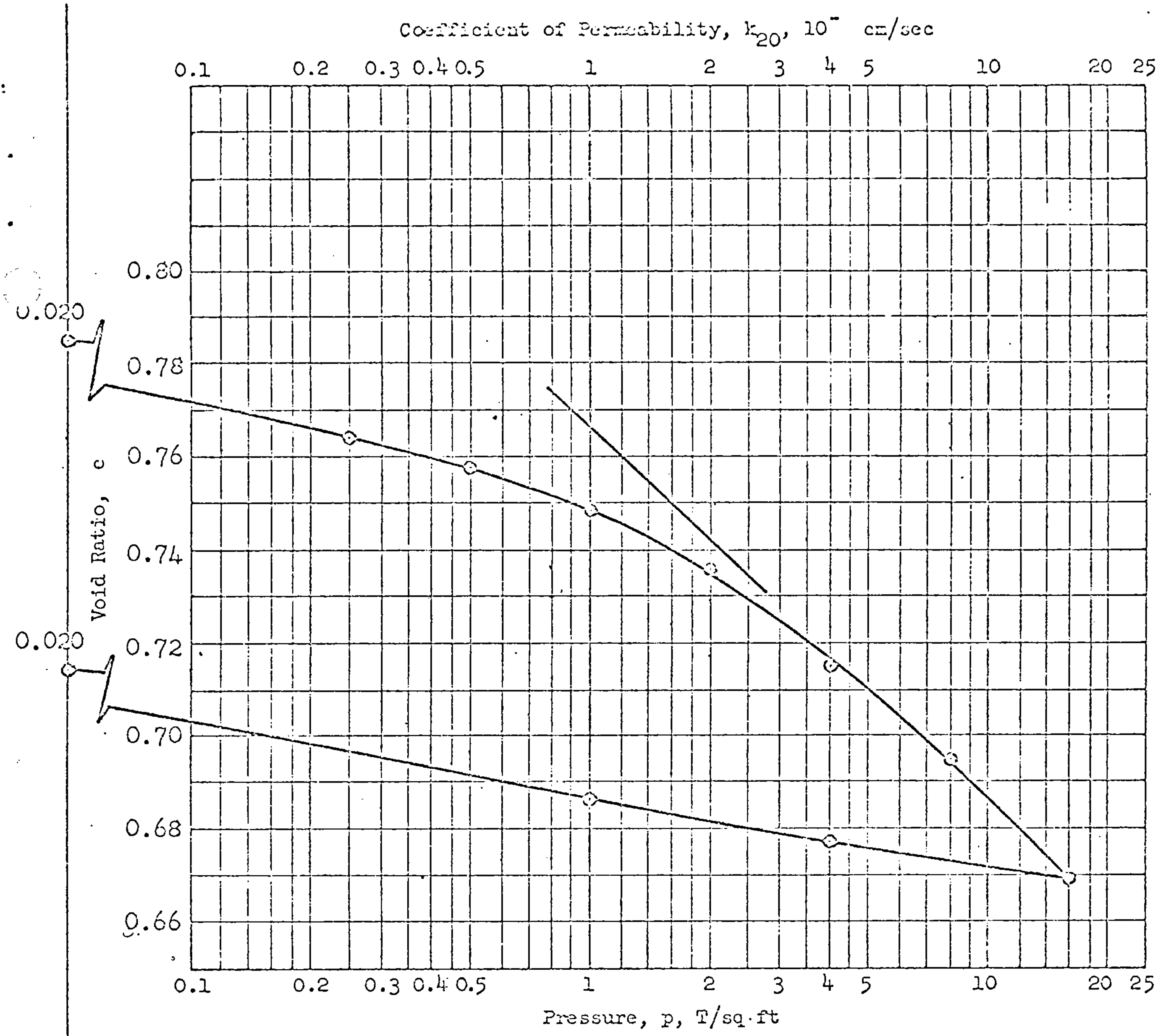
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

