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U. S. ARMY, CORPS OF ENGINEERS

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN

36WT, 37W, 39WU, 40W, 41W, 42W, 44W, 45W, 47W, 48W, 49W, 50WU, 51W, 52W, 53W, 64WA, 54WT, 55WU, 56W, 57W, 65WA, 46WU, 59WU 49

83-36, 37, 41, 48, 60, 50, 61, 62, 63, 64, 65

DESIGN MEMORANDUM NO. 2, GENERAL
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVENUE TO IHNC LOCK

RESEARCH CENTER LIBRARY
US ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

Prepared in the Office of the District Engineer (IHC)
New Orleans District, Corps of Engineers
New Orleans, Louisiana

March 1967

NEW ORLEANS

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no. 2
1967
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LMVED-TD (NOD 13 March 1967) 3d Ind
SUBJECT: Lake Pontchartrain, La. & Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General, Advance Supplement, IHNC West Levee - IHNC Lock to Florida Avenue

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39180 13 Jun 67

TO: District Engineer, New Orleans, ATTN: LMNED-PP

1. Referred to note approval as a basis of further planning, subject to comments of 1st and 2d Inds and to the following additional comments. References are to paras in 2d Ind.

2. Para 1a(2).

a. The third sentence should not be interpreted to mean that only a portion of the sheet pile wall would be effective to resist the upward thrust of the batter piles. Consideration, however, should be given to driving a portion of the sheet piling (say every third pile) to a greater depth to take advantage of the higher resistance developed at greater depths.

b. The type of compression piles (concrete or treated timber) as well as the type of wall selected should be based primarily on economics. Where there is little difference in cost, the preferable design based on other considerations can be selected.

3. Para 1b. Instructions for making the survey discussed in this paragraph were issued to you by our letter LMVED-TE, subject, Lake Pontchartrain, La., and Vicinity, General Advance Supplement Design Memorandum No.2, dated 24 May 1967.

4. Para 2. Results of pile load tests should be furnished LMVD in 16 copies. Advise this office by separate correspondence, ATTN: LMVED-TD the expected date of submission of these test results.

FOR THE DIVISION ENGINEER:

A. J. DAVIS
Chief, Engineering Division

ENGCW-EZ (ENGED-TP, 13 Mar 67) 2nd Ind
SUBJECT: Lake Pontchartrain, La. & Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General, Advance Supplement, IHNC West Levee - IHNC Lock to Florida Avenue

DA, CofEngrs, Washington, D. C., 20315, 31 May 1967 Ref. LMVD 8 June 67

TO: Division Engineer, Lower Mississippi Valley Division

1. Satisfactory as the basis for further planning subject to the comments of the Division Engineer and the following:

a. 1st indorsement paragraph 5:

(1) Paragraph 5b. Concur that the bending moments and stresses for I-type floodwalls should be computed using the same earth and water pressure diagrams as those used for pile penetration. An overstress in the piling is permissible for either shear strength that governs the design.

(2) Paragraph 5c. Concur that the higher I-type walls indicated may not be feasible. Redesign of the reaches with kicker piles to form a strutted I-wall, as suggested, will require longer compression piles (12" x 12" prestressed concrete) and much longer sheet piling to take the tension component. A sheet piling tension test should be made to determine proper design values for the section of piling that will be acting with the kicker pile for support. A T-type wall on concrete piles similar to the other T-type walls probably will be a better solution at these I-wall locations.

b. "Design of Structures," Paragraph 27, "Protective Measures Against Corrosion". The provisions to be provided for corrosion control are not based on definite information obtained by a survey, but upon an assumption that protection against galvanic corrosion as well as corrosion by stray direct-currents will be required. This may or may not be the case. A sound decision regarding the steps to be taken in regard to the corrosion mitigation solution for any given installation can only be made from an evaluation of information obtained from an adequate survey. The statement made in the last sentence of subparagraph 27a that the corrosion mitigation studies cannot be completed until the protective works have been constructed is not correct. An adequate survey of any given site to determine the possibilities of corrosion being a consideration can be and should be made prior to construction. The need for an adequate corrosion survey for the subject project as a whole was discussed with LMVD personnel over a year ago and it was the understanding of OCE that a representative of the Division would visit the New Orleans District and initiate the making of an adequate survey based on the basic procedures given in Civil Works Engineering Bulletin 49-19. Since to date a satisfactory survey

LMVED-TD (NOD 13 Mar 67) 1st Ind
SUBJECT: Lake Pontchartrain, La. & Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General, Advance Supplement, IHNC West Levee - IHNC Lock to Florida Avenue

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39180 13 Apr 67

TO: Chief of Engineers, ATTN: ENGCW-V/ENGCW-E

1. Subject design memorandum is forwarded for review and approval pursuant to para 17a, ER 1110-2-1150. Approval is recommended, subject to the following comments and to annotations in red on page I-5; plates III-2, IV-11, IV-19, IV-20, IV-21, IV-25, IV-26, IV-27; figures 4-23, 4-26, 4-29; and table IX-3.

2. Flyleaf map. The Parishes of St. Tammany and St. Bernard should be indicated on this map.

3. Section III, para 9, page III-3. The piezometer readings discussed in this paragraph should be presented.

4. Section III, para 15, and plates III-2 and III-3. The boring logs in the vicinity of the leveed portion of the protection from sta 34+95 to sta 44+12 indicate a silt stratum varying from about el -14.0 to el -18.0. Since the line of protection in this reach is only about 180 feet from the Florida Avenue drainage canal, the bottom of sheet piles for this reach should be extended to about el -20.0 to cut off this stratum. In design studies for connecting the protection to the siphon at Florida Avenue, the possible presence of a relatively pervious stratum in the foundation that could lead to critical uplift pressures under the ditch paving should be considered.

5. Section III, para 16a, page III-5, and Section IV, paras 12, 15, and 17, pages IV-5 and IV-6. a. The boring logs and shear strength data presented on plates III-6 through III-10 indicate that the "Q" shear strengths of the upper 30 to 40 feet of the foundation soils generally vary from about 250 psf to 400 psf. Using an available "Q" shear strength of c = 300 psf with a factor of safety of 1.5, the curves of head versus required sheet pile penetration presented in DIVR 1110-1-400, section 5, part 5, item 2 indicate that the "Q" shear strength will govern the design of the I-walls instead of the "S" shear strength in reaches where heads of about 8 feet or more occur. Based on the above, it appears that I-wall designs in the following reaches should be based on the "Q" shear strength.

Sta 49+68	to	Sta 52+94.75
Sta 59+30	to	Sta 59+49.5
Sta 59+96.75	to	Sta 64+05
Sta 64+05	to	Sta 65+39.75
Sta 106+76.5	to	Sta 107+03.5

b. The bending moments, stresses, and wall deflections for I-type floodwalls should be computed using the same earth and water pressure diagrams as those used in determining the pile penetration. For cases where the sheet piling is in clay, and the "S" shear strength is governing the design, a 1/3 overstress can be permitted in design. However, where the sheet piling is in clay and the "Q" shear strength is governing, normal stresses should be used.

c. Based on subparas 5a and 5b above, it appears that I-type floodwalls may not be feasible in the reaches indicated, with the possible exception of the reach from sta 49+68 to sta 52+94.75, due to excessive bending moments, stresses, and/or deflections. In the reaches from sta 59+30 to sta 59+49.5, sta 59+96.75 to sta 64+05, and sta 64+05 to sta 65+39.75, batter piles were provided to reduce the deflection of the wall. However, the design of the wall in these reaches neglects the effect of the batter pile on the stresses, penetration, and deflection of the sheet piling. This procedure is too conservative. It appears that in the reaches listed in subpara a above, it will be possible to use batter piles to reduce stresses and deflection in the wall or in some of the reaches a T-type wall on piles could be used. Where batter piles are used, their influence on stabilizing the wall and on the loads applied to the sheet piling should be properly considered.

6. Section III, para 20, page III-9. This paragraph indicates that where earthfilling is required near Florida Avenue, the fill will be placed "well in advance" of installation of the sheet piling and wall construction to reduce the ultimate settlement of the wall. The approximate time period being considered should be indicated. We assume that this time period will be sufficient to insure that future settlement of the wall will not exceed the one foot over build allowance in the construction grade of the wall as indicated in para 18 of section III. In this regard, the analysis used in estimating the amount of settlement of the leveed portion and the time rate of the settlement should be presented.

7. Plate III-2. The shear strength of the foundation between el -16.0 and el -44.0 is indicated as c = 300 + 7.1D. The term "D" in this equation of shear strength should be defined.

8. Figure 4-33. Consider eliminating the 1-inch tie rods and designing the hinges for the greater reactions.

9. Plates IV-19 and IV-20. Backfill adjacent to the wall should be somewhat higher than the adjacent ground to drain surface water away from the wall. Low areas should be filled to prevent ponding.

10. Plate IV-21. The rubber seal at the joint between the I-type and T-type walls should be set with about 1/4-inch deflection to insure contact and prevent possible leakage and flutter under water loading. The rubber/water stop in the typical floodwall joint should be set nearer the center of the wall stem than indicated to reduce the tendency to break off the corner. These changes can be made in the contract drawings.

11. Plate IV-22. a. Federal Spec. SS-S-00210 shown on section A-A is in error and should be corrected. The word "Bituminous" should be substituted for "Bitumastic" in the same section.

b. In section B-B, "plastic sealant" should be deleted and a Federal specification substituted therefor.

c. A simple method of introducing the sealing material should be provided. A pipe nipple welded to the top of the sleeve on each side of the piling would suffice.

12. Plates IV-23 and IV-24. a. A bottom seal plate of A441 steel or cast iron should be provided for the "J" seal to seat against.

b. Wind forces may cause the gate to sway and make closing difficult. Consideration should be given to providing top and bottom rollers on the gate with a rail embedded in the sill and a removable guide across the gate opening at the top. Force to move the gate could be provided by a pinch bar, block and tackle, or similar device.

c. Consideration should be given to grouping the piles near the gate posts so as to reduce torsional stresses in the gate sill.

13. Plate IV-25. a. The bottom seal should be set under slight compression to prevent leakage and possible seal flutter.

b. Suggest the 3/4-inch eye bolts should not be bent.

14. Plate IV-27. The use of a steel block to limit seal compression should be considered. See plate IV-25.

15. Plate IV-29. a. Since the gate will be raised by lifting at the ends, the torsional strength and rigidity should be checked.

b. Revise the hinge bearing to eliminate the bushings, and to provide a grease fitting at each block.

LMVED-TD (NOD 13 Mar 67) 1st 13 Apr 67
SUBJECT: Lake Pontchartrain, La. & Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General, Advance Supplement, IHNC West Levee - IHNC Lock to Florida Avenue

c. Provide a means of locking the gate in the raised position to prevent being blown over by the wind.

16. Table IX-3, page IX-2. The unit prices marked in red on this table should be used or the prices shown justified.

FOR THE ACTING DIVISION ENGINEER:

A. J. DAVIS
Chief, Engineering Division

1 Incl
wd 2 cy

Copy furnished:
NOD, ATTN: LMNED-PP
w/marked cy incl

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

IN REPLY REFER TO
LMNED-PP

13 March 1967

SUBJECT: Lake Pontchartrain, La. & Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General, Advance Supplement, IHNC West Levee - IHNC Lock to Florida Avenue

TO: Division Engineer, Lower Mississippi Valley
ATTN: LMVED-TD

1. The subject advance supplement is submitted herewith for review and approval in accordance with the provisions of ER 1110-2-1150 dated 1 July 1966.

2. Determinations of required pile penetrations and the structural adequacy of steel sheet piling for I-type walls included in this memorandum are based on criteria in effect prior to receipt of revisions resulting from review of LMNED-PP letter dated 1 November 1966 subject "Lake Pontchartrain, La. and Vicinity, General Design Memorandum No. 3, Chalmette Area Plan." A report on the revised criteria, their impact, and the case against retroactive application of them is contained in the 4th Indorsement to the above letter.

3. Approval of the advance supplement is recommended.

Thomas J. Bowen

THOMAS J. BOWEN
Colonel, CE
District Engineer

1 Incl (16 cys)
Adv. Supp.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2, GENERAL
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVENUE TO IHNC LOCK

STATUS OF DESIGN MEMORANDA

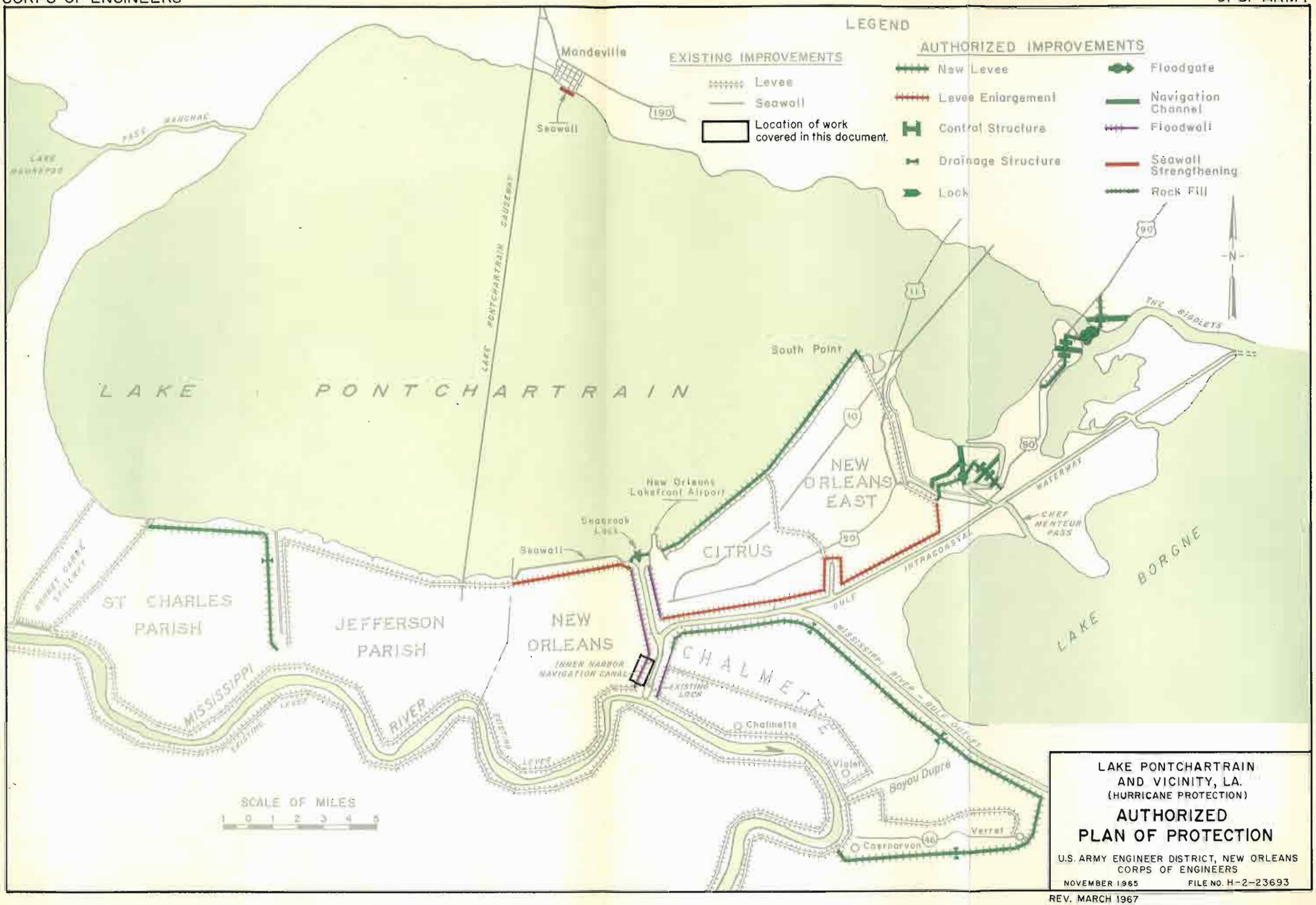
Design memo No.	Title	Status
1	Hydrology and Hydraulic Analysis	
	Part 1 - Chalmette	Approved 27 Oct 66
	Part 2 - Barrier	Scheduled Jun 67
	Part 3 - Lakefront	Scheduled Feb 68
	Part 4 - Chalmette Extension	Scheduled Aug 67
2	Lake Pontchartrain Barrier Plan, GDM, Advance Supplement, Inner Harbor Navigation Canal Levees	Submitted Mar 67
2	Lake Pontchartrain Barrier Plan, GDM, Citrus Back Levee	Scheduled Aug 67
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 1, Rigolets and Chef Menteur Complexes, Site Selection	Scheduled Mar 68
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 2, New Orleans East Back Levees	Scheduled Jul 69
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 3, Orleans Parish Lakefront Levees	Scheduled Jan 70
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 4, St. Charles Parish Lakefront Levees	Scheduled Jul 70
2	Lake Pontchartrain Barrier Plan, GDM, Supplement No. 5, St. Tammany Parish, Mandeville Seawall	Scheduled Feb 71
2	Lake Pontchartrain Barrier Plan, GDM Supplement No. 6, IHNC Remaining Levees	Scheduled Oct 67

STATUS OF DESIGN MEMORANDA (cont'd)

Design memo No.	Title	Status
3	Chalmette Area Plan, GDM	Approved 31 Jan 67
3	Chalmette Area Plan GDM Supplement No. 1 Chalmette Extension	Scheduled Feb 68
4	Lake Pontchartrain Barrier Plan & Chalmette Area Plan, GDM Florida Avenue Complex, IHNC	Not scheduled
5	Chalmette Area Plan, DDM, Bayous Bienvenue and Dupre Control Structures	Scheduled Nov 67
6	Lake Pontchartrain Barrier Plan, DDM, Rigolets Control Structure and Closure	Scheduled Jan 69
7	Lake Pontchartrain Barrier Plan, DDM, Chef Menteur Control Structure and Closure	Scheduled Apr 69
8	Lake Pontchartrain Barrier Plan, DDM, Rigolets Lock	Scheduled Jan 70
9	Lake Pontchartrain Barrier Plan, DDM, Chef Menteur Navigation Structure	Scheduled Apr 70
10	Lake Pontchartrain Barrier Plan, DDM, Gantry Crane - Chef Menteur Control Structure	Scheduled Aug 70
11	Lake Pontchartrain Barrier Plan, DDM, St. Charles Parish Drainage Structure	Scheduled Jun 71
12	Sources of Construction Materials	Approved 30 Aug 66
13	Lake Pontchartrain Barrier Plan, DDM, Gantry Crane - Rigolets Control Structure	Scheduled Feb 71
14	Beautification	Not scheduled

STATUS OF DESIGN MEMORANDA (cont'd)

Design memo No.	Title	Status
1	Lake Pontchartrain, La. and Vicinity, and Mississippi River-Gulf Outlet, La., GDM, Seabrook Lock	Scheduled Feb 68
2	Lake Pontchartrain, La. and Vicinity, and Mississippi River-Gulf Outlet, La., DDM, Seabrook Lock	Scheduled Oct 68



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. MARCH 1967

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 GENERAL
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVENUE TO IHNC LOCK

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APPENDIX I - Correspondence

PERTINENT DATA

Location of project	Southeastern Louisiana, Orleans Parish, IHNC
Hydrologic data	
Temperature: Maximum monthly	87.1° F.
Minimum monthly	43.0° F.
Average annual	69.7° F.
Annual precipitation: Maximum	85.73 inches
Minimum	31.07 inches
Average	60.58 inches
Hydraulic design criteria - tidal	
Design hurricane - Standard project hurricane (SPH)	
Frequency	1 in 200 yrs.
Central pressure index (CPI)	27.6 inches of mercury
Maximum 5-min. average wind	100 m.p.h.
Floodwall	
Linear feet	
I-type sta.	2150
T-type sta.	4900
Net grade	el. 14.0 feet m.s.l.
Rights-of-way	4.1 acres
Estimated first cost	
Federal	$2,270,000
Non-Federal	$ 970,000
Total	$3,240,000

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 GENERAL
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVENUE TO IHNC LOCK

SECTION I - GENERAL

1. Project location and description. The project "Lake Pontchartrain, La. and Vicinity," comprises two independent units--the Lake Pontchartrain Barrier Plan and the Chalmette Area Plan--and is located in southeast Louisiana in the parishes of St. Tammany, Orleans, St. Bernard, Jefferson, and St. Charles. The features of the project, as authorized, are shown on the flyleaf map. Only the Lake Pontchartrain Barrier Plan unit is pertinent to this supplement. The salient feature of the Barrier Plan is the Lake Pontchartrain Barrier, a system of embankments and structures in Orleans and St. Tammany Parishes, the purpose of which is to limit the uncontrolled entry of hurricane tides into Lake Pontchartrain while preserving navigation access. Also included in the Barrier Plan are new lakeshore levees in St. Charles Parish and the Citrus and New Orleans East areas of Orleans Parish and enlargement or strengthening of existing protective works in Jefferson and Orleans Parishes and at Mandeville, Louisiana.

2. Project authorization. Public Law 298, 89th Congress, 1st Session, approved 27 October 1965, authorized the Lake Pontchartrain, Louisiana and Vicinity, hurricane protection project, 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.

3. The report of the Chief of Engineers dated 4 March 1964 printed in House Document No. 231, 89th Congress, 1st Session, submitted for transmission to Congress the report of the Board of Engineers for Rivers and Harbors, accompanied by the reports of the District and Division Engineers and the concurring report of the Mississippi River Commission for those areas under its jurisdiction. The report of the Board of Engineers for Rivers and Harbors stated:

"...For protection from hurricane flood levels, the reporting officers find that the most suitable plan would consist of a barrier extending generally along United States Highway 90 from the easternmost levee to high ground east of the Rigolets, together with floodgates and a navigation lock in the Rigolets, and flood and navigation gates in Chef Menteur Pass; construction of a new lakeside levee in St. Charles Parish extending from the Bonnet Carre Spillway guide levee to and along the Jefferson Parish line; extension upward of the existing riprap slope protection along the Jefferson Parish levee; enlargement of the levee landward of the seawall along the 4.1-mile lakefront, and construction of a concrete-capped sheet-pile wall along the levee west of the Inner Harbor Canal in New Orleans; raising the rock dikes and landward gate bay of the planned Seabrook Lock; construction of a new levee lakeward of the Southern Railway extending from the floodwall at the New Orleans Airport to South Point; enlargement of the existing levee extending from United States Highway 90 to the Gulf Intracoastal Waterway, thence westward along the waterway to the Inner Harbor Canal, together with riprap slopes along the canal; construction of a concrete capped sheet-pile wall along the east levee of the Inner Harbor Canal between the Gulf Intracoastal Waterway and the New Orleans Airport...."

4. The report of the Chief of Engineers stated:

"...The Board [of Engineers for Rivers and Harbors] recommends authorization for construction essentially as planned by the reporting officers....I concur in the recommendation of the Board of Engineers for Rivers and Harbors."

5. Purpose and scope. This supplement presents the essential data, assumptions, criteria, and computations for developing the plan, design, and costs for the protective works for that portion of the Lake Pontchartrain Barrier Plan located on the west bank of the Inner Harbor Navigation Canal (IHNC) between Florida Avenue and the IHNC lock (see plate IV-1) and is being submitted in advance of the general design memorandum, to which it is supplementary, as a means of expediting construction in an area which proved to be highly vulnerable during hurricane "Betsy." Advance submission of a supplement covering the entire protective system for the IHNC was proposed by LMNED-PP letter dated 7 October 1965 subject "Outline of Proposed Planning Procedures for Proposed 'Lake Pontchartrain, La. and Vicinity,' Project," and approved by 1st Indorsement dated 8 December 1965 to LMNED-PP letter dated 5 November 1965, subject "Revised Outline of Planning Procedures for 'Lake Pontchartrain, La. and Vicinity,' Project." Subsequently, based on a request by the Orleans Levee District, local sponsors of

the project, the supplement coverage was reduced to further expedite construction. The reduction in coverage was proposed in LMNED-PP letter dated 8 November 1966 subject "Lake Pontchartrain, La. and Vicinity - Revised Approach to Advance Supplement on Inner Harbor Navigation Canal Levees," and approved by 1st Indorsement thereto dated 18 November 1966. Copies of the correspondence referred to in this paragraph are contained in appendix I. The design of the remainder of the protective works on the IHNC will be covered in an additional supplement to the general design memorandum, and, in addition, a feature design memorandum will be prepared covering the protective works on both banks of the canal in the vicinity of the Florida Avenue siphon crossing.

6. Local cooperation. The conditions of local cooperation, pertinent to this supplement, specified in the report of the Board of Engineers for Rivers and Harbors, and concurred in by the report of 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 works;
>
> "(3) Hold and save the United States free from damages due 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 easements 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...."

7. Investigations. Studies and investigations made in connection with the report on which authorization is based (H.D. No. 231, 89th Congress, 1st Session) consisted of: research of information which was available from previous reports and existing projects in the area; extensive research in history and records of hurricane damage and characteristics of hurricanes; extensive tidal hydraulics investigations involving both office and model studies; an economic survey; and preliminary design and cost studies. A public hearing was held in New Orleans on 13 March 1956 to determine the views of local interests. Subsequent to project authorization, detailed investigations were undertaken as follows:

a. Aerial and topographic surveys of the IHNC west levee between the IHNC lock and Florida Avenue;

b. Soils investigation including general and undisturbed type borings and associated laboratory evaluations;

c. Detailed design studies for levee, I-type and inverted T-type floodwall, and gap closures including levee section stability determinations;

d. Tidal hydraulics studies required for establishing design grades for protective works based on revised hurricane parameters furnished subsequent to project authorization by the U. S. Weather Bureau;

e. Real estate requirements and appraisals;

f. Cost estimates for levees, floodwalls, gap closures, and relocations.

8. Status of local cooperation. The conditions of local cooperation as specified by the authorizing law are quoted in paragraph 6. Essentially local interests must:

a. Provide all lands, easements, and rights-of-way required for construction;

b. Accomplish necessary alterations and relocations to existing facilities required by construction of the project;

c. Hold and save the United States free from damages due to the construction works;

d. Bear 30 percent of the first cost including the fair market value of items a. and b. above;

e. Provide an additional cash contribution equivalent to the estimated capitalized value of operating and maintaining the Rigolets lock;

f. Provide all interior drainage and pumping plants required for development of the protected areas;

g. Maintain and operate the project works in accordance with regulations prescribed by the Secretary of the Army; and

h. Acquire adequate easements to prevent encroachments on existing ponding areas and/or provide substitute storage or pumping capacity.

9. 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 responsible for the fulfillment of the conditions of local cooperation are as follows:

Mr. Leon Gary, Director
State of Louisiana
Department of Public Works
Baton Rouge, Louisiana 70804

Mr. Milton E. Dupuy, President
Board of Levee Commissioners
Orleans Levee District
Room 200, Wild Life and Fisheries Building
418 Royal Street
New Orleans, Louisiana 70130

10. Views of local interests. The Board of Levee Commissioners of the Orleans Levee District represents local interests. The plan presented herein was coordinated in detail with the Board's engineering staff and bears the approval of the Board. The estimated non-Federal contribution applicable to the work presented herein is $970,000. The intention and capability of the local sponsor to provide the required non-Federal contribution have been amply demonstrated; in fact, considerable work which will ultimately be incorporated into the overall project has already been accomplished by the sponsor.

11. Coordination with other agencies. The approval of the Orleans Levee District covers all agencies, firms, and individuals having a legitimate interest in the work covered in this supplement. General coordination for the overall Lake Pontchartrain Barrier Plan is being accomplished in connection with the preparation of the general design memorandum for that plan, and results of such coordination will be reported on in that memorandum.

12. Datum plane. Unless otherwise noted, all elevations herein are in feet referred to mean sea level datum.

13. Protective works. The plan presented herein covers all of the project works on the west bank of the IHNC between the IHNC lock and Florida Avenue, except for those required in a reach of about 116 feet (sta. 56+54 to sta. 57+70), which is occupied by a rock storage bin of the Lone Star Cement Company. The design of this reach will be covered in the supplement on the remaining IHNC levees previously referred to. The protective works covered herein consist of approximately 2,150 feet of "I"-type cantilever floodwall and 4,900 feet of inverted "T"-type floodwall. Eleven overhead roller gates and three swing gates are provided where the alignment crosses vehicular roads and railroads, and a flap gate is provided at the loading platform of the Jones & Laughlin Steel Company warehouse (sta. 63+27 to sta. 64+05). A detailed description of the protective works is presented in Section IV - Structural Design.

14. Departures from project document plan. The plan presented herein is similar to that presented in the project document. The following changes, which are considered to be within the discretionary authority of the Chief of Engineers, were adopted:

a. The net grades were revised upward one foot (13.0 to 14.0) in accordance with the results of tidal hydraulics studies utilizing the latest hurricane parameters developed by the U. S. Weather Bureau subsequent to the studies contained in House Document No. 231/89.

b. Engineering investigations and designs during the planning stage show that the use of the "sheet piling wall with concrete cap" provided in the project document plan is impracticable since the required height of the wall above the ground is in excess of six feet. Accordingly, an "I"-type floodwall was adopted where the height above ground is less than ten feet and a bearing pile supported concrete inverted "T"-type floodwall where the height above ground is greater than ten feet. In addition to its structural inadequacy, the exposed steel of the concrete capped sheet pile wall would be subject to rapid corrosion due to the highly saline water in the IHNC.

15. Costs. The estimated total first cost of the improvements covered herein is $3,240,000, comprising $2,270,000 in Federal costs and $970,000 in non-Federal costs. Detailed cost estimates are shown in Section VI.

16. Economic justification. The work covered herein is not a separable unit of the Lake Pontchartrain Barrier Plan. Economic data for the overall Lake Pontchartrain Barrier Plan will be included in

the general design memorandum for that plan. The current LMV Form 23 indicates that the benefit-cost ratio for the entire "Lake Pontchartrain, La. and Vicinity,"project, including the Lake Pontchartrain Barrier Plan and the Chalmette Area Plan, is 14.6 to 1.

SECTION II - HYDROLOGY AND HYDRAULICS

1. General. The hydrology and hydraulic analysis and design for the portion of the IHNC levee covered herein is presented in Design Memorandum No. 1, "Hydrology and Hydraulic Analysis, Part I - Chalmette," dated August 1966, which contains descriptions and analyses of the methods and procedures used in the tidal hydraulic design and covers essential data, climatology, assumptions, and criteria used, and the results of studies which provide the bases for determining surges, routings, wind tides, runup, overtopping, and frequencies.

2. Design elevations. The design hurricane for the protective works on the IHNC is the standard project hurricane (SPH) having a frequency of about once in 200 years, a central pressure index of 27.6 inches of mercury, a maximum 5-minute average wind velocity of 100 m.p.h. at 30 feet above ground level and a radius of 30 nautical miles from the center, moving on a track critical to the IHNC at a forward speed of 11 knots. Detailed information on the design hurricane is contained in the referenced D.M. No. 1. The design hurricane will produce a maximum wind tide level of 13.0 feet along the portion of IHNC in question. Waves are not a factor. One foot of freeboard was added to the wind tide level producing a net grade of 14.0.

SECTION III - FOUNDATION INVESTIGATION

GEOLOGY

1. Physiography. The project[1] area is located within the central Gulf Coastal Plain, or, more specifically, on the eastern flank of the Mississippi River Deltaic Plain. Dominant physiographic features are marshes and natural levees. Relief in the area is slight with a maximum of 6 feet between the landside slopes of the natural levee along the Mississippi River and adjacent marshlands. Maximum natural ground elevations vary from approximately 5 along the landside slopes of the natural levees of the Mississippi River to 1.5 near Florida Avenue.

2. General geology. Only the geologic history since the end of the Pleistocene period is significant for this project. At that time, with sea level about 450 feet below its present level, the project area was a flat, highland plain area bordering on the northeast of the deeply entrenched Mississippi River. During this period, the upper part of the Pleistocene was desiccated and weathered. About 5,000 years ago, sea level reached its present stand and the Mississippi River began to migrate laterally back and forth across the alluvial valley. Approximately 4,500 to 4,000 years ago, the first Recent deltaic and alluvial sediments were carried into the project area when the Mississippi River occupied the Cocodrie Course. About 3,500 years ago, the Mississippi shifted its course over to the western part of the delta and occupied the Teche Course until approximately 2,800 years ago. During the interim, the project area was subjected to erosion and subsidence. The period ended when the river shifted eastward to the LaLoutre or St. Bernard Course and sediments were once again carried into the area. A major distributary at this time was Bayou Metairie-Bayou Sauvage, trending east-northeast through New Orleans and passing just north of the project area. About 1,500 years ago, the Mississippi River abandoned the LaLoutre Course and occupied the Lafourche Course to the west. The project area was not subjected to a heavy influx of sediments again until approximately 1,200 years ago when the Mississippi River shifted its course back into the study area and occupied the present Plaquemine Course. Construction of levees along the Mississippi River has eliminated floodwaters from the region and at present no sediments are being introduced into the project area.

3. Subsidence. Progressive subsidence and downwarping of the region in the vicinity of the project area have been occurring since

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[1] The term "project" as used in this section refers only to the portion of the overall project covered by this supplement.

the end of the Pleistocene epoch. The Pleistocene surface has been downwarped towards the south and west from zero at the Pleistocene outcrop north of Lake Pontchartrain to a maximum of about 500 feet near the edge of the continental shelf, about 80 miles south of New Orleans. The overall rate of subsidence in the project area has been about 0.39 foot per century. In addition to the regional subsidence, large settlements of the ground surface have occurred in the marsh and swampland areas that have been reclaimed and drained, as a result of the shrinking of the highly organic surface soils after drainage.

4. Investigations performed. Eighteen general-type and four undisturbed borings to a maximum depth of 85 feet were made in connection with this project. In addition, borings and geologic information from other sources were available for the interpretation of the physiography, subsurface, and foundation conditions of the area.

5. Foundation conditions. The subsurface, as shown on plate III-1, consists of Recent deposits varying in thickness from 60 to 70 feet overlain by 6 to 16 feet of fill material. The Recent deposits are underlain by Pleistocene deposits (Prairie Formation). Generally, the Recent consists of a discontinuous layer of soft to stiff natural levee clays underlain by very soft marsh clays with organic matter and peat. Underlying the marsh and natural levee deposits are very soft to soft interdistributary clays with lenses and layers of silt and sand. Estuarine deposits of sand, clay, and silt with shell fragments underlie the interdistributary deposits and lie unconformably on top of the Pleistocene deposits.

6. Mineral resources. Oil and gas production are not found in the immediate vicinity of the project. Exploration and production may eventuate in the area. Such development, if it occurs, will not be adversely affected by the recommended protective works.

7. Sources of construction materials. Sources of construction materials are covered in Design Memorandum No. 12, "Sources of Construction Materials," dated June 1966.

8. Conclusions. Because of the low shear strength of some of the Recent materials and because of the compressibility of some of these sediments, stability and settlement are major problems in the area. In addition, the existence of large sand and silt layers results in seepage and uplift problems.

SOILS AND FOUNDATION INVESTIGATIONS AND DESIGN

9. Field investigations. Four 5-inch diameter undisturbed soil borings and eighteen 1-7/8-inch I.D. general-type core soil borings were made along the project alignment. The borings were made at intervals varying from about 100 to 600 feet along the project location. The borings extended in depth to elevations -48 to -75. Locations of the borings are shown on plates IV-1 through IV-6. Four piezometers were installed on a range located at the floodwall centerline station 43+37 to obtain existing pore pressures in the foundation clays for estimating residual settlement beneath the fill material (see plate IV-10). Two piezometers are located beneath the existing levee crown and two beneath the existing ground surface 100 feet canalward of the levee. The upper piezometer tips are in the clayey marsh deposit and the lower piezometer tips are in the clayey interdistributary deposit. These piezometers were read daily to determine existing piezometric conditions. One 5-inch diameter undisturbed soil boring and seven 1-7/8-inch I.D. general-type soil borings were made along an alternate alignment later rejected. The locations are shown on plate IV-1. Logs of the general-type borings are shown on plate IV-8. Test data for the undisturbed boring (WU-59) are shown on plate III-10.

10. Laboratory tests. Visual classifications were made on all samples obtained from the soil borings. Water content determinations were made on all cohesive soil samples. Consolidation (C) tests, Unconfined Compression (UC), Unconsolidated-Undrained (Q), Consolidated-Undrained (R), and Consolidated-Drained (S) shear tests were performed on representative soil samples taken from the undisturbed borings. Liquid and plastic limits were determined for all cohesive samples on which consolidation and shear tests were performed. The locations and results of the soil tests on the undisturbed borings are shown on plates III-6 through III-14, and those for the general-type borings are shown on plates IV-7 and IV-8.

11. Foundation conditions. The subsurface along this project consists generally of 6 to 16 feet of artificial fill overlying 60 to 70 feet of Recent deposits of clays, silts, and sands which are underlain by the Pleistocene soils that were encountered at elevations -75 at the north end and -65 at the south end of the project. A generalized soil and geologic profile along the baseline is shown on plate III-1. The portion of the subsurface soils above the Pleistocene deposit which directly affects the design of this project consists generally of the following: (Stationing referred to is along the floodwall centerline.)

a. Station 34+95 (Florida Avenue end of the project) to station 70+00. Predominately clay fill exists down to elevations varying from -5 to -10 underlain by a 3- to 9-foot layer of organic clay extending down to elevations varying from -15 to -20. Beneath the organic clay is a 22- to 30-foot layer of fat clay with some silt strata near its top. The fat clay is underlain by a fine sand deposit extending from about elevation -40 to elevation -75, where the top of the Pleistocene formation was encountered.

b. Station 70+00 to station 100+00. Clay predominates down to elevations varying from -3 to -10 underlain by a 3- to 8-foot layer of organic clay extending down to elevations varying from -15 to -20. Beneath the organic clay there is a 26- to 40-foot layer of fat clay, with some silt strata near its top, extending down to approximate elevation -45. The fat clay layer is underlain by a deposit containing layers of fat and lean clays and fine sands extending down to approximate elevations -65 to -75, where the top of the Pleistocene formation was encountered.

c. Station 100+00 to station 108+00 (lock end of project). Except for a 3- to 4-foot surface layer of silty sand, clay predominates down to approximate elevation -18 where a 3- to 4-foot layer of silt overlies a clay layer extending down to approximate elevation -48. The clay layer is underlain by a clay deposit containing a 2- to 3-foot layer of fine sands at its top and extends down to approximate elevations -68 to -75, where the top of the Pleistocene formation was encountered.

d. Water content of soils. The clays in the fill material have water contents varying from about 24 to 88 percent. The water contents of the natural levee deposit vary from about 50 to 70 percent. The water contents of the organic clays range from about 91 to 488 percent depending on the organic content. The Recent clays between approximate elevations -20 to -45 have water contents varying from about 50 to 80 percent. Below approximate elevation -45, the clays have water contents varying from about 40 to 60 percent. The Pleistocene clays have water contents of about 30 percent.

12. Design and construction problems. Because of the low shear strengths and compressible nature of the Recent foundation clays, the numerous gated structures, and the limited space available for the construction of the required protection, the following are the principal design and construction problems arising in connection with this project:

a. Type of protection

b. Location of protection

c. Seepage

d. Stability of levee and cantilever sheet pile walls

e. Bearing pile type and penetrations

f. Settlement

g. Sources of fill material

h. Methods of construction

13. Type of protection. Because of the limited space available due to the proximity of roads, railroads, and existing industrial plant facilities, and the necessity for providing protection against seepage and potential erosion, the protective works will consist of cantilever "I"-type floodwalls of steel sheet piling capped with a concrete wall where the wall height is less than 10 feet, and "T"-type concrete floodwalls, with steel sheet pile cutoffs,supported by 12-inch by 12-inch square prestressed concrete bearing piles where the wall height is more than 10 feet.

14. Location of protection. The protection works are located so that they will preserve and not interfere with, insofar as practicable, existing roads, railroads, and industrial plant facilities. The alignment of the protective works is shown on plates IV-1 through IV-6. Specific data relative to location and type of protection are listed in table III-1.

15. Seepage. The steel sheet piling associated with the "I" and "T" walls and gated structures will provide protection against hazardous seepage. The minimum depth of cutoff was that required to penetrate the upper marsh deposit, and where the "I" wall sheet pile penetration required for stability did not meet the requirement for cutoff, the necessary extension was made. (See plates III-2, 3, & 4.)

16. Stability.

a. Floodwalls. Cantilever "I"-type floodwalls in levee fill were designed for the following loading conditions: top of wall at elevation 15.0; water level on the floodside 6 inches below the top of the wall (1.5 feet above stillwater level at elevation 13.0); and ground water on the protected side at elevation 0.0. The

TABLE III-1
Location and Type of Protection

Station location From	To	Elevation m.s.l. Levee crown	"I"-wall Top	"I"-wall Sheet pile tip	"T"-wall Top	"T"-wall Sheet pile tip	Stability plate number III-
34+95	38+65	9.0	15.0	-14.0			2 & 3
38+65	39+75				14.0	-14.0	-
39+75	44+12	9.0	15.0	-14.0			2 & 3
44+12	49+68				14.0	-15.0	-
49+68	51+07		14.5	-22.0			4
51+07	51+65.75				14.0	-22.0	-
51+65.75	52+94.75		14.5	-22.0			4
52+94.75	53+41				14.0	-20.0	-
53+41	55+74		14.5	-20.0			4
55+74	56+19				14.0	-20.0	-
56+19	56+54		14.5	-20.0			4
56+54	57+70*						
57+70	58+25.75		14.5	-20.0			4
58+25.75	58+69.75				14.0	-20.0	-
58+69.75	59+30		14.5	-20.0			4
59+30	59+49.5		14.0	-36.0			4
59+49.5	59+96.75				14.0	-25.0	-
59+96.75	64+05		14.5	-36.0			4
64+05	65+39.75		14.5	-29.0			4
65+39.75	106+76.5				14.0	-20.0	-
106+76.5	107+03.5		14.5	-29.0			4
107+03.5	107+24.5		14.5	-23.0			***
107+24.5	107+66.5		14.5	-21.0			***
107+66.5	107+75.5		14.5	-14.5			***
107+75.5	107+87		14.5	-6.5			***
107+87	108+01**		13.5	-6.5			

*No work--existing rock storage bin.
**Sheet pile only.
***Separate stability analyses not made. Tip elevation estimated from analyses in other areas.

remaining "I"-type walls, with top at elevation 14.5, were designed with water 6 inches below top on floodside (1.0 foot above stillwater level at elevation 13.0), and ground water on the protected side at elevation 0.0. In the vicinity of the Chase Bag Company warehouse (plate III-3), an "I"-type wall analysis was performed for a reverse loading condition on the protected side due to a 200 p.s.f. load on the warehouse platform, with ground water at elevation 0.0 on both sides of the wall. The stability and required penetrations of the steel sheet piles below the earth surface at the piles were determined by the method of planes using the (S) shear strengths shown on the stability plates. In determining the minimum penetration required for stability, a factor of safety of 1.5 was applied to the design shear strengths as follows: ϕ developed = $\tan^{-1}\left(\frac{\tan\phi}{f.s.}\right)$. Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these distributions of pressure, summations of horizontal forces were equated to zero for various tip penetrations. At these penetrations, summations of overturning moments about the bottom of the sheet pile were determined. The required depths of penetration were determined as those at which summation of moments were equal to zero. These analyses are shown on plates III-3 and III-4. Using the required penetrations shown on the stability sections, factors of safety were also determined for the water surface at the top of the walls. These factors of safety are shown by note on the stability sections. The foregoing procedures also were used in determining the penetrations and loading diagrams for analyzing the structural member by applying a factor of safety of 1.0 to the (S) soil shear strengths.

b. Levees. Using sections representative of existing conditions along the leveed portion of the wall alignment, the slopes and berm distances for the recommended levee were designed with the "I"-type wall in place for a hurricane water condition with water to elevation 14.5 on the flood side and varying from elevation 0.0 to -6.0 on the protected side, as shown on plate III-2, with assumed failure toward the protected side. The stability of the levee was determined by the method of planes using the design (Q) shear strengths and assigned piezometric conditions shown on the stability section. A design levee section was determined for a minimum factor of safety of 1.3 based on the (Q) shear strengths. This analysis is shown on plate III-2.