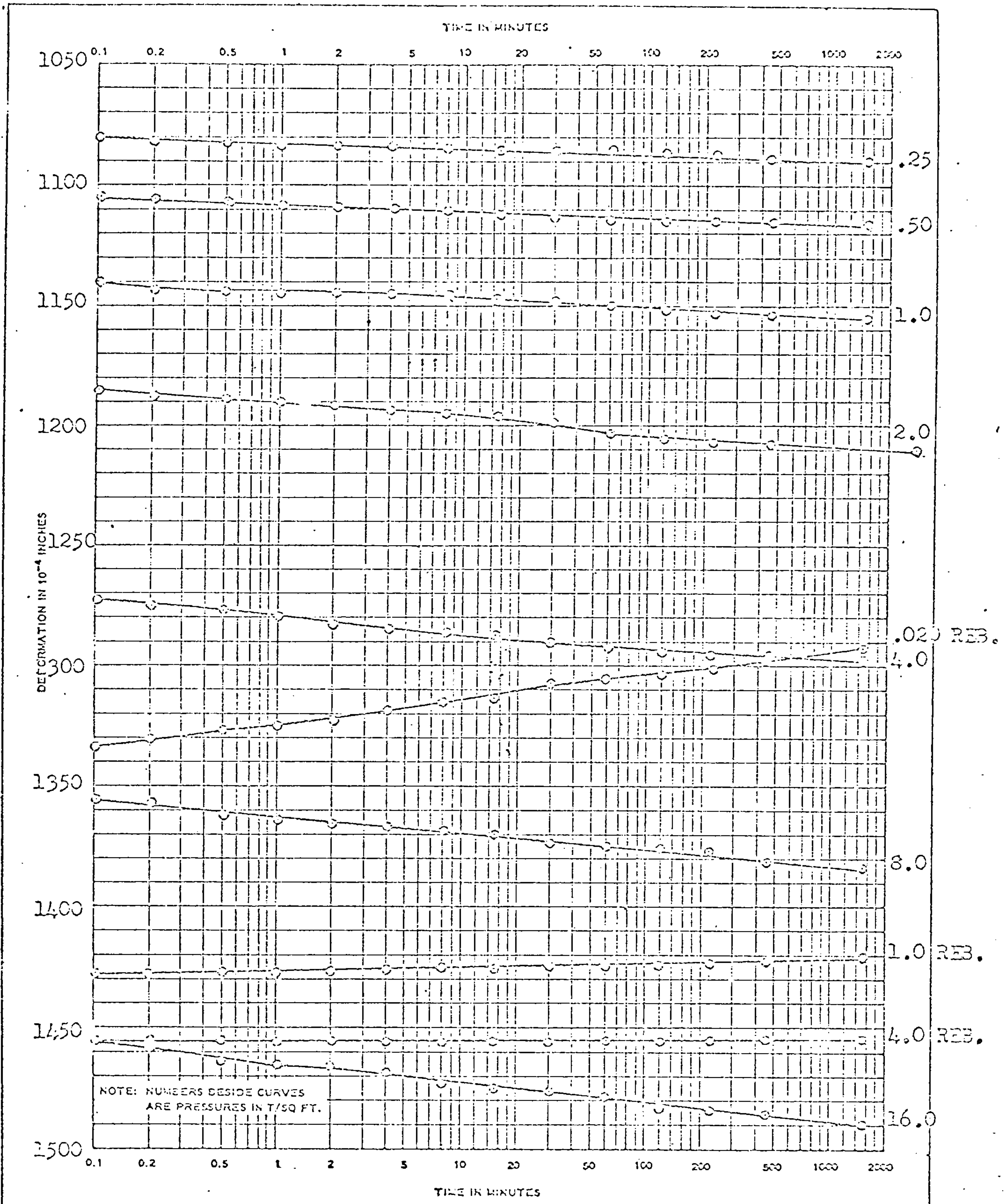
(TRANSLUCENT)

C 3427

FIG. A23c



Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 2.50 in.	Ht 0.753 in.	Water Content, w_o	27.5 %	w_f	26.8 %
Overburden Pressure, p_o 1.03 T/sq ft		Void Ratio, e_o	0.786	e_f	0.715
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	93.4 %	S_f	100 %
Compression Index, C_c = 0.08		Dry Density, γ_d	93.2 lb/ft^3		97.1
Classification SANDY SILT(ML), brown		k_{20} at e_o = × 10^{-} cm/sec			
LL —	G_s = 2.67	Project SEABROOK LOCK			
PL —	D_{10}				
Remarks		Area			
		Boring No. U-2		Sample No. 12	
		Depth 38.0 - 40.5		Date 8 FEB 1966	
		JB **CONSOLIDATION TEST REPORT**			