17. Foundations for structures. 12-inch by 12-inch prestressed concrete piles will be used to support the "T"-type walls and gated structures. Design bearing and tension capacities versus tip elevations were determined for four representative foundation conditions along the project alignment. Design data were determined for the

(Q) and (S) shear strengths, disregarding the skin friction above the bottom of the Recent marsh deposit. In compression, a factor of safety of 1.75 was applied to the shear strengths and a conjugate stress ratio (K_o) = 1.0 was used in determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strength and a K_o = 0.7 was used. The results are shown on plate III-5. Steel sheet pile seepage cutoffs will be provided beneath the "T"-type walls and gated structures. Bearing pile tip elevations used for cost estimating purposes are shown on plates IV-19 and IV-20. Prior to construction, three 12-inch by 12-inch precast prestressed concrete piles of different lengths will be driven at each of the three locations shown on plates IV-3, IV-5, and IV-6. At each site, the short pile and the intermediate pile will be tested in compression. If test results show that either of these two piles can safely carry twice the design loads, the long piles will not be tested. One pile at each site will be tested in tension. The test sites will be in the vicinity of boring 39WU (site 1), boring 51W (site 2), and boring 57W (site 3). The elevations of the tips of the test piles will be located generally as follows: site 1, -38, -44, and -54; site 2, -42, -52, and -62; and site 3, -45, -51, and -61.

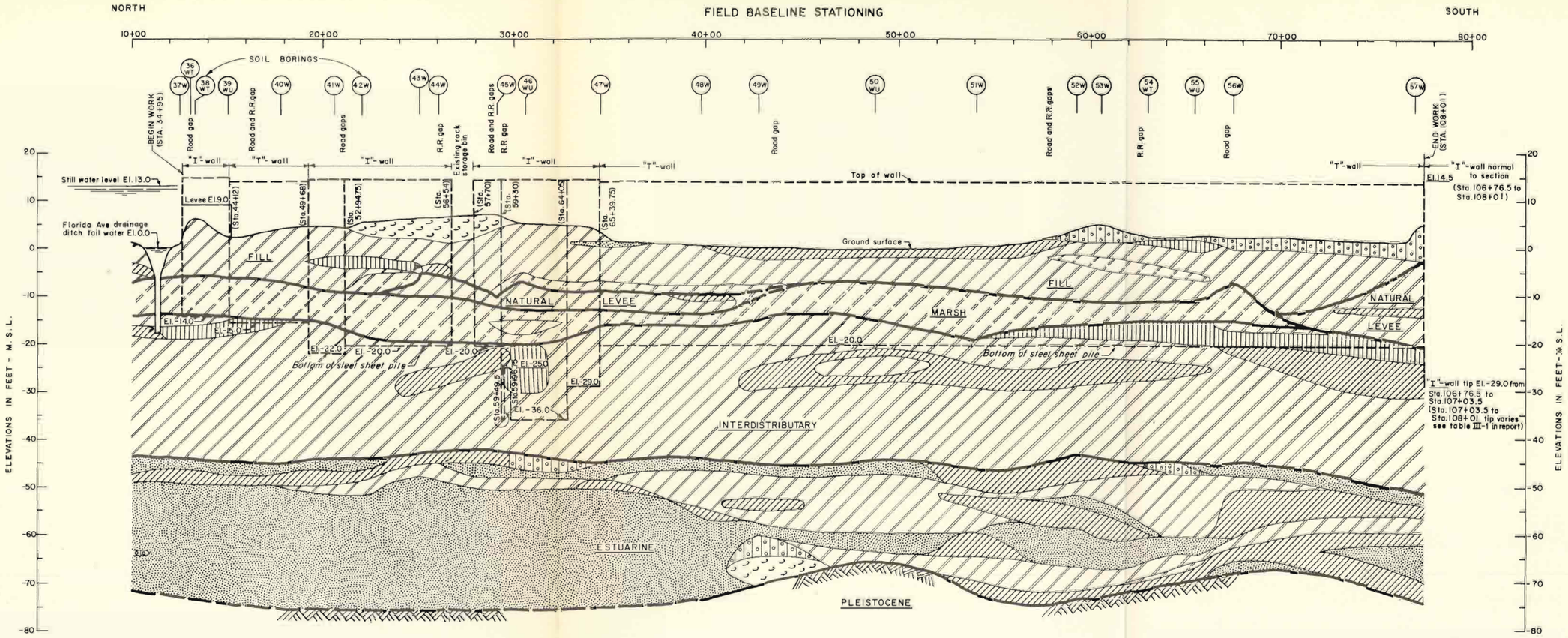
18. Settlement. Based on the foundation conditions determined from the soil borings, consolidation test data, and existing piezometric pressure conditions indicated by the piezometers, estimates of settlement were made along the line of protection. Along the leveed portion, estimates indicate that the proposed wall construction grade of 15.0 is sufficient to provide ultimate protection to the net grade of 14.0. Except for minor local settlement, and provided that the existing fill is not raised, data indicate that the foundation along the remainder of the project alignment is normally loaded and settlement is not a major problem. However, the "I"-type walls in this reach will be constructed to elevation 14.5 to compensate for possible minor settlement.

19. Sources of fill material. After reshaping the existing fill along the leveed portion of the project, additional fill, consisting of stiff Pleistocene clays, for completing levees to design grade and section, will be obtained from a borrow area in the bottom of Lake Pontchartrain along its north shore and barged to the construction site, inasmuch as satisfactory borrow is not available in the immediate vicinity of the project.

20. Methods of construction. Where earthfilling is required along the leveed portion of the alignment near Florida Avenue, the fill will be placed using semicompacted methods well in advance of installation of the steel sheet piling and wall construction to reduce the ultimate settlement of the wall. For a distance of 50 feet on each side of the "T"-type walls, in areas where "T"-type walls tie into levees incorporating "I"-type walls, the earthfill will be placed and the "I"-type wall constructed well in advance of the "T"-type construction, in order to reduce the effect of negative skin friction on the bearing piles caused by settlement of the earth foundation beneath the "T"-walls adjacent to the newly constructed levee.

21. Erosion protection. Due to the short duration of hurricane floods, the inherent resistance of the clayey soils to erosion, and the limited opportunity for wave generation, no surface erosion protection is considered necessary along the protection line.

22. Settlement observations. Settlement observations will be made along the "I"-type walls located in levees after the levee is completed; after the wall is constructed; and yearly thereafter until settlement has essentially ceased. Settlement observations will be made on the remaining walls and gated structures after the concrete portion is constructed and after each hurricane flood thereafter.



LEGEND

CH - Fat clay	MARSH - very soft clays with organic matter and peat
CHO - Fat clay with organic matter and peat	NATURAL LEVEE - soft to stiff clays with lenses and layers of silt
CL - Lean clay	INTERDISTRIBUTARY - very soft to soft clays with lenses and layers of silt and sand
ML - Silt	ESTUARINE - sand, clay and silt with shell fragments
SM - Silty sand	PLEISTOCENE - stiff to very stiff clays
SP - Fine sand	
SI - Shells	
Pleistocene Horizon	

NOTE:
Stationing in brackets indicates floodwall and ℄ stationing.
See plates III-6 thru III-10 for undisturbed boring logs and plates IV-7 and IV-8 for general type boring logs.
See plates IV-2 thru IV-6

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

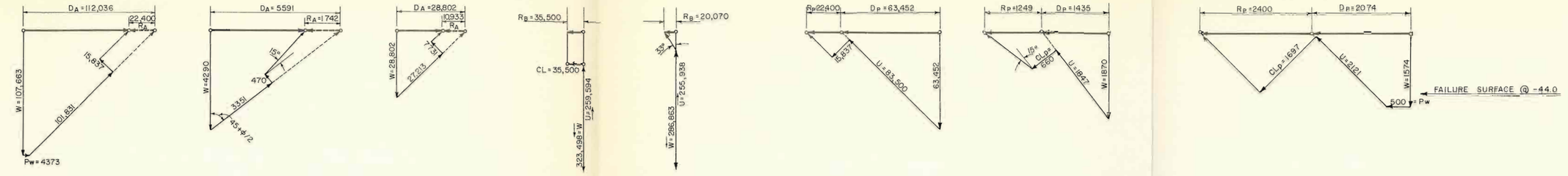
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

GENERALIZED SOIL AND GEOLOGIC PROFILE

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

FEBRUARY 1967 FILE NO. H-2-23909

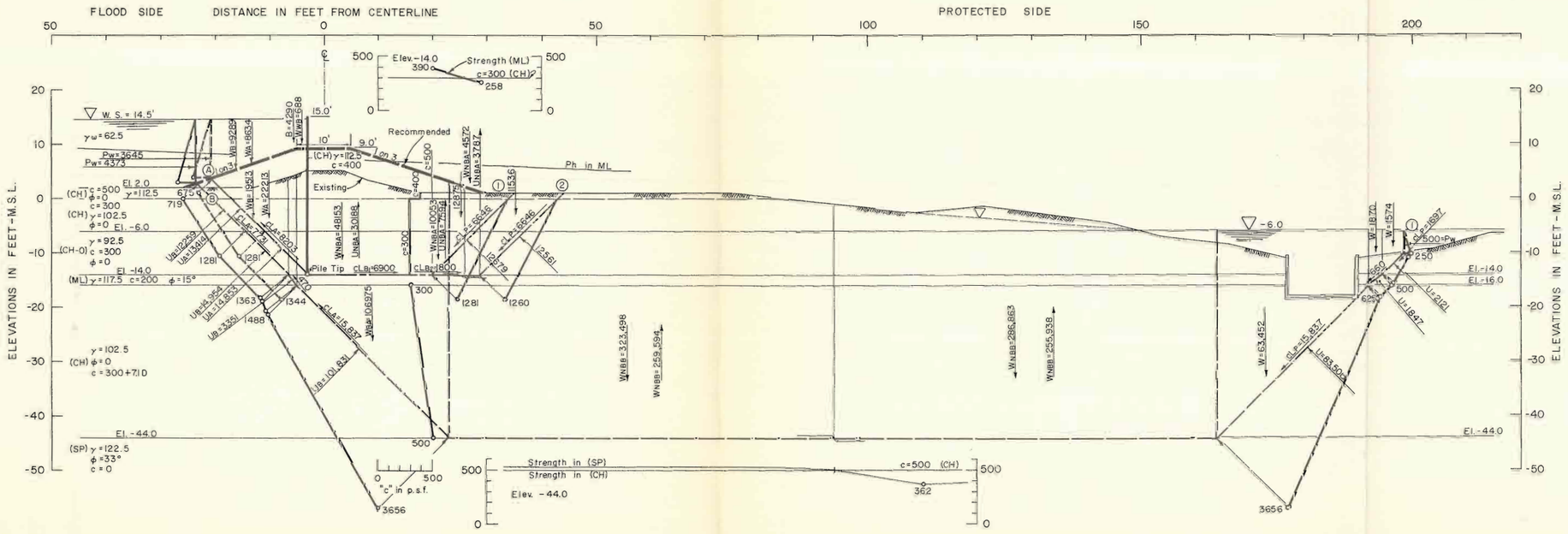
PLATE III - 1



STABILITY CALCULATIONS

LEVEE FEATURE	SLIP SURFACE			DRIVING			RESISTING				FACTOR OF SAFETY
	NO.		EL.	$+D_A$	$-D_P$	ΣD	$+R_A$	$+R_B$	$+R_P$	ΣR	$\Sigma R/\Sigma D$
STA. 34+95 STA. 38+65 AND STA. 39+75 STA. 44+09	A	1	-14.0	34,700	12,857	21,843	11,600	6,900	9,400	27,900	1.28
		2			11,356	23,344		9,508	9,400	30,508	1.31
	B	1	-44.0	146,429	66,961	79,468	35,075	55,570	26,049	116,694	1.47 *

NOTE: * This factor of safety was computed using the full head at Elev. 14.5' in the sand strata below Elev. -44.0'.

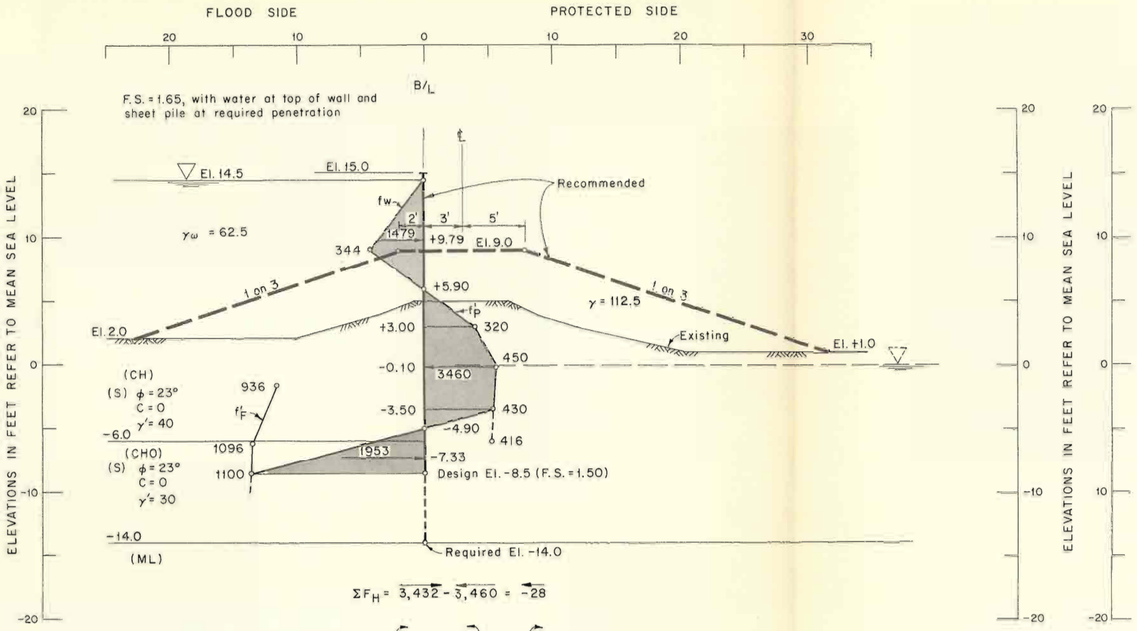


GENERAL NOTES

(Q) - Unconsolidated-Undrained shear strength in lbs. per sq. ft.
(γ) - Unit weight of soil water system in pounds per cu. ft.
(γ') - Submerged unit weight in pounds per cu. ft.
(φ) - Angle of internal friction in degrees.
R - Horizontal resisting force in pounds
D - Horizontal driving force in pounds
F.S.- Factor of safety with respect to (Q) shear strength.
U.E.- 5" Undisturbed soil boring.
γw - Unit weight of water in p.c.f.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
LEVEE (Q) STABILITY
STA. 34+95 TO STA. 38+65 AND
STA. 39+75 TO STA. 44+12
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

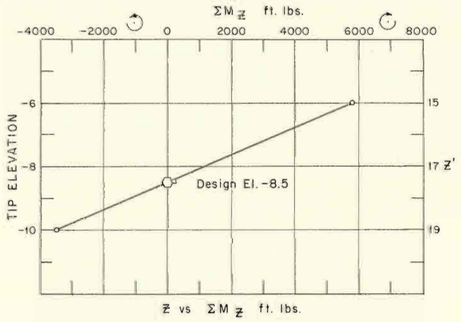
PLATE III-2



$\Sigma F_H = \overrightarrow{3,432} - \overleftarrow{3,460} = \overleftarrow{-28}$

$\Sigma M_{\bar{z}} = 29,355 - 29,064 = 291$ ft. lbs.

NET PRESSURE DIAGRAM

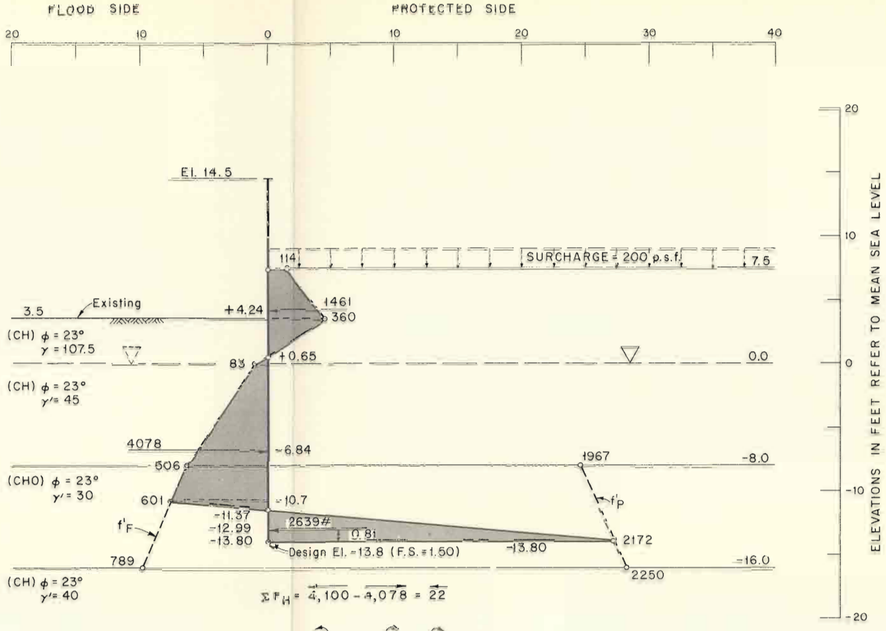


IN LEVEE-STA. 34+95 TO STA. 44+12

GENERAL NOTES

(S) - Consolidated-drained shear strength.
Cantilever sheet pile stability by method of planes analysis.

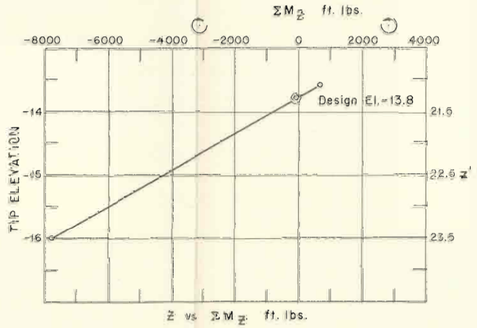
f_w - Lateral water pressure.
f'_F - Net lateral pressure on flood side, earth and water.
f'_P - Net lateral pressure on protected side earth and water.
γ_w - Unit weight of water in p.c.f.
γ - Unit weight of soil-water system in p.c.f.
γ' - Submerged unit weight of soil in p.c.f.
c - Unit cohesion in p.s.f.
ΣF_H - Summation of horizontal forces in pounds.
ΣM_z - Summation of moments about tip, in foot-pounds.
$\bar{z}$ - Penetration of sheet pile below levee crown.
F.S. - Factor of safety with respect to (S) shear strength, $\phi' = \tan^{-1}\left(\frac{\tan\phi}{F.S.}\right)$.
ϕ - Angle of internal friction in degrees.



$\Sigma F_H = \overrightarrow{4,100} - \overleftarrow{4,078} = \overrightarrow{22}$

$\Sigma M_{\bar{z}} = 28,275 - 28,383 = -108$ ft. lbs.