Project SEABROOK LOCK

Area

Boring No. U-2

Sample No. 12

Depth 38.0-40.5

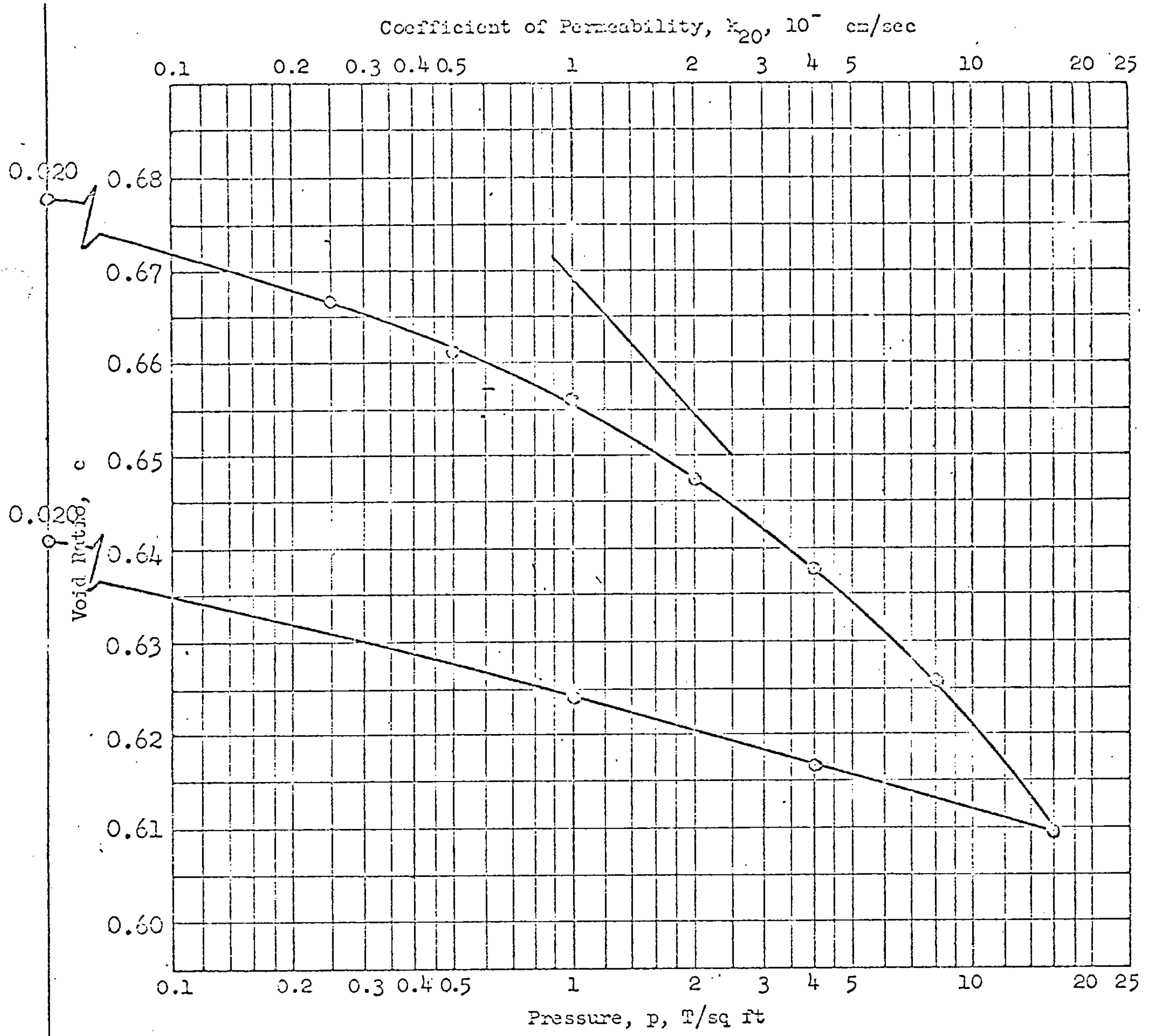
Date 8-FEB-66

ENG FORM 2033
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

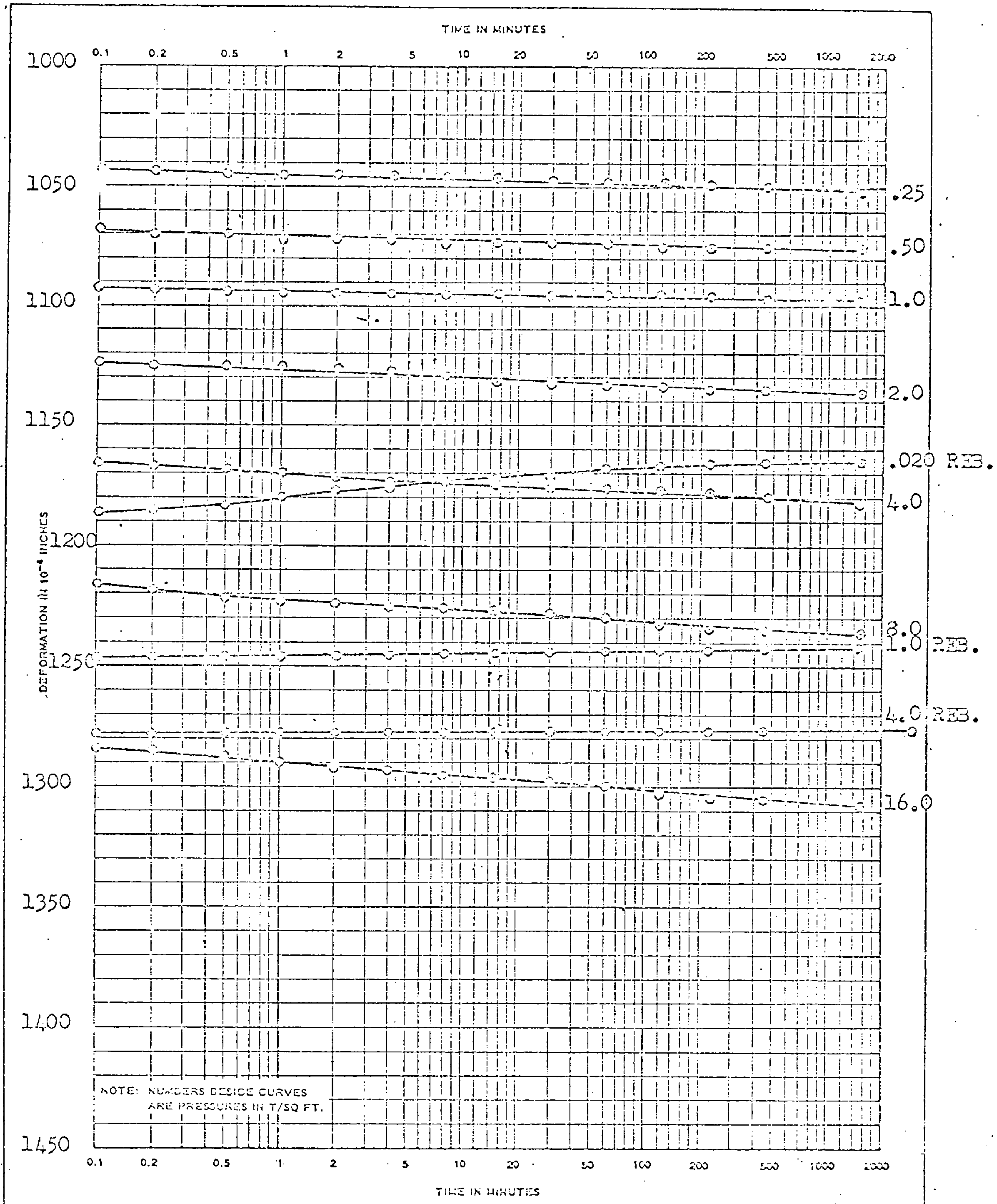
(TRANSLUCENT)

FIG. A24a



Type of Specimen UNDISTURBED		Before Test		After Test	
Diam 2.50 in.	Ht 0.751 in.	Water Content, w_o	25.1 %	w_f	24.1 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.678	e_f	0.642
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	98.5 %	S_f	99.8 %
Compression Index, C_c = 0.05		Dry Density, γ_d	98.8 lb/ft^3		101.0
Classification SAND(SP), brown		k_{20} at e_o = × 10^{-} cm/sec			
LL ——	G_s = 2.66	Project SEABROOK LOCK			
PL ——	D_{10}				
Remarks		Area			
		Boring No. U-2		Sample No. 19	
		Depth 63.0 - 65.5		Date 10 FEB 1966	
		JB CONSOLIDATION TEST REPORT			

FIG. [illegible]



Project SEABROOK LOCK

Area

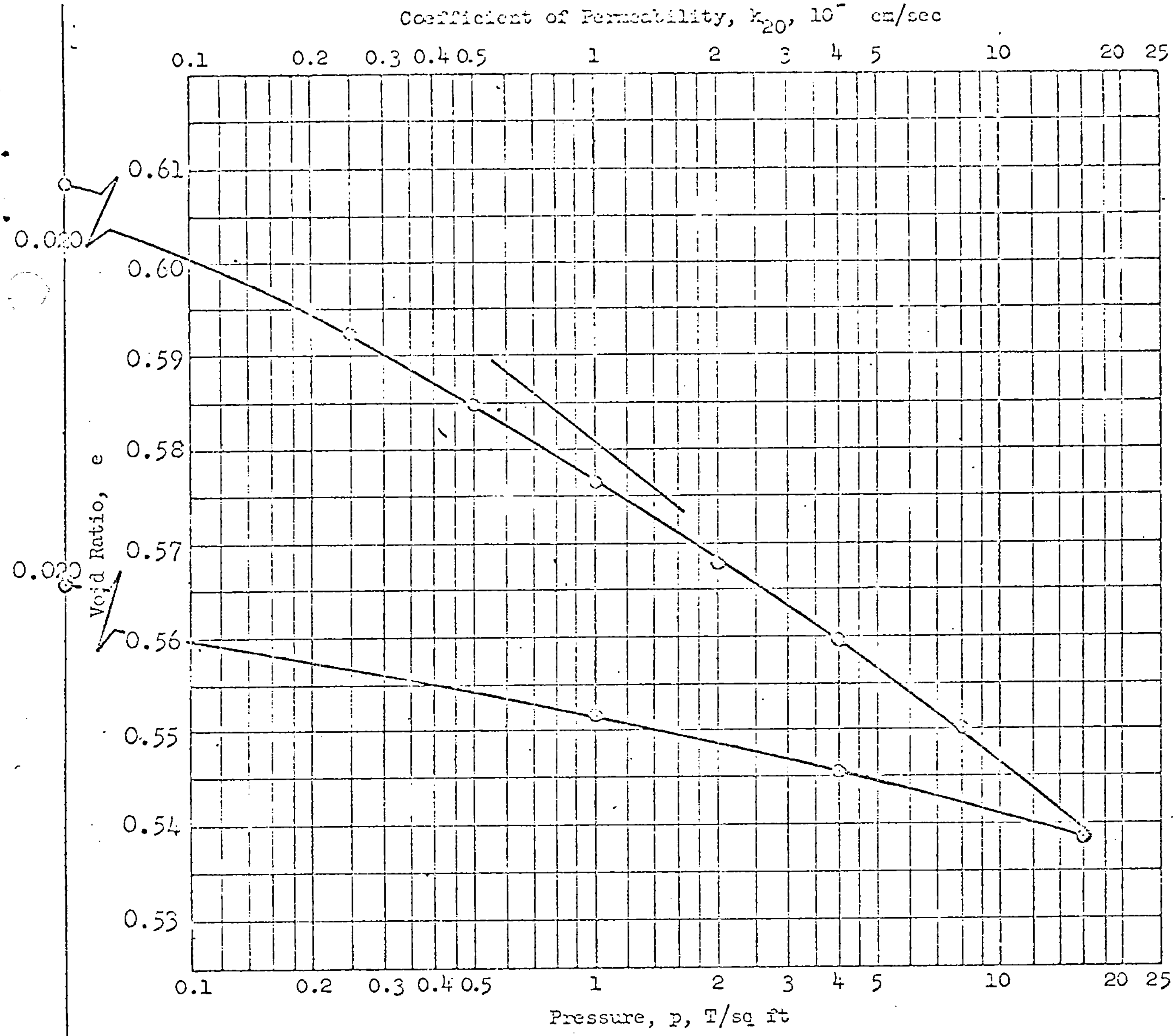
Boring No. U-2 | Sample No. 19 | Depth 63.0-65.5 | Date 10-FEB-66