NET PRESSURE DIAGRAM
(REVERSE HEAD LOADING)

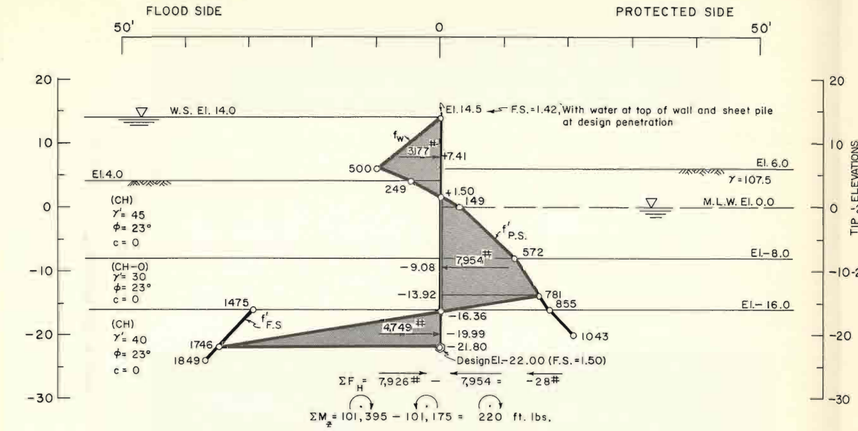


VICINITY OF CHASE BAG CO.-STA. 49+68 TO STA. 51+07

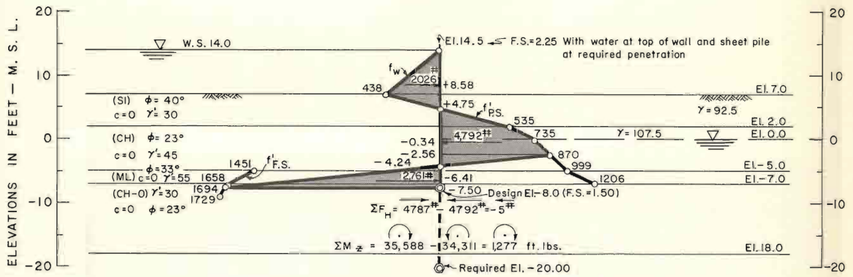
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

CANTILEVER SHEET PILE (S) STABILITY

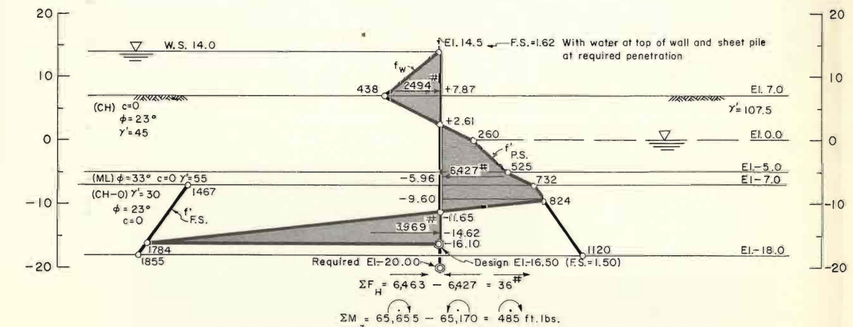
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909



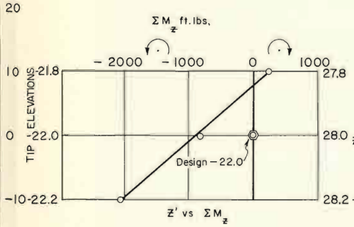
VIC. CHASE BAG CO – STA. 49+68 TO STA. 52+94.75
(EXCEPT GATE)



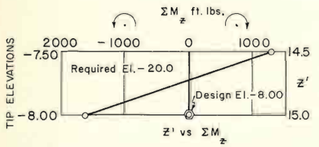
VIC. LONE STAR CEMENT CO. – STA. 53+41 TO STA. 55+74



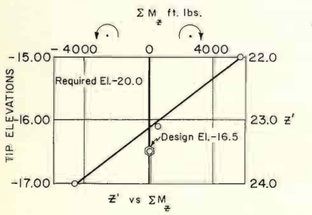
VIC. LONE STAR CEMENT CO. – STA. 56+19 TO STA. 59+30



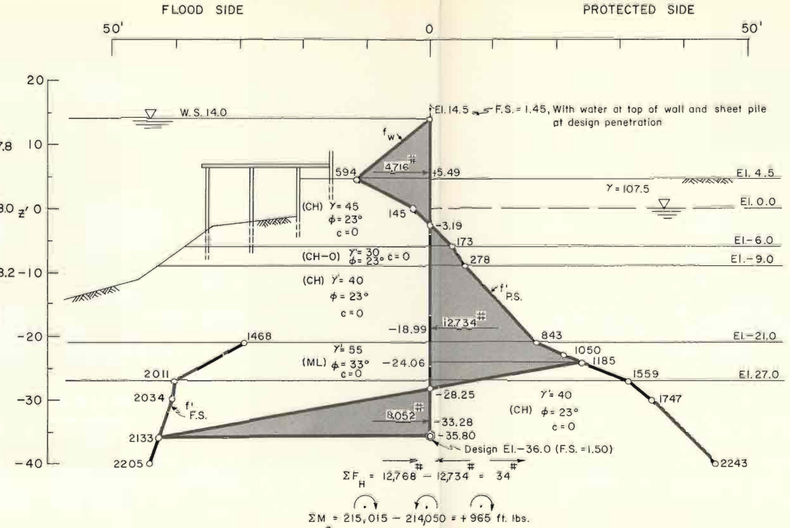
STA. 49+68 TO STA. 52+94.75
VIC. CHASE BAG CO.
(EXCEPT GATE)



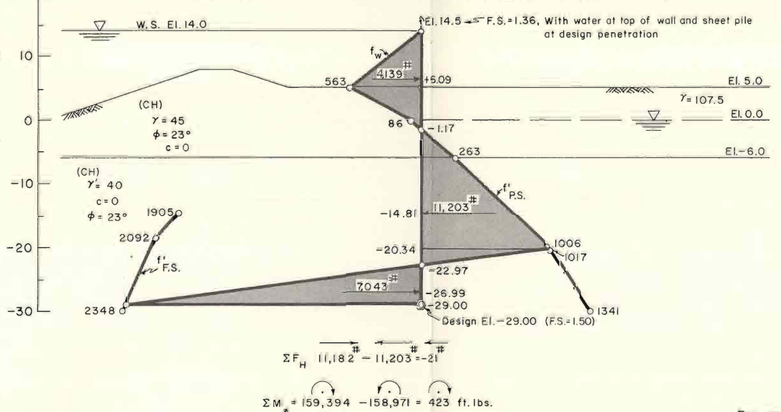
STA. 53+41 TO STA. 55+74
VIC. LONE STAR CEMENT CO.



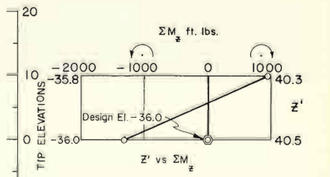
STA. 56+19 TO STA. 59+30
VIC. LONE STAR CEMENT CO.
(EXCEPT ROCK STORAGE BIN AND GATE)



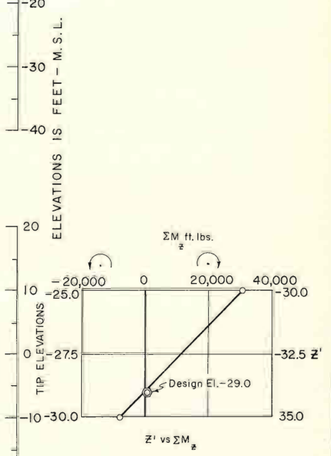
VIC. J.&L. STEEL CO. – STA. 59+30 TO STA. 59+49.5
STA. 59+96.75 TO STA. 64+05



VIC. J.&L. STEEL CO. – STA. 64+05 TO STA. 65+39.75
VIC. I. H. N. C. LOCK – STA. 106+76.5 TO STA. 107+03.5



STA. 59+30 TO STA. 59+49.5
STA. 59+96.75 TO STA. 64+05
VIC. J.&L. STEEL CO.



STA. 64+05 TO STA. 65+39.75
VIC. J.&L. STEEL CO.
STA. 106+76.5 TO STA. 107+03.5
VIC. I. H. N. C. LOCK

For general notes see plate III-3.

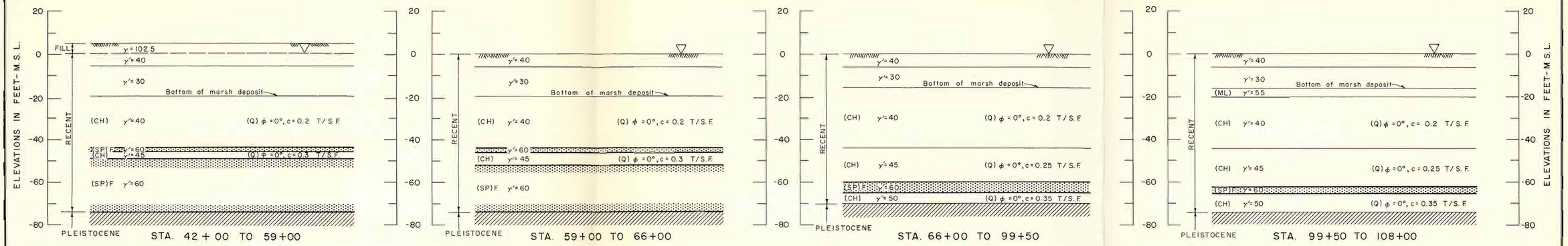
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

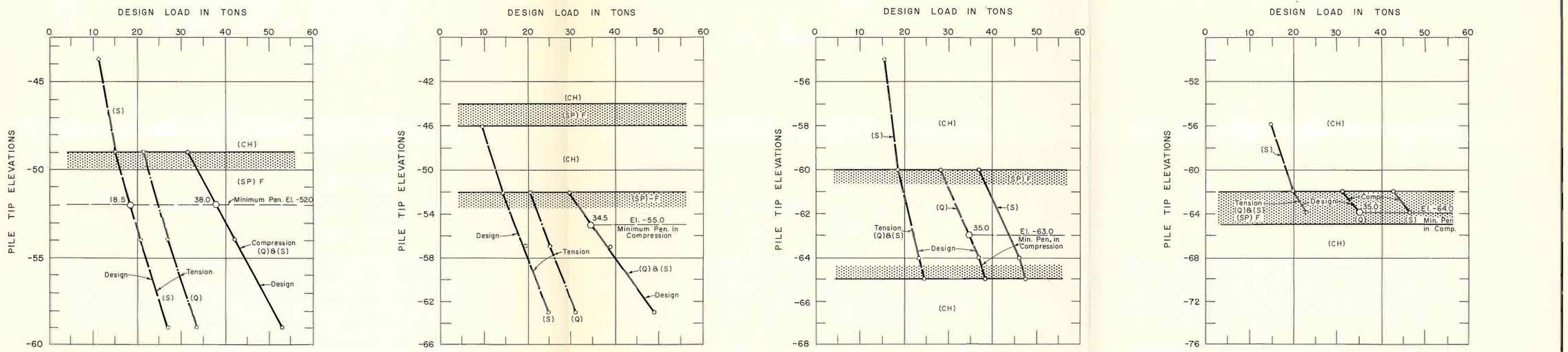
CANTILEVER SHEET PILE WALL (S) STABILITY

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

FEBRUARY 1967 FILE NO. H-2-23909



SOIL SECTIONS



DESIGN LOAD VS TIP ELEVATION

NOTE: Skin friction disregarded above bottom of marsh deposit.

SHEAR STRENGTH DESIGN DATA

Applied factors of safety: 1.75 in compression and 2.0 in tension.
Applied conjugate stress ratios - K: 1.00 in compression and 0.7 in tension.
(S) Strength in clays: $\phi = 23°$, $c = 0$, (Q) & (S) strengths in sands: $\phi = 33°$, $c = 0$.
(Q) Strengths in clays are as shown in soil sections.

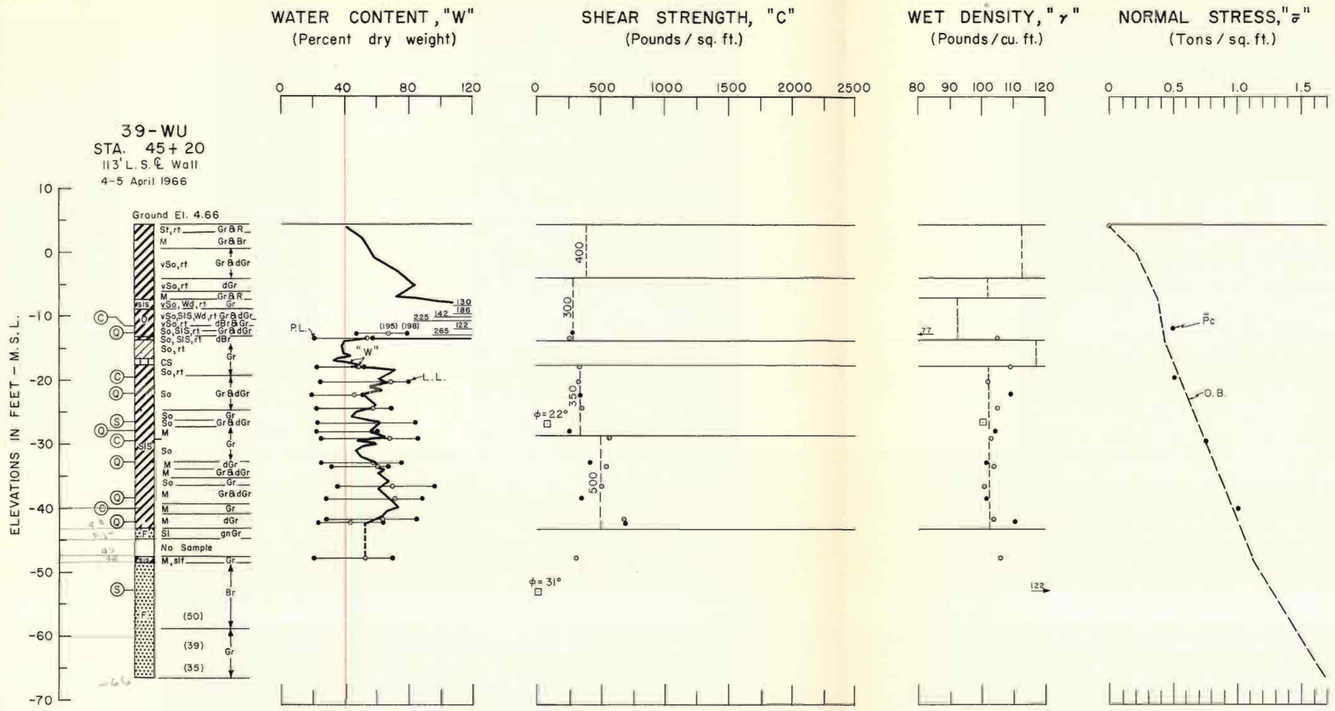
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

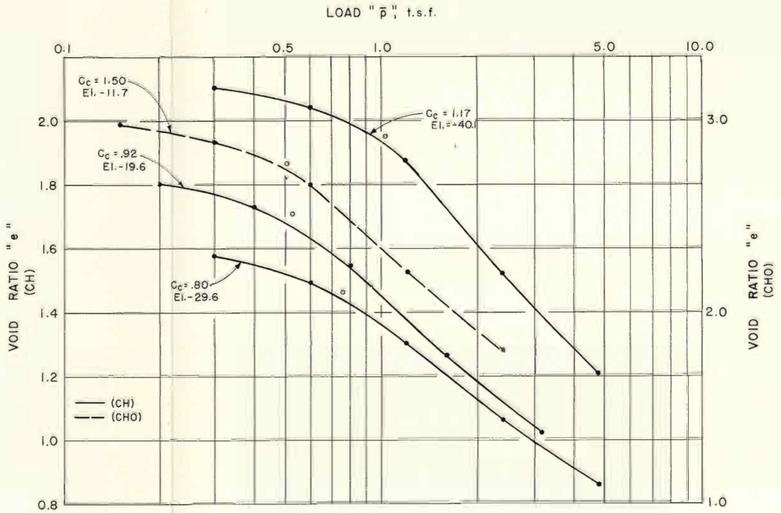
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

PILE DESIGN LOAD VS TIP ELEVATION
(Q) AND (S) CASES FOR 12" x 12" SQUARE PRESTRESSED CONCRETE PILES

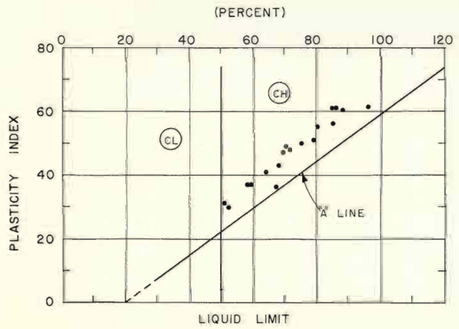
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-23909

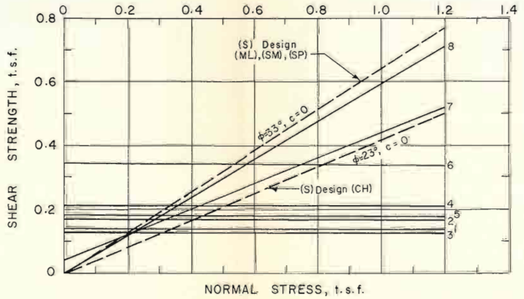




CONSOLIDATION DATA



PLASTICITY CHART



ENVELOPE No.	ENVELOPE El.	TYPE	STRENGTH φ	STRENGTH c (t.s.f.)	CLASS
1	-12.4	Q	0	0.14	CH
2	-22.0			0.17	CH
3	-27.7			0.13	CH
4	-32.7			0.21	CH
5	-38.3			0.18	CH
6	-42.0			0.34	CH
7	-26.3	S	22	0.04	CH
8	-52.7		31	0.00	SP

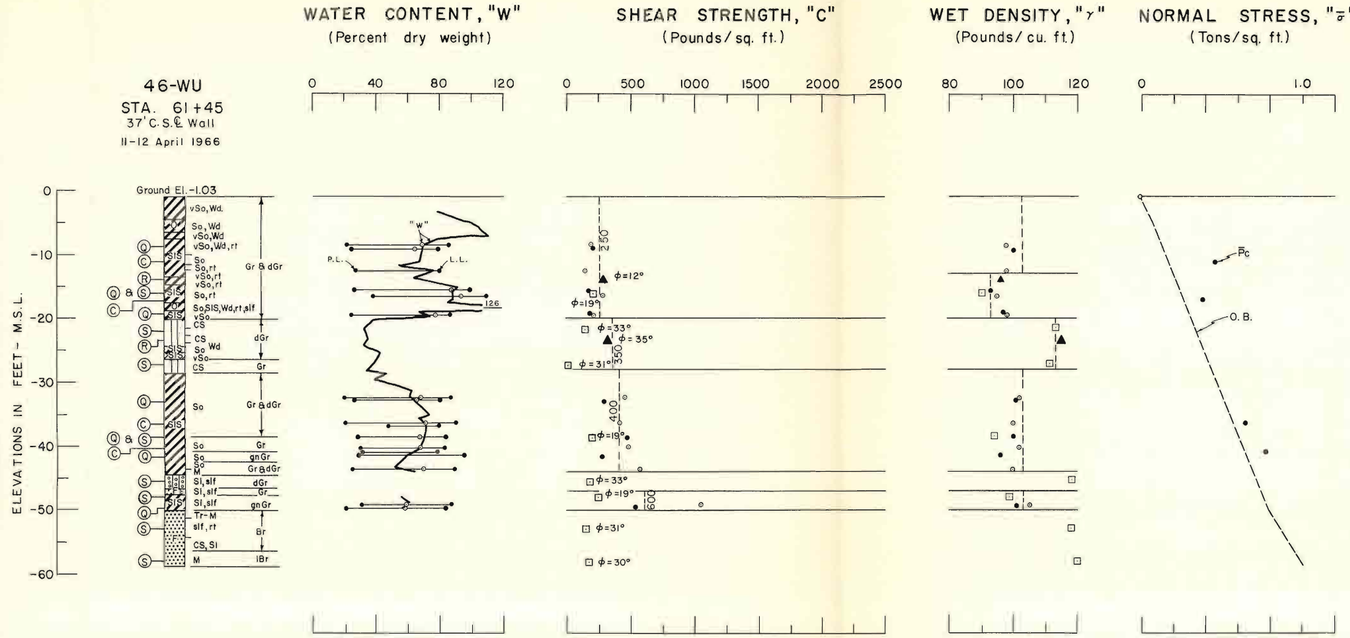
SHEAR STRENGTH DATA

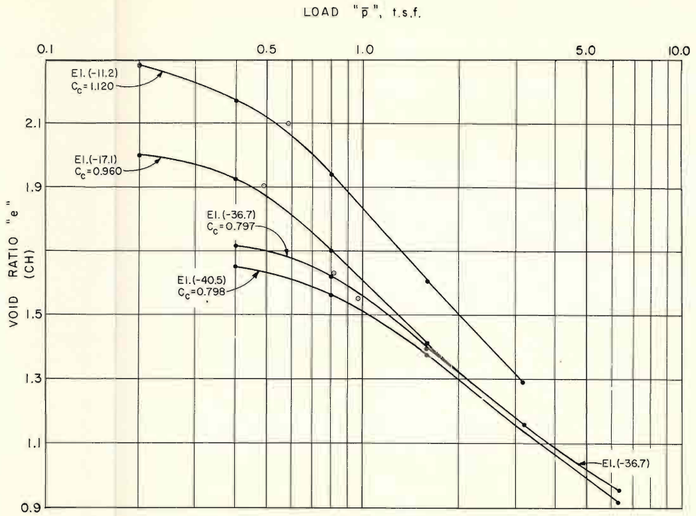
See Plate A for soil boring legend.
See Plate III-11 for detail shear test data.
See Plate IV-3 for soil boring location.

GENERAL NOTES

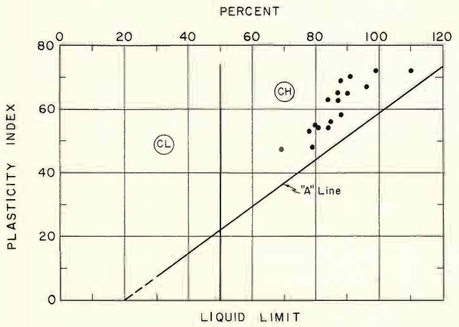
- ○ UC – Unconfined compression shear
- ● Ⓠ – Unconsolidated undrained triaxial shear
- ▲ Ⓡ – Consolidated undrained triaxial shear
- □ Ⓢ – Consolidated drained direct shear
- Ⓒ – Consolidation test
- w – Natural water content
- L.L. – Liquid limit
- P.L. – Plastic limit
- c – Unit cohesion
- φ – Angle of friction
- γ – Unit weight of soil - water system
- σ̄ – Normal stress
- O.B. – Overburden
- P̄c – Preconsolidation pressure
- e – Void ratio
- Cc – Compression index

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 – GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
UNDISTURBED BORING
39-WU DATA
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

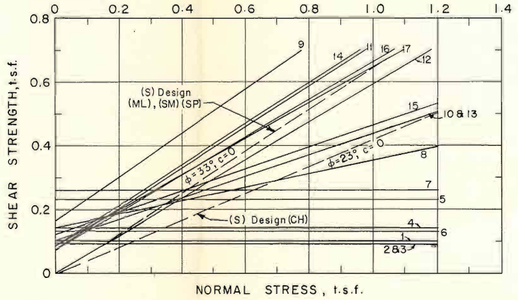




CONSOLIDATION DATA



PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE		TYPE	STRENGTH		CLASS
No.	El.		φ°	c (t.s.f.)	
1	-8.7	Q	0	0.10	CH
2	-15.9		0	0.09	CH
3	-19.2		0	0.09	CH
4	-32.9		0	0.14	CH
5	-39.3		0	0.23	CH
6	-41.3		0	0.13	CH
7	-49.5		0	0.26	CH
8	-13.9	R	12	0.14	CH
9	-23.3		35	0.16	SM
10	-16.2	S	19	0.10	CH
11	-21.7		33	0.07	ML
12	-27.3		31	0.00	ML
13	-39.3		19	0.10	CH
14	-45.5		33	0.08	SM
15	-47.8		19	0.14	CH
16	-52.9		31	0.07	SM
17	-58.2		30	0.08	SM

See Plate A for soil boring legend
See Plate III-6 for general notes
See Plate III-12 for detail shear test data
See Plate IV-3 for soil boring location

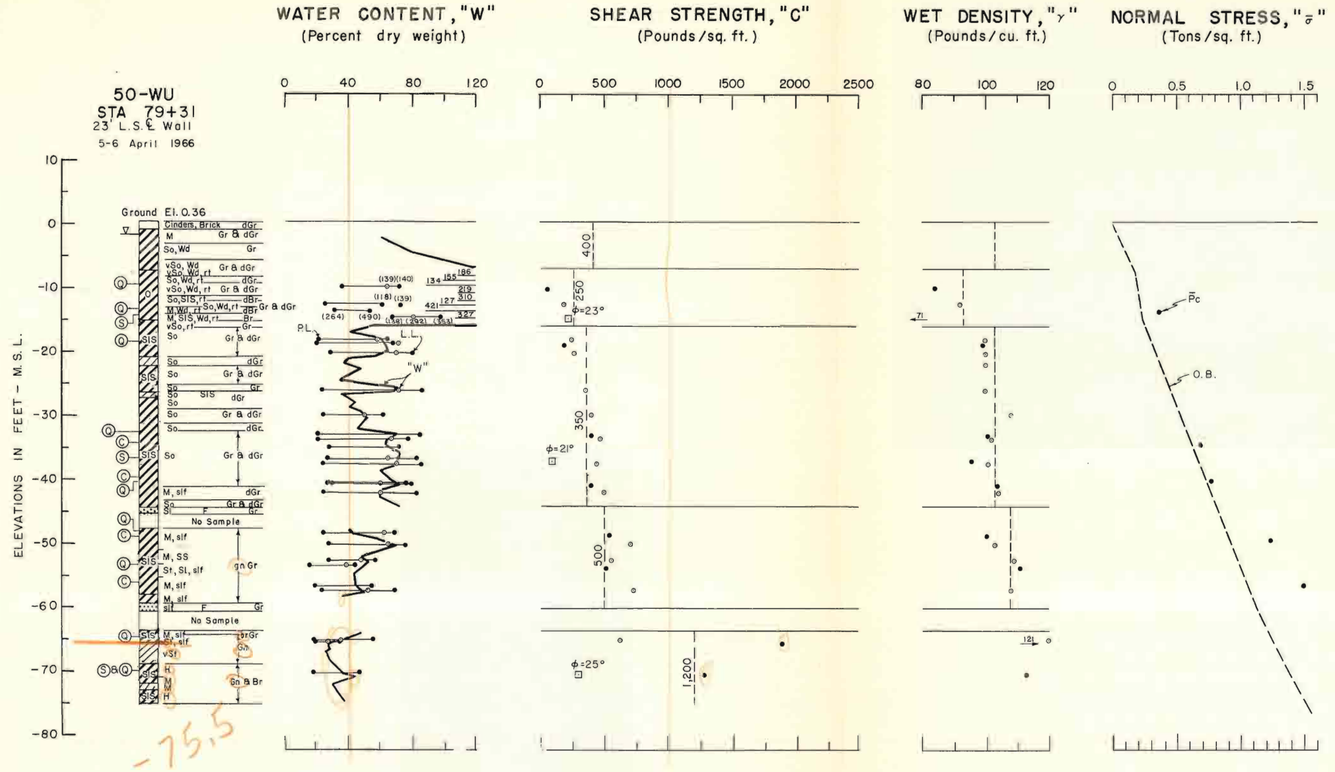
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

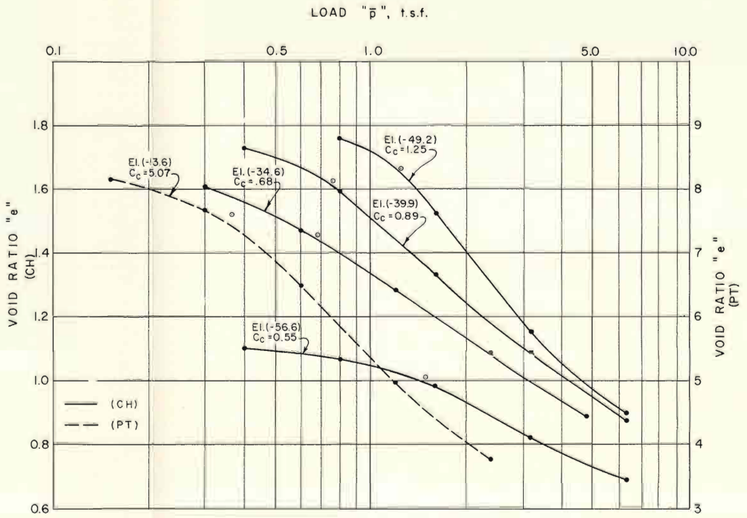
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

UNDISTURBED BORING 46-WU DATA

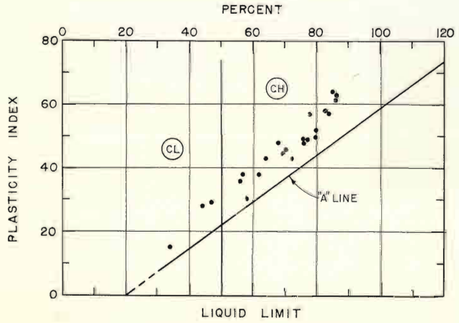
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-23909

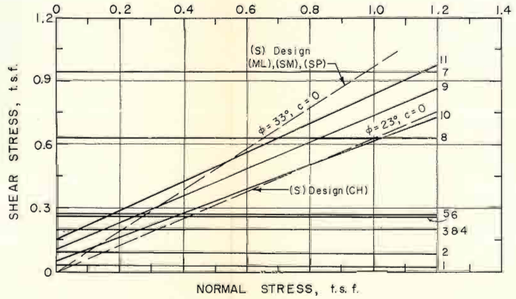




CONSOLIDATION DATA



PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE No.	ENVELOPE El.	TYPE	STRENGTH φ°	STRENGTH c (t.s.f.)	CLASS
1	- 9.6	Q	0	0.03	CH
2	-18.6			0.09	CH
3	-32.8			0.20	CH
4	-40.7			0.20	CH
5	-48.5			0.27	CH
6	- 53.6			0.26	CL
7	-65.3			0.95	CL
8	-70.1			0.64	CL
9	-14.4	S	23	0.11	PT
10	-36.7		21	0.05	CH
11	-70.1		25	0.15	CL

See Plate A for soil boring legend
See Plate III-6 for general notes
See Plate III-13 for detail shear test data
See Plate IV-4 for soil boring location

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

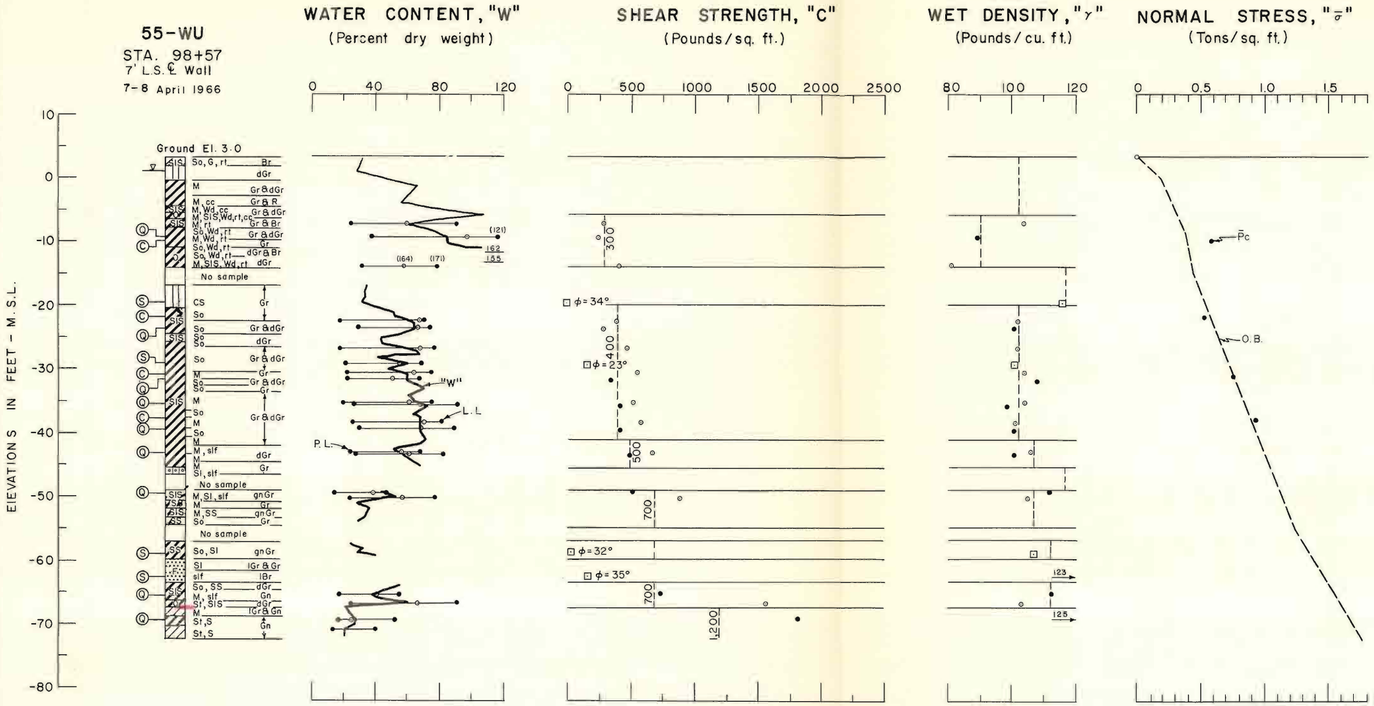
UNDISTURBED BORING
50-WU DATA

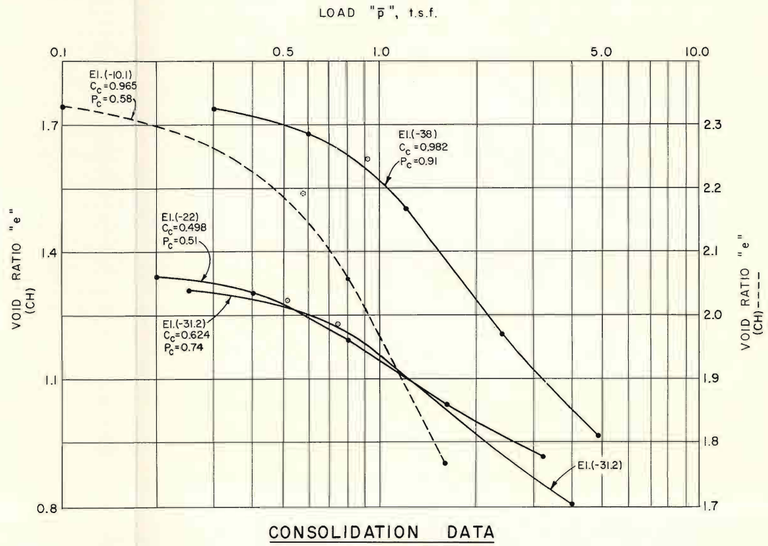
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-23909

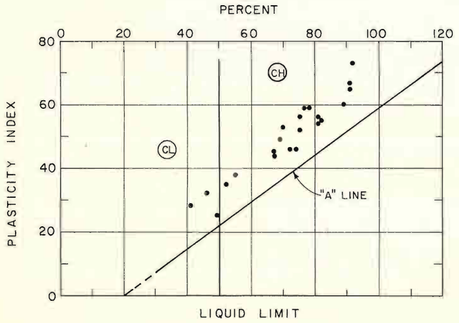
PLATE III - 8

83-50

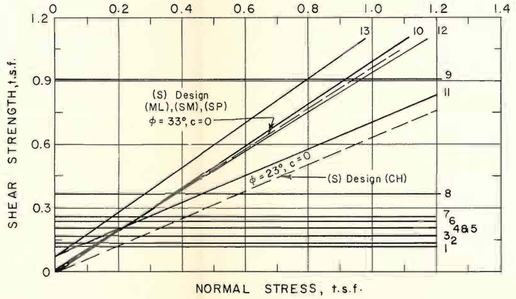




CONSOLIDATION DATA



PLASTICITY CHART



SHEAR STRENGTH DATA

ENVELOPE No.	ENVELOPE El.	TYPE	STRENGTH φ°	STRENGTH c (t.s.f.)	CLASS
1	-9.5	Q	0	0.12	CH
2	-23.8			0.14	CH
3	-31.9			0.17	CH
4	-35.9			0.21	CH
5	-39.7			0.21	CH
6	-43.6			0.24	CH
7	-49.4			0.26	CL
8	-65.4			0.37	CH
9	-69.4			0.91	CH
10	-19.8	S	34	0	ML
11	-29.4		23	0.08	CH
12	-59.0		32	0.02	ML
13	-62.8		35	0.08	SM

See Plate A for soil boring legend.
See Plate III-6 for general notes.
See Plate III-14 for detail shear test data.
See Plate IV-5 for soil boring location

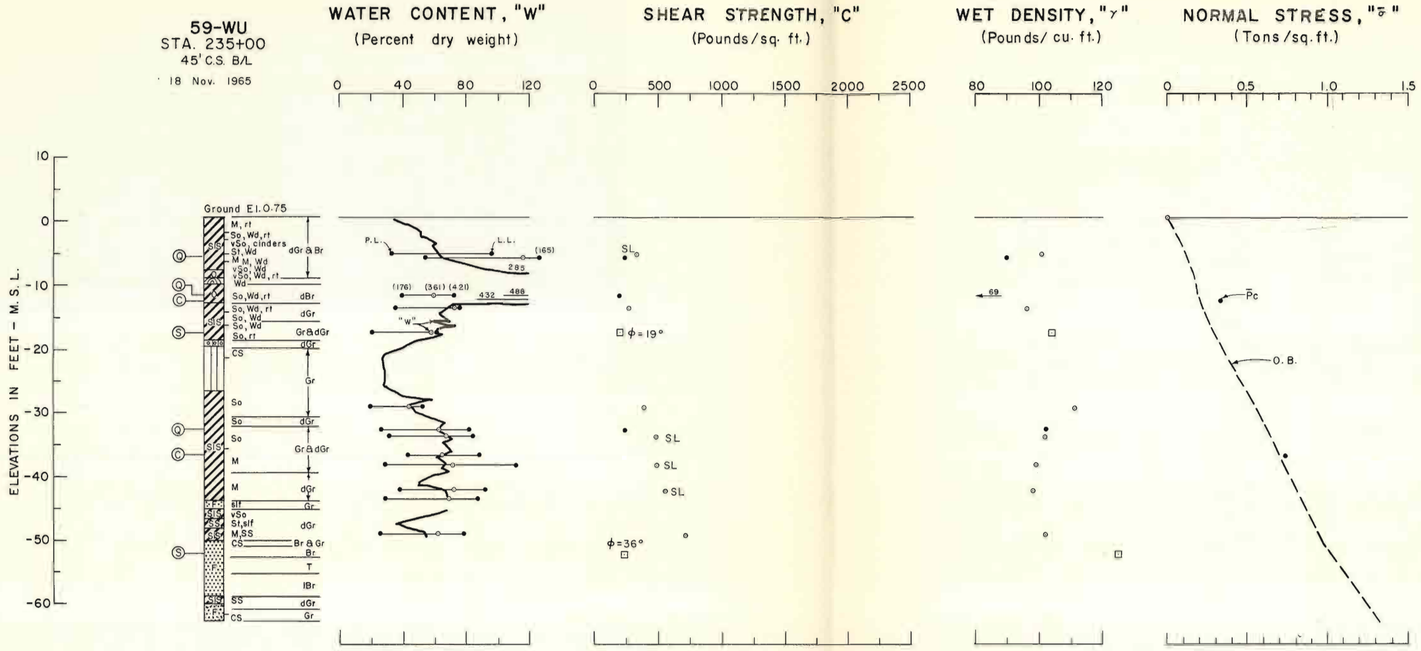
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

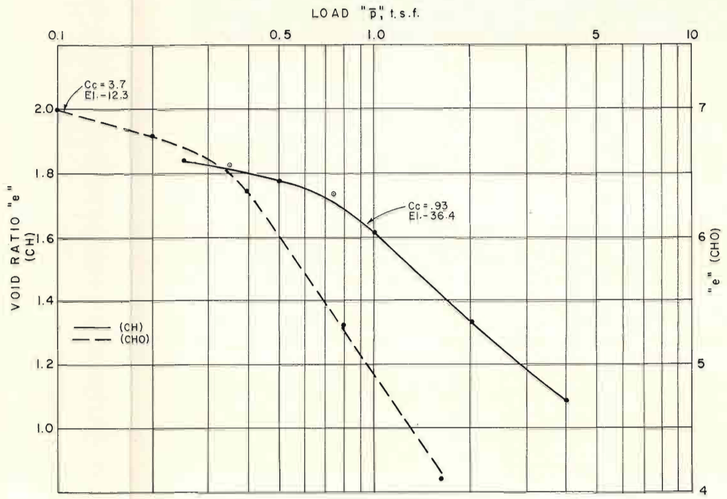
UNDISTURBED BORING 55-WU DATA

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

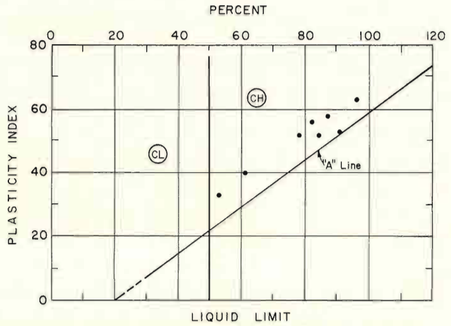
FEBRUARY 1967 FILE NO. H-2-23909

59-WU
STA. 235+00
45' C.S. B/L
18 Nov. 1965

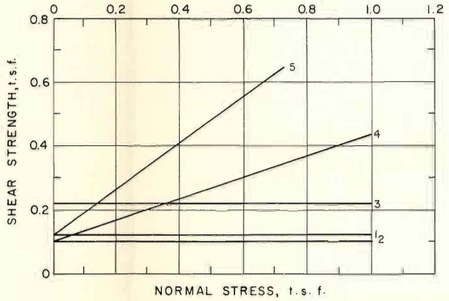




CONSOLIDATION DATA



PLASTICITY CHART



SHEAR STRENGTH DATA

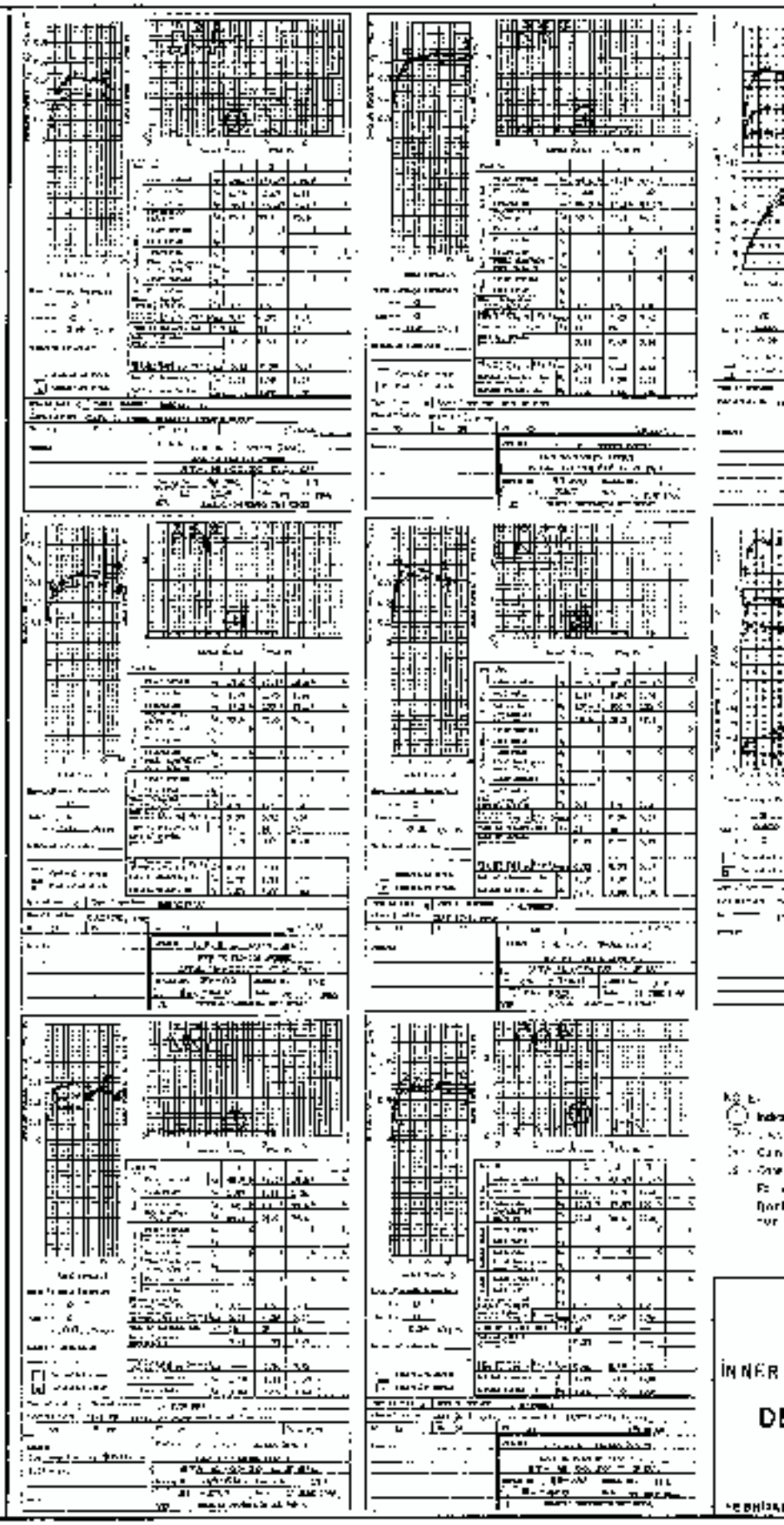
ENVELOPE No.	ENVELOPE El.	TYPE	STRENGTH φ°	STRENGTH c (t.s.f.)	CLASS
1	-5.4	Q	0	0.12	CH
2	-11.6			0.10	PT
3	-32.4			0.22	CH
4	-17.2	S	19	0.10	CH
5	-51.9		36	0.12	SP