ENG FORM 2086
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

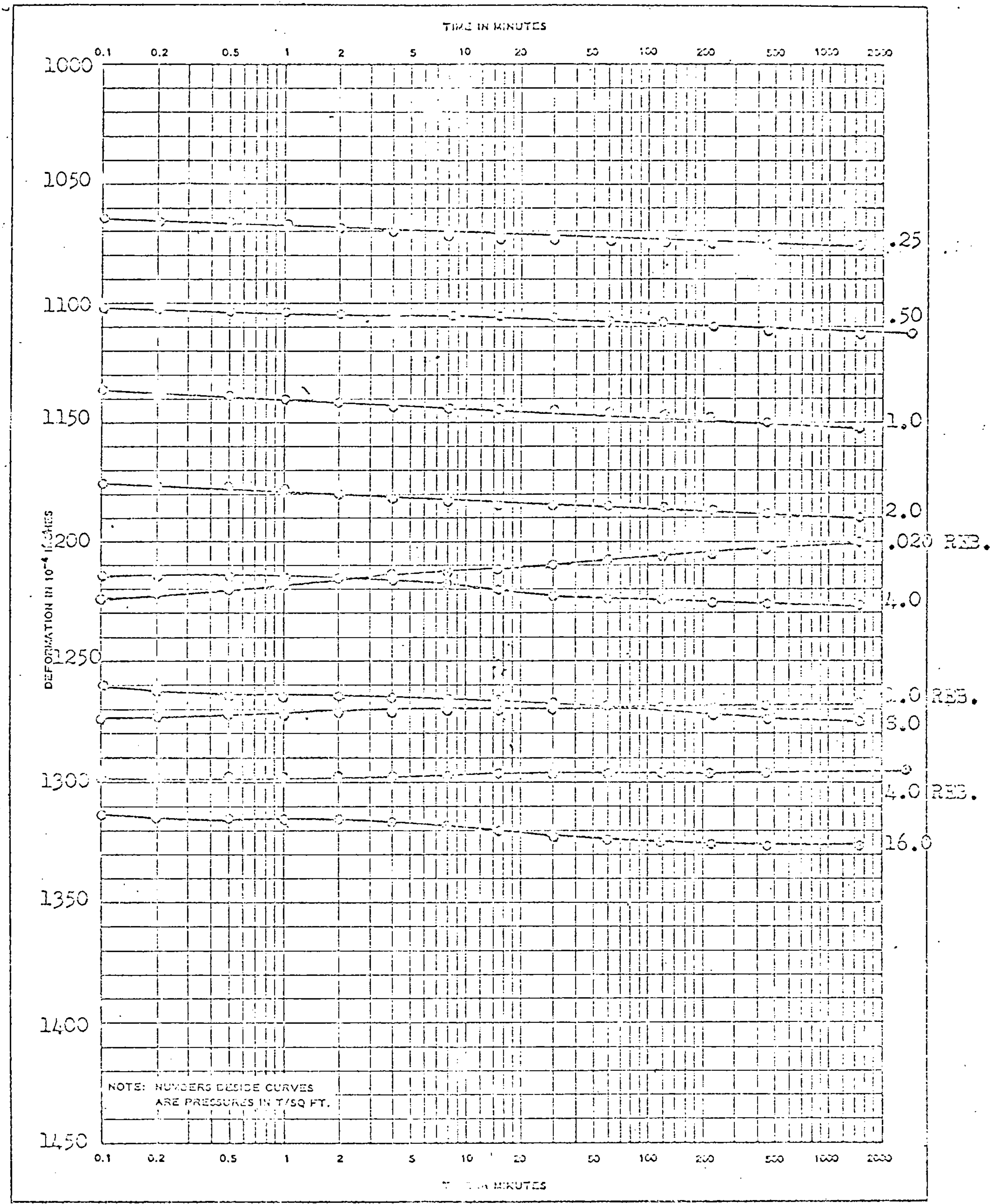
CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

FIG. A25a



Type of Specimen UNDISTURBED		Before Test		After Test
Diam 2.50 in.	Ht 0.751 in.	Water Content, w_o	20.0 %	w_f 21.0 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.609	e_f 0.566
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	87.4 %	S_f 98.7 %
Compression Index, C_c = 0.03		Dry Density, γ_d	103.1 lb/ft^3	105.9
Classification SAND(SP), brown		k_{20} at e_o = × 10^{-} cm/sec		
LL ——	G_s = 2.66	Project SEABROOK LOCK		
PL ——	D_{10}			
Remarks		Area		
		Boring No. U-2		Sample No. 27
		Depth 103.0 - 105.5		Date 10 FEB 1966
		JB CONSOLIDATION TEST REPORT		



Project SEABROOK LOCK

Area

Boring No. U-2 | Sample No. 27 | Depth 103.0-105.5 | Date 10-FEB-66

ENG FORM 2086
1 MAY 63
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES

(TRANSLUCENT)

FIG. A26a