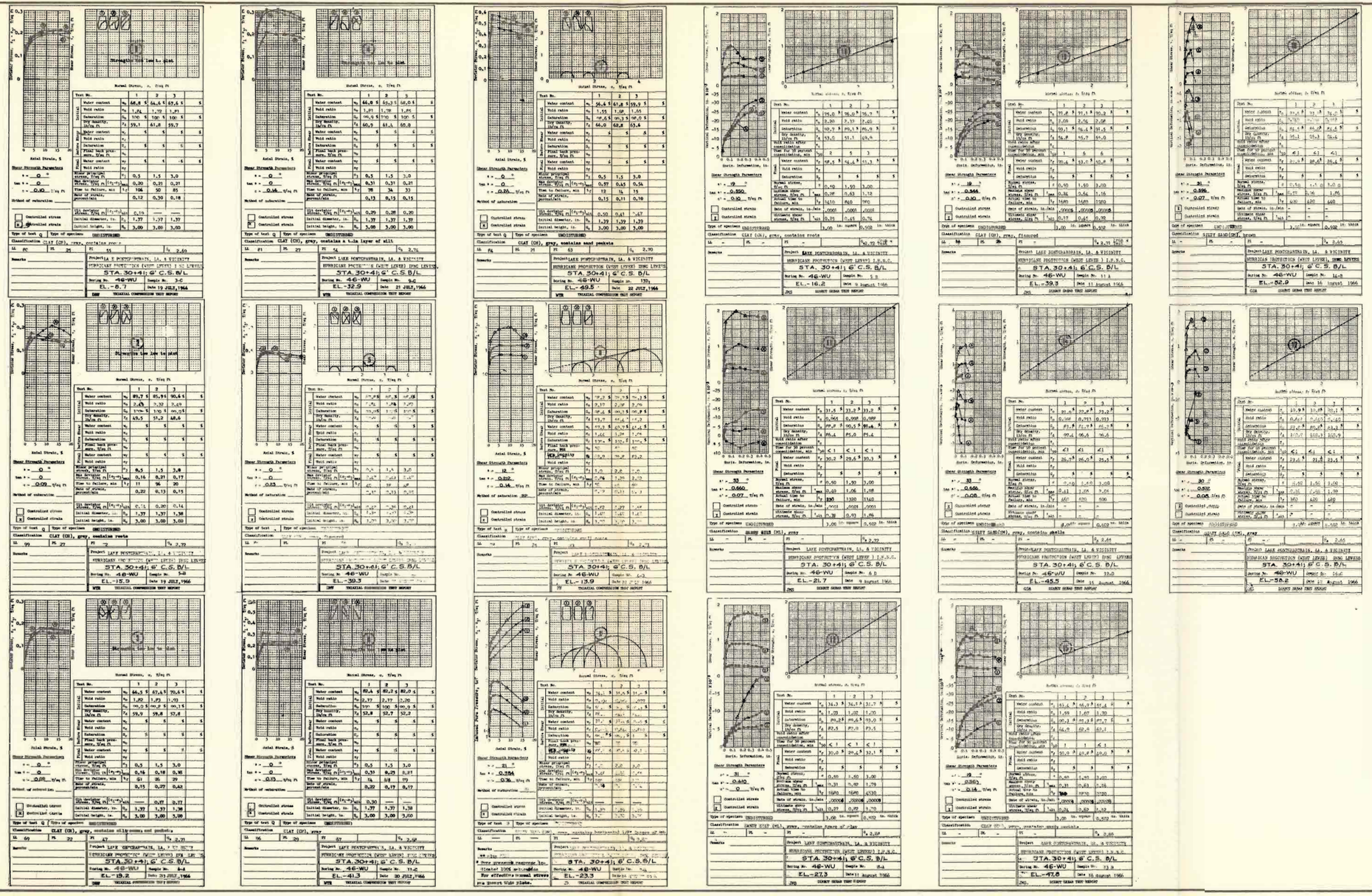
See Plate A for soil boring legend
See Plate III-6 for general notes
See Plate III-13 for detail shear test data
Boring station refers to abandoned alternate route, for location see plan map Plate IV-1

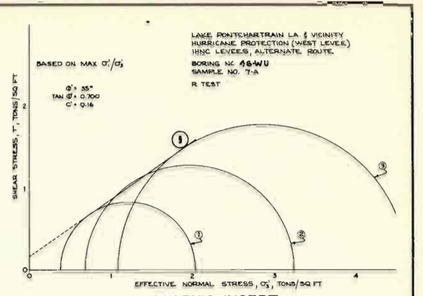
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

UNDISTURBED BORING
59-WU DATA

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909







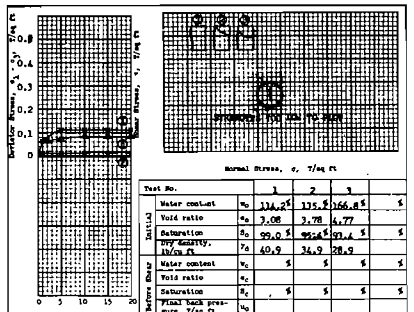
GRAPHIC INSERT

NOTE:
Refer to (1) for normal stress.

NOTE:
(1) Indicates reference number shown under shear data on Plate III-7
(Q) - Unconsolidated-undrained triaxial compression test
(R) - Consolidated-undrained triaxial compression test
(S) - Consolidated-drained direct shear test
For undisturbed soil boring data see Plate III-7
Boring station refers to field staked baseline, for location see Plate IV-3

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2-GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
DETAIL SHEAR STRENGTH DATA
BORING 46-WU
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

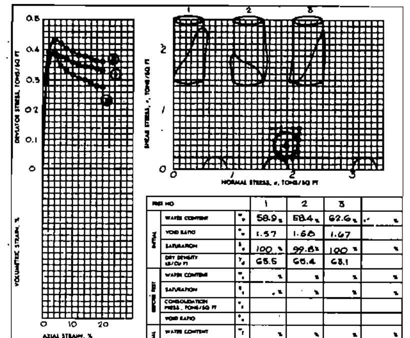
Triaxial Compression Test Report — Boring 50-WU, Sample 5-C



Test No.	1	2	3
Initial water content w_o, %	114.2	115.5	166.8
Void ratio e_o	3.08	3.78	4.77
Saturation S_o, %	99.0	95.4	93.4
Dry density, lb/cu ft	40.9	34.9	28.9
Minor principal stress, T/sq ft σ_3	0.5	1.5	3.0
Max deviator stress, T/sq ft $(\sigma_1-\sigma_3)_{max}$	0.11	0.01	0.08
Time to failure, min t_f	16	2	17
Rate of strain, percent/min	0.32	0.59	0.60
Initial diameter, in. D_o	1.39	1.40	1.39
Initial height, in. H_o	3.00	3.00	3.00

Shear Strength Parameters: φ = 0°; tan φ = 0; c = 0.03 T/sq ft. Controlled strain (x).
Type of test: Q. Type of specimen: UNDISTURBED. Classification: CLAY (CH), gray, contains organic matter. LL 140, PL 36, PI 104, G_s = 2.67.
Project: I. H. N. C. LEVEES (WEST); LOCK TO FLORIDA AVENUE – STA. 48 + 94; 63' – L.S. Boring No. 50-WU, Sample No. 5-C, EL. -9.6, Date 24 JUNE 1966. JL

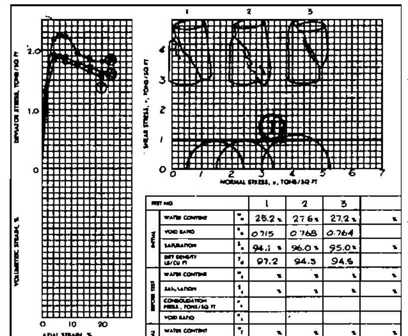
Triaxial Compression Test Report — Boring 50-WU, Sample 13-B



Test No.	1	2	3
Water content, %	58.9	68.4	62.6
Void ratio	1.57	1.68	1.67
Saturation, %	100	99.8	100
Dry density, lb/cu ft	65.5	66.4	63.1
Major principal stress, tons/sq ft	0.89	1.98	3.40
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	15	16	17
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.20 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.3%/MIN. Classification: GRAY CLAY, (CH), WITH 0.25-IN. SILT SEAM (CENTER). LL 76, PL 27, PI 49, G_s = 2.70.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 13-B, EL.-40.7, Date 7/6/66.

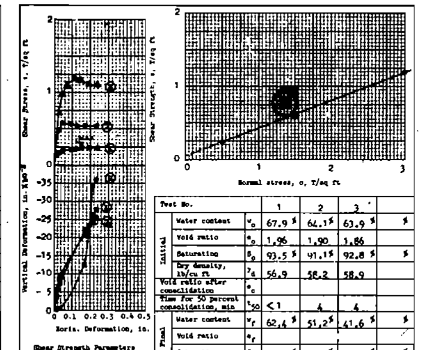
Triaxial Compression Test Report — Boring 50-WU, Sample 19-C



Test No.	1	2	3
Water content, %	26.2	27.6	27.2
Void ratio	0.715	0.768	0.764
Saturation, %	94.1	96.0	95.0
Dry density, lb/cu ft	97.2	94.3	94.6
Major principal stress, tons/sq ft	2.41	3.38	5.24
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	22	39	21
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.95 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain ★. Classification: GRAY SANDY CLAY, (CL), WITH SAND POCKETS. LL 34, PL 19, PI 15, G_s = 2.67.
Remarks: ★ 0.2%/MIN, TEST 1 & 2; 0.4 " " 3.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 19-C, EL.-65.3, Date 7/13/66.

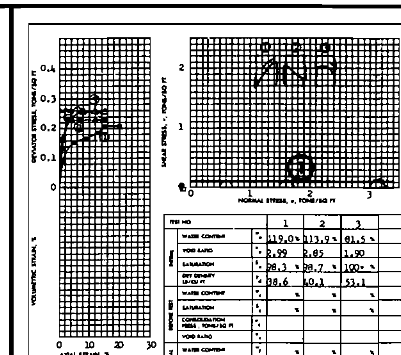
Direct Shear Test Report — Boring 50-WU, Sample 12-B



Test No.	1	2	3
Initial water content w_o, %	67.9	64.1	63.9
Void ratio e_o	1.96	1.90	1.86
Saturation S_o, %	93.5	91.1	92.8
Dry density, lb/cu ft	56.9	58.2	58.9
Time for 50 percent consolidation, min t_{50}	<1	4	4
Final water content w_f, %	62.4	51.2	41.6
Normal stress, T/sq ft σ	0.50	1.50	3.00
Maximum shear stress, T/sq ft τ_{max}	0.24	0.58	1.19
Actual time to failure, min t_f	1110	360	840
Rate of strain, in./min	.0001	.0001	.0001
Ultimate shear stress, T/sq ft τ_{ult}	0.24	0.56	1.10

Shear Strength Parameters: φ' = 21°; tan φ' = 0.380; c' = 0.05 T/sq ft. Controlled strain (x).
Type of specimen: UNDISTURBED, 3.00 in. square, 0.486 in. thick. Classification: CLAY(CH), gray. LL 84, PL 27, PI 57, G_s = 2.70.
Project: I.H.N.C. LEVEES (WEST); LOCK TO FLORIDA AVENUE – STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 12-B, EL.-38.7, Date 20 June 1966. JHS

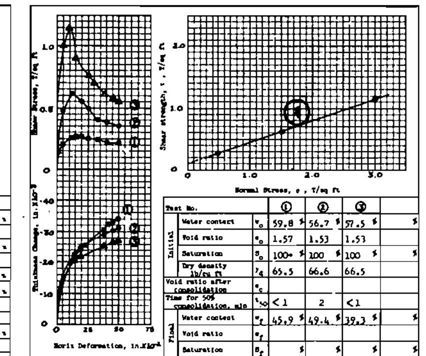
Triaxial Compression Test Report — Boring 59-WU, Sample 4-C



Test No.	1	2	3
Water content, %	119.0	113.9	81.5
Void ratio	2.99	2.85	1.90
Saturation, %	98.3	98.7	100+
Dry density, lb/cu ft	38.6	40.1	53.1
Major principal stress, tons/sq ft	0.71	1.73	3.26
Minor principal stress, tons/sq ft	0.5	1.5	3.0
Time to failure, min	60	7	33
Initial diameter, in.	1.38	1.38	1.38
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.12 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.25, 0.45, 0.24 %/min. Classification: CLAY (CH), gray, contains roots and other organic matter. LL 165, PL 54, PI 111, G_s = 2.47.
Project: LAKE PONTCHARTRAIN HURRICANE PROTECTION. STA.235+00;45' ℄ B/L ALT. RT. Boring No. 59-WU, Sample No. 4-C, El -5.4, Date 4 JAN 1966. WTR

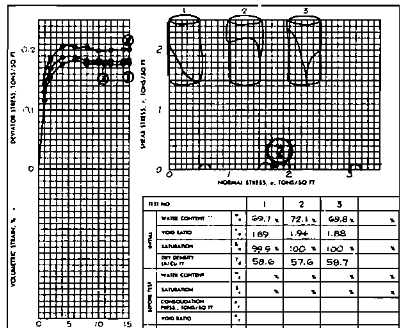
Direct Shear Test Report — Boring 59-WU, Sample 7-C



Test No.	①	②	③
Initial water content, %	59.8	56.7	57.5
Void ratio	1.57	1.53	1.53
Saturation, %	100+	100	100
Dry density, lb/cu ft	65.5	66.6	66.5
Time for 50% consolidation, min	<1	2	<1
Final water content, %	45.9	49.4	39.3
Actual time to failure, min	1110	870	630
Normal stress, T/sq ft	0.50	1.50	3.00
Maximum shear strength, T/sq ft	0.27	0.62	1.14

Shear Values: φ' = 19°; tan φ' = 0.344; c' = 0.10. Test Type: S. Controlled, strain.
Type of Specimen: UNDISTURBED, 3.00 in. Square, 0.486 in. Thickness. Classification: CLAY(CH), gray, contains thin horizontal silty seams. LL 61, PL 21, PI 40, G_s = 2.70.
Project: LAKE PONTCHARTRAIN HURRICANE PROTECTION. STA.235+00;45' ℄ B/L ALT. RT. Boring No. 59-WU, Sample No. 7-C, El -17.2, Date 17-Jan-66. JL

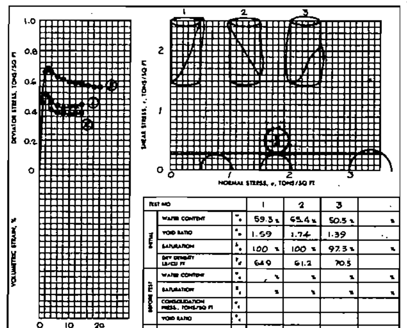
Triaxial Compression Test Report — Boring 50-WU, Sample 7-D



Test No.	1	2	3
Water content, %	69.7	72.1	69.8
Void ratio	1.89	1.94	1.88
Saturation, %	99.5	100	100
Dry density, lb/cu ft	58.6	57.6	58.7
Major principal stress, tons/sq ft	0.68	1.71	3.19
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	22	22	15
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.09 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.3%/MIN. Classification: GRAY CLAY, (CH), WITH SILT LAMINATIONS. LL 68, PL 20, PI 48, G_s = 2.71.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48+94; 63'-L.S. Boring No. 50-WU, Sample No. 7-D, EL-18.6, Date 7/6/66.

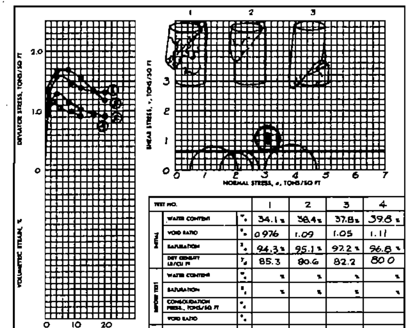
Triaxial Compression Test Report — Boring 50-WU, Sample 15-B



Test No.	1	2	3
Water content, %	59.3	65.4	50.5
Void ratio	1.59	1.74	1.39
Saturation, %	100	100	97.3
Dry density, lb/cu ft	64.9	61.2	70.5
Major principal stress, tons/sq ft	1.04	2.02	3.68
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	11	16	26
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.27 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain ★. Classification: GRAY CLAY, (CH), WITH SAND POCKETS. LL 69, PL 24, PI 45, G_s = 2.69.
Remarks: ★ 0.2%/MIN, TEST 1; 0.133 " " 2 & 3.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48+94; 63'-L.S. Boring No. 50-WU, Sample No. 15-B, EL.-48.5, Date 7/7/66.

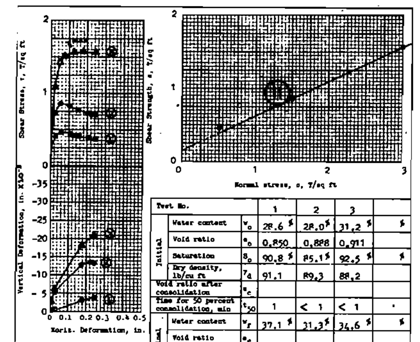
Triaxial Compression Test Report — Boring 50-WU, Sample 20-D



Test No.	1	2	3	4
Water content, %	34.1	38.4	37.8	39.8
Void ratio	0.976	1.09	1.05	1.11
Saturation, %	94.3	95.1	97.2	96.8
Dry density, lb/cu ft	85.3	80.6	82.2	80.0
Major principal stress, tons/sq ft	2.08	2.79	4.69	2.61
Minor principal stress, tons/sq ft	0.50	1.50	3.00	1.50
Time to failure, min	17	17	25	13
Initial diameter, in.	1.41	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.64 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.3%/MIN. Classification: GRAY & TAN CLAY, (CL), WITH SILTY CLAY POCKETS & ★. LL 47, PL 18, PI 29, G_s = 2.70.
Remarks: ★ BLOCKY, FLOCULATED STUCTURE.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 20-D, EL.-70.1, Date 7/14/66.

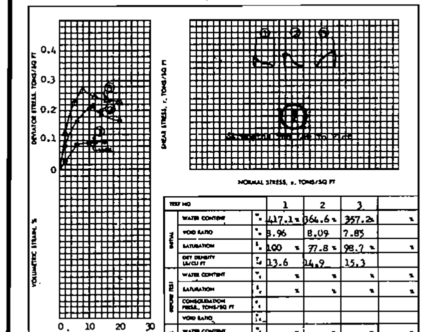
Direct Shear Test Report — Boring 50-WU, Sample 20-D



Test No.	1	2	3
Initial water content, %	28.6	28.0	31.2
Void ratio	0.850	0.888	0.911
Saturation, %	90.8	85.1	92.5
Dry density, lb/cu ft	91.1	89.3	88.2
Time for 50 percent consolidation, min	1	< 1	< 1
Final water content, %	37.1	31.3	34.6
Normal stress, T/sq ft	0.50	1.50	3.00
Maximum shear stress, T/sq ft	0.46	0.86	1.57
Actual time to failure, min	420	420	840
Rate of strain, in./min	.0001	.0001	.0001
Ultimate shear stress, T/sq ft	0.38	0.70	1.56

Shear Strength Parameters: φ' = 26°; tan φ' = 0.478; c' = 0.15 T/sq ft. Controlled strain (x).
Type of specimen: UNDISTURBED, 3.00 in. square, 0.486 in. thick. Classification: CLAY (CL), brown and gray, contains silty pockets. LL —, PL —, PI —, G_s 2.70 From Q Test.
Project: I.H.N.C. LEVEES (WEST); LOCK TO FLORIDA AVENUE – STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 20-D, EL.-70.1, Date 20 June 1966. JHS

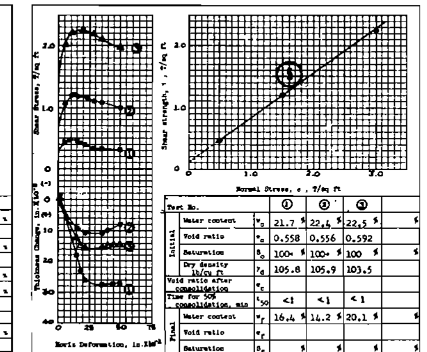
Triaxial Compression Test Report — Boring 59-WU, Sample 6-A



Test No.	1	2	3
Water content, %	417.1	364.6	357.2
Void ratio	8.96	8.09	7.85
Saturation, %	100	97.8	98.7
Dry density, lb/cu ft	13.6	14.9	15.3
Major principal stress, tons/sq ft	0.60	1.72	3.27
Minor principal stress, tons/sq ft	0.5	1.5	3.0
Time to failure, min	22	21	15
Initial diameter, in.	1.35	1.38	1.38
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.10 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.54, 0.51, 0.53 %/min. Classification: PEAT (PT), black, contains a few large roots. LL 421, PL 176, PI 245, G_s = 2.17.
Project: LAKE PONTCHARTRAIN HURRICANE PROTECTION. STA.235+00;45' ℄ B/L. ALT. RT. Boring No. 59-WU, Sample No. 6-A, El -11.6, Date 5 JAN 1966. WTR

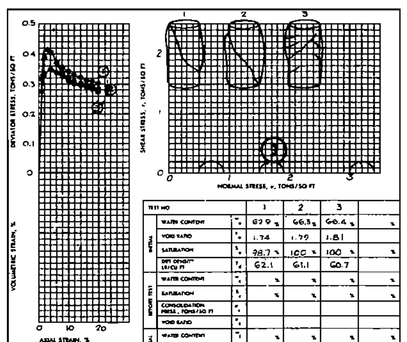
Direct Shear Test Report — Boring 59-WU, Sample 16-B



Test No.	①	②	③
Initial water content, %	21.7	22.4	22.5
Void ratio	0.558	0.556	0.592
Saturation, %	100+	100+	100
Dry density, lb/cu ft	105.8	105.9	103.5
Time for 50% consolidation, min	<1	<1	<1
Final water content, %	16.4	14.2	20.1
Actual time to failure, min	600	840	1320
Normal stress, T/sq ft	0.50	1.50	3.00
Maximum shear strength, T/sq ft	0.47	1.21	2.25

Shear Values: φ' = 36°; tan φ' = 0.726; c' = 0.12. Test Type: S. Controlled, strain.
Type of Specimen: UNDISTURBED, 3.00 in. Square, 0.486 in. Thickness. Classification: SAND (SP), gray, contains free water. LL —, PL —, PI —, G_s = 2.64.
Project: LAKE PONTCHARTRAIN HURRICANE PROTECTION. STA.235+00;45' ℄ B/L ALT. RT. Boring No. 59-WU, Sample No. 16-B, El -51.9, Date 24-Jan-66. JL

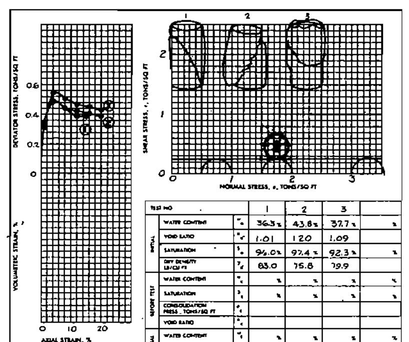
Triaxial Compression Test Report — Boring 50-WU, Sample 11-B



Test No.	1	2	3
Water content, %	62.9	66.3	66.4
Void ratio	1.74	1.79	1.81
Saturation, %	98.7	100	100
Dry density, lb/cu ft	62.1	61.1	60.7
Major principal stress, tons/sq ft	0.94	1.90	3.35
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	17	17	22
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.20 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain ★. Classification: GRAY CLAY, (CH), WITH TAN OXIDIZED PLANES. LL 85, PL 21, PI 64, G_s = 2.73.
Remarks: ★ 0.2%/MIN., TEST 1 & 2; 0.3 " " 3.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 11-B, EL.-32.8, Date 7/5 & 7/6/66.

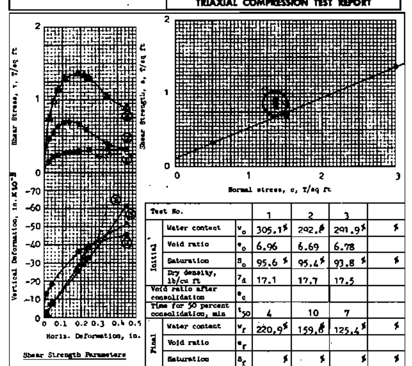
Triaxial Compression Test Report — Boring 50-WU, Sample 16-C



Test No.	1	2	3
Water content, %	36.3	43.8	37.7
Void ratio	1.01	1.20	1.09
Saturation, %	96.0	97.4	92.3
Dry density, lb/cu ft	83.0	75.8	79.9
Major principal stress, tons/sq ft	1.00	2.01	3.56
Minor principal stress, tons/sq ft	0.50	1.50	3.00
Time to failure, min	18	16	18
Initial diameter, in.	1.41	1.41	1.41
Initial height, in.	3.00	3.00	3.00

Shear values: φ = 0; tan φ = 0; c = 0.26 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.2%/MIN. Classification: GRAY CLAY, (CL), WITH SAND SEAMS & POCKETS, AND ★. LL 44, PL 16, PI 28, G_s = 2.67.
Remarks: ★-SMALL SHELL FRAGMENTS.
Project: IHNC LEVEES (WEST), LOCK TO FLORIDA AVENUE. STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 16-C, EL.-53.6, Date 7/7 & 7/12/66.

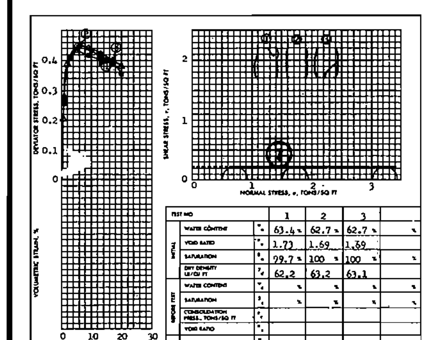
Direct Shear Test Report — Boring 50-WU, Sample 6-D



Test No.	1	2	3
Initial water content, %	305.1	292.8	291.9
Void ratio	6.96	6.69	6.78
Saturation, %	95.6	95.4	93.8
Dry density, lb/cu ft	17.1	17.7	17.5
Time for 50 percent consolidation, min	4	10	7
Final water content, %	220.9	159.8	125.4
Normal stress, T/sq ft	0.50	1.50	3.00
Maximum shear stress, T/sq ft	0.32	0.69	1.36
Actual time to failure, min	2490	990	1320
Rate of strain, in./min	.0001	.0001	.0001
Ultimate shear stress, T/sq ft	0.32	0.32	0.89

Shear Strength Parameters: φ' = 23°; tan φ' = 0.416; c' = 0.11 T/sq ft. Controlled strain (x).
Type of specimen: UNDISTURBED, 3.00 in. square, 1&2-0.486, 3-0.748 thick. Classification: PEAT (PT), brown, contains roots. LL 353, PL 138, PI 215, G_s 2.18.
Project: I.H.N.C., LEVEES (WEST); LOCK TO FLORIDA AVENUE – STA. 48 + 94; 63'-L.S. Boring No. 50-WU, Sample No. 6-D, EL.-14.4, Date 17 June 1966. JHS

Triaxial Compression Test Report — Boring 59-WU, Sample 11-B



Test No.	1	2	3
Water content, %	63.4	62.7	62.7
Void ratio	1.73	1.69	1.69
Saturation, %	99.7	100	100
Dry density, lb/cu ft	62.2	63.2	63.1
Major principal stress, tons/sq ft	0.93	1.95	3.45
Minor principal stress, tons/sq ft	0.5	1.5	3.0
Time to failure, min	25	27	20
Initial diameter, in.	1.39	1.38	1.38
Initial height, in.	3.00	3.00	3.00

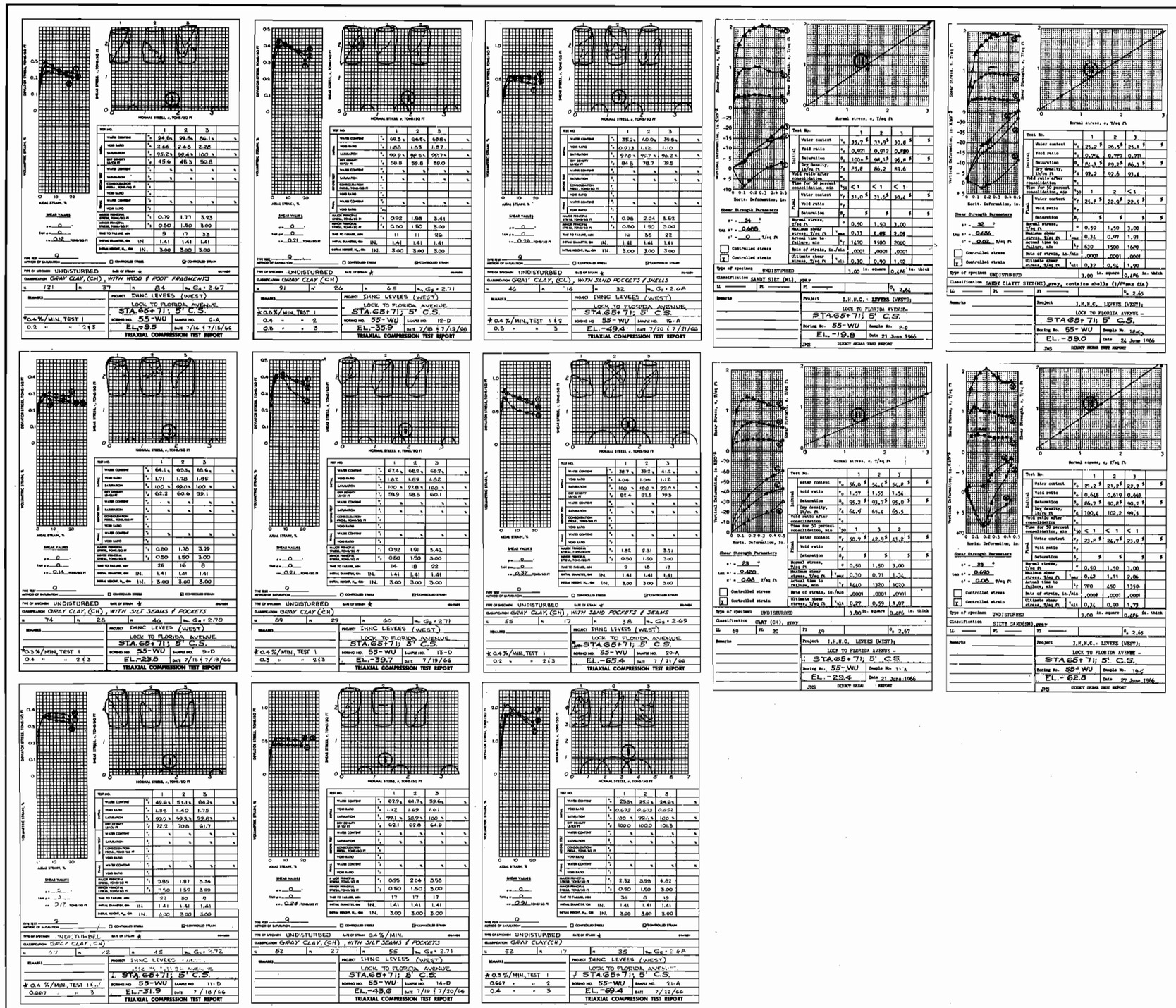
Shear values: φ = 0; tan φ = 0; c = 0.22 tons/sq ft. Type test: Q. Controlled strain.
Type of specimen: UNDISTURBED. Rate of strain 0.22, 0.23, 0.25 %/min. Classification: CLAY (CH), gray, fissured. LL 82, PL 26, PI 56, G_s = 2.72.
Project: LAKE PONTCHARTRAIN HURRICANE PROTECTION. STA.235+00;45' ℄ B/L. ALT. RT. Boring No. 59-WU, Sample No. 11-B, El -32.4, Date 5 JAN 1966. WTR

NOTE:
① Indicates reference number shown under shear data on Plates III-8 III-10
(Q) - Unconsolidated-undrained triaxial compression test.
(S) - Consolidated-drained direct shear test.
For undisturbed soil boring data see Plates III-8 III-10
Boring 50-WU station refers to field staked baseline, for location see Plate IV. 4
Boring 59-WU refers to abandoned alternate route, for location see plan map Plate IV. 1

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE FLORIDA AVE. TO IHNC LOCK
DETAIL SHEAR STRENGTH DATA BORINGS 50-WU AND 59-WU

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967 FILE NO. H-2-23909



NOTE:

(1) Indicates reference number shown under shear data on Plate III-9

(Q) - Unconsolidated-undrained triaxial compression test.

(S) - Consolidated-drained direct shear test

For undisturbed soil boring data see Plate III-9

Boring station refers to field staked baseline, for location see Plate IV-5

LAKE PONTCHARTRAIN, LA. AND VICINITY

LAKE PONTCHARTRAIN BARRIER PLAN

DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN

ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE

FLORIDA AVE. TO IHNC LOCK

DETAIL SHEAR STRENGTH DATA

BORING 55-WU

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS

CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-23909

SECTION IV - STRUCTURAL DESIGN

CRITERIA FOR STRUCTURAL DESIGN

1. General. Structural design has been accomplished in accordance with standard engineering practice and criteria set forth in Engineering Manuals for Civil Works Construction published by the Office, Chief of Engineers. Agreements reached in a conference on design problems held in the New Orleans District on 13-14 December 1966 provided the bases for certain design procedures and decisions. Procedures for the preparation of this design memorandum were reviewed at this conference, which was attended by OCE, LMVD, and NOD representatives. Minutes of the conference were forwarded to LMVD by LMNED-DD letter dated 30 December 1966, "Lake Pontchartrain, La. and Vicinity - Inner Harbor Navigation Canal (IHNC) - Floodwall Conference," copy of which is included in appendix I.

2. Basic data. Basic data relevant to the design of the protective works are shown in the following table:

		Elevations
a.	Water elevations.	
	Project flow line (surge elevation from design hurricane)	13.0
	Landside of floodwall	0.0
b.	Floodwall grades.	
	Net grade (one foot freeboard over project flow line)	14.0
	Top of wall, I-type wall in levee (as constructed)	15.0
	Top of wall, I-type wall in natural ground (as constructed)	14.5
	Top of wall, T-type wall (as constructed)	14.0
	Top of access gates (as constructed)	14.0

c. Unit weights.

Item	Lb.per cu.ft.
Water	62.5
Concrete	150
Steel	490
Earth	See plates III-2 through III-10

d. Design loads.

(1) Earth pressures (lateral). See figures 4-1 through 4-7.

(2) Water loads.

(a) No wave forces will occur.

(b) One (1) foot freeboard. Water at elevation 14.0.

(c) Design water elevations as follows:

	Flood side	Protected side
1. I-wall in levee	14.5	0
2. I-wall in nat. ground	14.0	0
3. T-wall	14.0	0

(3) Wind loads.

(a) On walls	30 p.s.f.
(b) On overhead beams	50 p.s.f.

3. Allowable working stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM 1110-1-2101, of 6 January 1958, revised August 1963. Concrete will be designated by basic minimum strength 3,000 p.s.i. concrete, except for prestressed concrete piling which shall be designated 5,000 p.s.i. concrete. Steel sheet piling meeting the requirements of ASTM A328-54, "Standard Specifications for Steel Sheet Piling," will be used. For convenient reference, pertinent allowable stresses are tabulated below:

Reinforced concrete	Stress p.s.i.
f'_c	3,000
f_c	1,050
v (without web reinforcement)	60
v (with web reinforcement)	274
f_s	20,000
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel	0.0020 bt
Structural steel (ASTM A-36)	
Basic stress	18,000

DESIGN OF STRUCTURES

4. Location and alignment. The floodwall will be constructed along the west bank of the IHNC from Florida Avenue to the IHNC lock. The area includes industrial plants, wharves, railroad tracks, barge and truck loading facilities, machine shops, warehouses, an overhead bridge, and a U. S. Coast Guard Station. Beginning at Florida Avenue, the floodwall will extend along the existing levee for approximately 900 feet, then pass between the Florida Avenue Wharf and the Chase Bag Company site, thence across the Lone Star Cement Company site, thence east through the north side of the Jones and Laughlin Steel Company warehouse, thence south to the south end of the Galvez Street Wharf, thence east along the south end of the wharf, thence south passing under the Claiborne Avenue Bridge, thence along the west side of the U. S. Coast Guard Station, thence east through the Coast Guard Station to the IHNC lock. The alignment is shown on plates IV-1 through IV-6 and, in greater detail, on plates IV-9 through IV-18. Stationing in this section refers to floodwall centerline unless otherwise stated.

5. A large rock storage bin belonging to the Lone Star Cement Company is located between stations 56+54 to 57+70. The design of protective works in this reach will be contained in the supplement covering the remainder of the protective works on the IHNC.

6. Between the Galvez Street Wharf and the IHNC lock, various floodwall alignments were investigated. These investigations established that economies could be effected by adopting the dogleg east in the alignment at the south end of the Galvez Street Wharf, rather than continuing south generally parallel to the canal from this point, and locating the alignment south from the dogleg so as to skirt the U. S. Coast Guard Station. This would reduce

the required railroad gap closures, provide adequate horizontal clearance of the North Claiborne Avenue bridge piers, eliminate a very large gate near the IHNC lock (or track relocation in lieu thereof), and reduce real estate requirements.

7. Between Florida Avenue and North Roman Street, an alternate alignment 1,200 to 1,400 feet landward of the IHNC also was investigated. This alignment was not adopted for a number of reasons, including: greater length of wall; lower average ground surface resulting in a higher wall; and failure to provide protection to industries located along the IHNC.

8. Foundation. The results of subsurface explorations, soils tests, and foundation studies are presented in Section III. Logs for the general-type borings are plotted on plates IV-7 and IV-8. The undisturbed boring data are shown on plates III-6 through III-10.

9. Selection of wall type. The type of wall to be constructed will depend on the height of wall required which, in turn, will depend on the elevation of the ground surface. In the area covered by this supplement, the ground surface varies from elevation 1.5 to 5.0, except that the crown of an emergency levee south of the Galvez Street Wharf is at elevation 12.5. A cantilever I-type wall will be constructed where floodwall heights above ground surface are not in excess of 10 feet and deflection of the wall will not be a problem. Two types of I-walls will be used--one will be a conventional I-type wall and the other located in the crown of a levee and designated as an "I-type wall in levee."

10. Investigations were made of a number of types of floodwalls to be used where the height of the wall will exceed 10 feet. Included in these investigations were bearing-pile supported T-type and L-type walls, strutted I-type walls, prestressed concrete cylinder pile walls, and sand-filled steel sheet pile cofferdams. These investigations indicated that the bearing-pile supported T-type wall would be the most suitable and economical.

11. I-type wall in levee. From stations 34+95 to 38+65 and stations 39+75 to 44+12, the floodwall will consist of I-type wall constructed in an existing levee which will be reshaped. Design analysis of the floodwall is shown on figure 4-1. Bending moments and deflections for structural design are based on a factor of safety of 1.0 applied to the soils since the structural steel has an inherent safety factor of about 2.0. Plan and profile of the wall are shown on plates IV-2 and IV-3, a design section on plate IV-19, and details on plate IV-21. The strength of the wall was checked for the case with water at the top of the wall, as initially constructed, and found to be adequate as shown on figure 4-1. Where possible, expansion joints in I-type walls will be spaced 30 feet apart.

12. I-type wall. Except for the gate monoliths (gates 4 through 8), an I-type wall will be constructed between stations 49+68 and 65+39.75 and stations 106+77 and 108+01. The plan and profile of the wall are shown on plates IV-3, IV-4, and IV-6, sections on plates IV-19 and IV-20, and details on plate IV-21.

13. The design analysis for the section of the wall between the Chase Bag Company and the Florida Avenue Wharf (stations 49+68 to 51+07) is shown on figure 4-2. A reverse earth and surcharge loading condition is shown on figure 4-3.

14. Figures 4-2, 4-4, and 4-5 show the design analysis for the I-wall in the Lone Star Cement Plant (stations 51+67.75 to 52+94.75, 53+41 to 55+74, 56+19 to 56+54, 57+70 to 58+25.75, and 58+69.75 to 59+30). A tied-back concrete sheet pile bulkhead is presently located along the eastern side of the plant at the edge of a turning basin, from which barges are loaded. In certain locations it will be necessary to cut the tie rods before the sheet piling is driven, then splice them back together. The floodwall will pass through a small concrete block building, a portion of which will have to be rebuilt. It will also be necessary to dismantle and reassemble a conveyor line and small cover shed to drive the sheet piling. Between stations 58+86 to 59+20, the sheet piling will be MA-22 instead of Z-27 because of the small clearance between a chimney foundation and the tie rod anchor blocks as shown on the section on plate IV-19.

15. From station 59+30 to 65+39.75, the floodwall is located on property leased to the Jones and Laughlin Steel Company. At station 59+64 the floodwall will enter the warehouse, cross a railroad track (gate 8), then turn east and run parallel to the north side of the warehouse. Figures 4-6 and 4-7 show the design analysis of the I-type wall in this reach. The height of the wall is approximately 10 feet above ground surface. The sheet piling ranges in length from 34 feet to 41 feet. The overhead clearance inside the warehouse is 19.8 feet at crane girders (spaced 80 feet on centers) and 26.6 feet to the skylight between these girders. To avoid splicing each pile, a 20-foot section of the roof will be removed and replaced with corrugated metal sheeting for the entire length of the building. Theoretical analyses indicate that the wall would deflect 4 to 4-1/2 inches. To reduce this deflection, creosoted timber kicker piles will be located 6 feet on centers at the bottom of the I-type wall as shown in plan on plate IV-13. Design sections are shown on plates IV-19 and IV-20 and details on plate IV-21. The deflection of the I-type wall will produce a lateral displacement at the junction of gate posts and I-type walls. A special seal located in a notch in the I-type wall will prevent water from passing through the expansion joints when the wall is deflected. The seal detail is shown on plate IV-21.

16. Five 80-foot bays are located in the Jones and Laughlin Steel warehouse between stations 60+06 and 64+06. Steel plates and rods will be transported over the wall using existing overhead cranes to load and unload railroad cars. The track on which the cars travel is located 16 feet north of the wall. In the first four bays, the top 3 feet of the wall will consist of a hinged metal flap which can be lowered to provide the necessary clearance. The last bay is located adjacent to a barge loading platform and will require a 5'-8" hinged metal flap (gate 9). All of the flaps will be raised with "jib" cranes fastened to the building columns. Plans and details of the hinged flaps are shown on plate IV-29.

17. In the vicinity of the IHNC lock, a conventional I-type wall will be constructed between stations 106+77 and 107+87. Figure 4-7 shows the analysis of this wall. Between station 107+87 and station 108+01 tie-in between the I-wall and the existing lock levee will be effected by continuation of the steel sheet piling of the standard I-type wall. The top of the sheet piling will, however, be below the top of the levee and the concrete portion will, accordingly, be omitted. The plan and profile are shown on plate IV-6, section and elevations on plate IV-20, and details on plate IV-21.

18. <u>T-type wall</u>.

a. General. Between stations 44+12 to 45+74, 46+62.25 to 49+68, and 65+39.25 to 106+77, a bearing pile supported T-type floodwall will be constructed. Gates 10 through 15, located between these stations, are discussed in paragraph 25. Plan and profile for these reaches are shown on plates IV-2 through IV-6, sections on plates IV-19 and IV-20, and details on plate IV-21. An emergency levee was constructed south of the Galvez Street Wharf during the past hurricane season by the Orleans Levee District. This levee, located between stations 90+42 and 95+30.25, will be degraded to elevation 5.0 and a T-type floodwall built in its place. Even though portions of the floodwall between stations 89+06 to 95+30.5 are less than 10 feet above ground, excessive deflections prevent the use of an I-type wall.

b. Type of piling. Factors considered in the selection of the type of piling include availability, economy, resistance to decay, resistance to corrosive soil and water conditions, and fitness for driving. In general, the above considerations indicate 12-inch by 12-inch square precast prestressed concrete piles to be the most suitable. The prestressed concrete piles will meet the requirements of the Joint AASHO and PCI Committee Standard Specifications for "Square Concrete Prestressed Piles." Overhead clearances limit the lengths of pile that can be driven in one

piece under the Claiborne Avenue bridge. Standard steel pipe pile, 12 inches in diameter and filled with concrete, will be used in this location. Such piling can be driven in short lengths and readily spliced. The steel piles will be protected against corrosion by coal tar epoxy coating and by cathodic protection.

c. Allowable pile loads. Allowable pile loads for various reaches are shown on plate III-5. The required lengths to develop these loads will be determined from test piles.

d. Test piles. Data pertaining to the test piles are presented in Section III, paragraph 17.

e. Design of T-type wall. The T-type floodwalls were designed for the following conditions:

Case I - Water at elevation 14.0 on flood side and water at elevation 0.0 on protected side. Sheet pile cutoff pervious. Uplift varies uniformly from full head on flood side to tailwater on the protected side.

Case II - Same as Case I except sheet pile cutoff impervious. Uplift full head on flood side of cutoff and tailwater on protected side.

Case III - Water at elevation 10.0 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.

Case IV - Same as Case III except cutoff impervious and uplift as in Case II.

Case V - Water at elevation 7.5 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.

Case VI - Same as Case V except cutoff impervious and uplift as in Case II.

In all cases, the earth pressure was assumed to be balanced.

19. The top of the T-type walls is at elevation 14.0. The top of the base is at elevation 2.75 from station 44+12 to station 45+74 and station 46+62.25 to station 49+68; elevation 1.0 from station 65+39.25 to station 106+10; and elevation 2.0 from station 106+10 to station 106+77. In some areas between station 65+39.25 and station 106+55, the ground surface is at elevation 1.5 (6 inches above the base), but since this section of wall is in a highly

industrialized area and because the mean water level in the canal is at elevation 0.5, it is considered unlikely that the ground surface will ever be lower than elevation 1.0. A maximum ground surface elevation of 5.0 will exist from stations 89+06 to 95+30.25.

20. The base width will be 8 feet throughout. After analyzing the wall with the stem at various locations on the base, it was determined that the centerline of the stem should be 5'-9" from the flood side edge of the base to minimize the number of piles required to support the floodwalls.

21. Various pile foundations were investigated including a layout consisting of two piles battered in opposite directions and two layouts with one vertical and two batter piles. Three methods of analysis were used to check the pile foundations as follows:

a. "Analysis of Pile Foundations with Batter Piles," by A. Hrennikoff, Transactions, ASCE Vol. 115 (1950). (Used for checking all layouts.)

b. "Design of Pile Foundations," by G. Vetter, Transactions, ASCE Vol. 104 (1939). (Used for checking the layout with two batter piles.)

c. "Culmann's method for the Design of Pile Foundations" from "Theoretical Soil Mechanics" by K. Terzaghi. (Used for checking the two layouts with one vertical and two batter piles.)

22. These studies indicate that a foundation consisting of two piles battered in opposite directions is the most suitable and economical for the T-type walls. Where the floodwall changes direction, certain piles will be driven either vertically or at angles to the wall as required for driving clearances and to resist horizontal forces acting in more than one direction.

23. Figures 4-8 through 4-19 show computations for the design of the T-type wall between stations 99+84 and 106+10. These computations are typical also for the other reaches of T-type wall where different conditions obtain; i.e., when the allowable pile loads are different; when the top of the base is not at elevation 1.0; or when the modulus of soil reaction (K) is not 135 p.s.i. as shown on figure 4-11. Figures 4-8 and 4-9 show the loading conditions, the horizontal and vertical loads, and the moments obtained for the 6 cases described in paragraph 18 of this section. Figure 4-10 shows the pile loads obtained using Hrennikoff's method. Case II produced maximum axial pile loads and Case III maximum transverse pile loads. Determination of the

allowable transverse loads is shown on figure 4-11. In this determination, the soil was considered to have a constant modulus of subgrade reaction (K) with depth. Curves of actual and allowable transverse loads and deflections for various values of K are shown on figure 4-12. A comparison of these curves indicates that, for soils having a K value greater than 50 p.s.i., the actual axial and transverse loads and transverse deflections are considerably less than the allowable values and do not govern the design of the pile foundation. An approximate graphical solution using Vetter's method is shown on figure 4-13 for Cases I and II. The small eccentricity obtained indicates that the two pile layout is satisfactory. Figure 4-14 shows two arrangements, "A" and "B," of a three pile foundation analyzed in accordance with Hrennikoff's method. In both arrangements, a number of different locations for the interior pile were evaluated. The graphical analysis of the three pile foundation using the Culmann's method is shown on figures 4-15 and 4-16. Arrangement "B" shows that the interior pile will carry a very small load, thereby indicating that a two pile layout is sufficient. Concrete and reinforcement requirements are determined on figures 4-17 through 4-19. Transverse and longitudinal reinforcement will be provided to distribute the concentrated forces and moments induced by gate posts, irregular pile spacing, and changes in direction of the wall. Where possible, expansion joints in the T-type wall will be spaced 60 feet apart.

24. Figures 4-20 and 4-21 show computations for the allowable pile loads used in the design of the T-type walls not covered in the above paragraph. Figure 4-21 lists a tabulation of the spacing, transverse and axial loads, and transverse deflection of the piles supporting all of the T-type floodwall.

25. Gates.

a. General. Fifteen gaps will be left in the wall at street, loading platform, and railroad crossings. Gates stored at the site will be used to close the gaps. Two major requirements governed the design of the gates: (1) the need for rapid closure of the gates; and (2) the need for closure of the gates without special equipment; i.e., cranes or lift trucks. To meet these requirements, it was determined that either overhead roller or swing-type gates should be employed. The bases of railroad gaps are designed to distribute the surcharge load resulting from a Cooper E-60 loading at railroad crossings and H-20 loading for street and platform crossings. To resist water forces when the gates are closed, the bases will be supported, in general, by piles having a maximum spacing equal to that shown on figure 4-21 plus an additional row of piles at each gate post. The gates are shown in plan and profile on plates IV-2 through IV-6 and in plan only on plates IV-9 through IV-18.

b. Overhead roller gates. Eleven overhead roller gates will be constructed where storage areas adjacent to the gaps are available. Nine gates will be used to close street openings and two to close gaps at railroad crossings. The overhead beam and rail of the street gates will be removable to provide unlimited vertical clearance with the gates open. The clear opening of the overhead gates will vary from 15 feet to 30 feet horizontally. Design of the 30-foot gate (gate 10) located at Galvez Street is shown on figures 4-22 through 4-31. A plan, elevation, and section of this gate are shown on plate IV-23. A typical plan, elevation, section, and gate schedule for all other overhead gates are shown on plate IV-24. Details of the overhead roller gate, including the type of trolley used and the bottom and side seals, are shown on plate IV-25.

c. Swing gates. Three swing gates will be constructed at railroad crossings located where the floodwall makes a right angle turn and where gate storage adjacent to the wall is not practicable. The gates, in the closed position, will be slightly tilted towards the protected side to provide additional clearance between the railroad tracks and the bottom of the gates when opening and closing the gates. Adjustment of the gates in a vertical direction will be made by raising or lowering the hinges and adjusting the tie rods. Provisions will be made for adjusting the vertical seal and block in a horizontal direction and the bottom seal in a vertical direction. The design of a typical swing gate is shown on figures 4-33 through 4-38. Typical plan, elevation, section, and gate schedule are shown on plate IV-26. Swing gate and hinge details are shown on plates IV-27 and IV-28.

d. Flap gate. A 5'-8" hinged flap gate (gate 9) will be constructed at the loading platform in the Jones and Laughlin Steel Company warehouse. Design of gate 9 is shown on figures 4-39 through 4-41. Plan and details are shown on plate IV-29.

26. Utility crossings. Details of water, sewer, gas pipelines, and cable crossings are shown on plate IV-22.

27. Protective measures against corrosion.

a. General. Based on instructions contained in 2d Indorsement to LMNED-PP letter dated 1 November 1966 subject "Lake Pontchartrain, La. & Vicinity, General Design Memorandum No. 3, Chalmette Area Plan," all steel piling in contact with backfill or new levee fill will be coated with 20 mils of coal tar epoxy. This includes the steel sheet piling of the cantilever I-type floodwall, and the steel sheet pile cutoff of the T-type floodwall. Inasmuch as investigations required to establish the need for

cathodic protection would, if undertaken now, delay construction in the critical area covered in this supplement, provision has been made in the design of the protective works covered herein for adding such cathodic protection as may be found necessary at a later date. These provisions are described in subsequent paragraphs. The studies required to establish the nature and extent of cathodic protection required will be initiated at an early date, and a detail design memorandum covering corrosion protection in general will be prepared. Inasmuch as subterranean electrical measurements subsequent to the construction of the walls are considered to be an essential element of the above studies, the detail design memorandum cannot be completed until the protective works have been constructed.

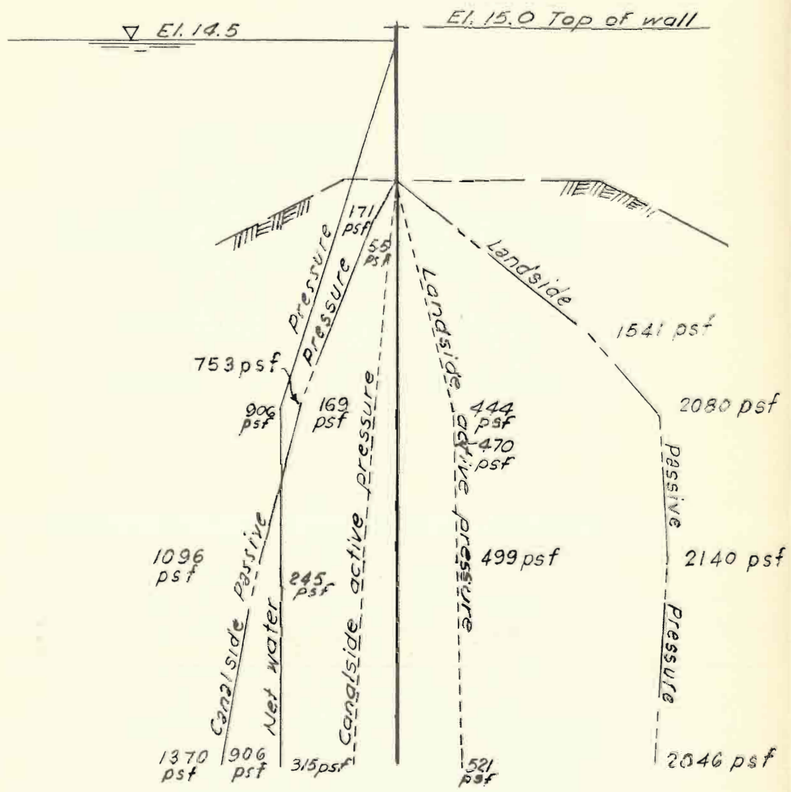
b. Interim provisions to cathodic protection.

(1) I-type floodwall. The sheet piles will be electrically bonded together with a No. 6 reinforcing bar welded to the piles near the top. Flexible jumpers will be provided at intervals of 120 feet.

(2) T-type floodwall. In addition to the electrical bonding described above, No. 6 reinforcing bars welded to the sheet piles at 100-foot intervals will be brought up through the concrete to elevation 6.0. The bars will terminate in 6-inch diameter ferrules, set in the concrete wall, and equipped with screw-type brass closure plugs. The ferrules will be set at intervals of 100 feet, and adjacent ferrules will be located on opposite sides of the wall, providing connection points at intervals of 200 feet on either side of the wall.

(3) Steel pipe piles. The steel pipe piles supporting the T-type floodwall under the North Claiborne Avenue bridge will be coated with 20 mils of coal tar epoxy and bonded to the sheet pile cutoff with No. 6 reinforcing bars.

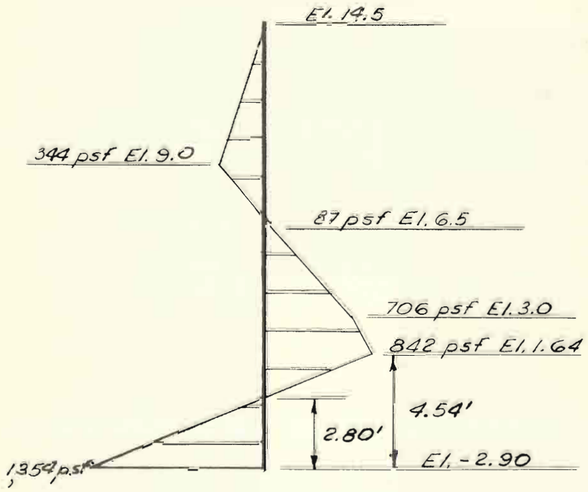
(4) The above provisions will permit the installation of sacrificial anode cathodic protection as found necessary at a later date. They will also permit future installation of low resistance metallic paths and drainage anodes for return, to its source, of stray direct current resulting from operation of direct current equipment at existing or future installations in the industrial complex.



PRESSURE DIAGRAM (F.S.=1.0)

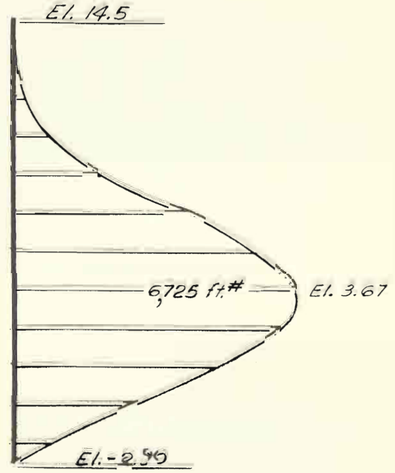
SCALES: 1" = 5'
1" = 1,000 psf

NOTE: With water at El. 15.0, max. moment = 7,203'# at El. -3.72



NET PRESSURE DIAGRAM (F.S.=1.0)

SCALES: 1" = 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 3,000 ft.#

STA. 34+95 TO STA. 38+65
STA. 39+75 TO STA. 44+12

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
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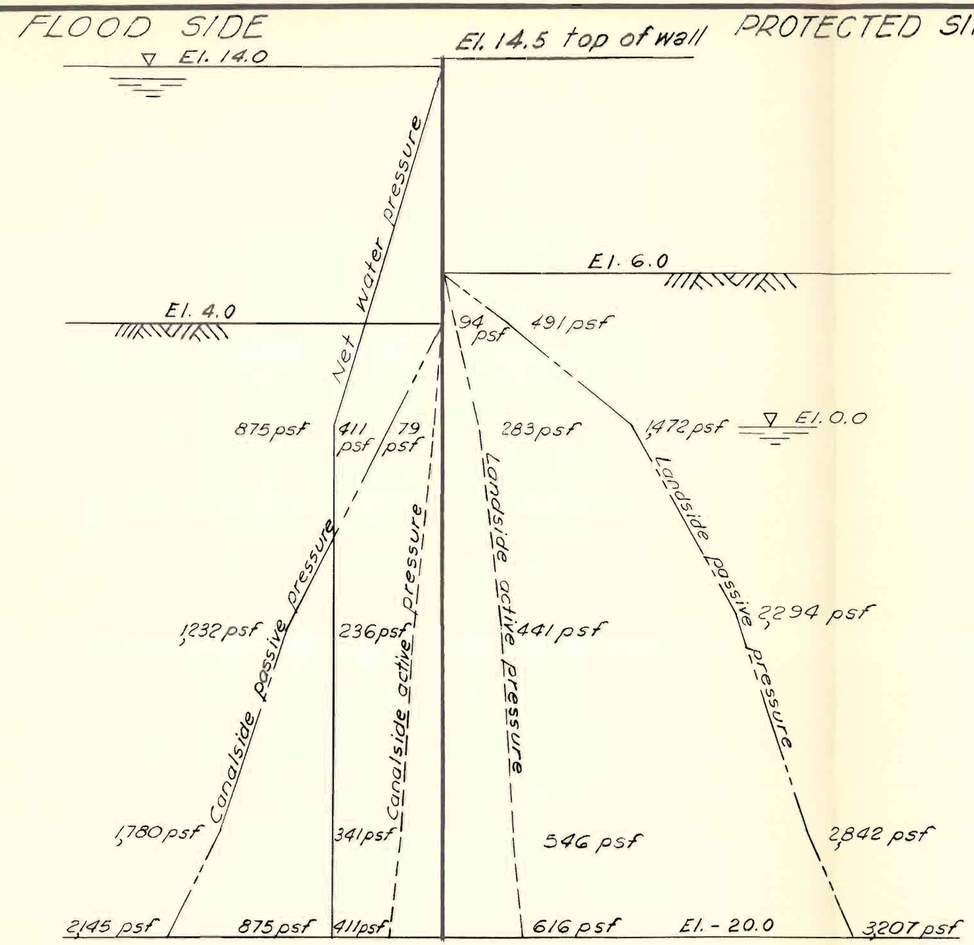
FLORIDA AVE. TO I.H.N.C. LOCK
I - WALL DESIGN ANALYSIS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1957
FILE NO. H-2-23909

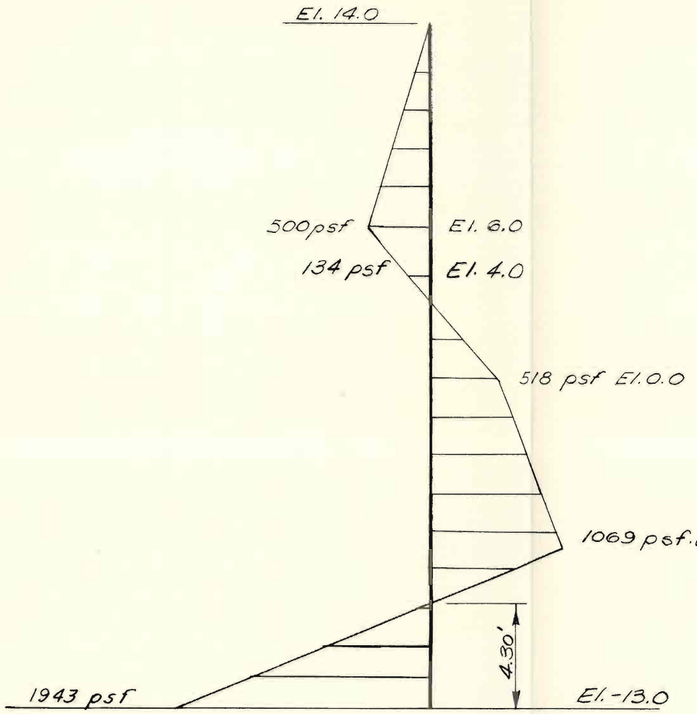
FIGURE 4-1



PRESSURE DIAGRAM (F.S. = 1.0)

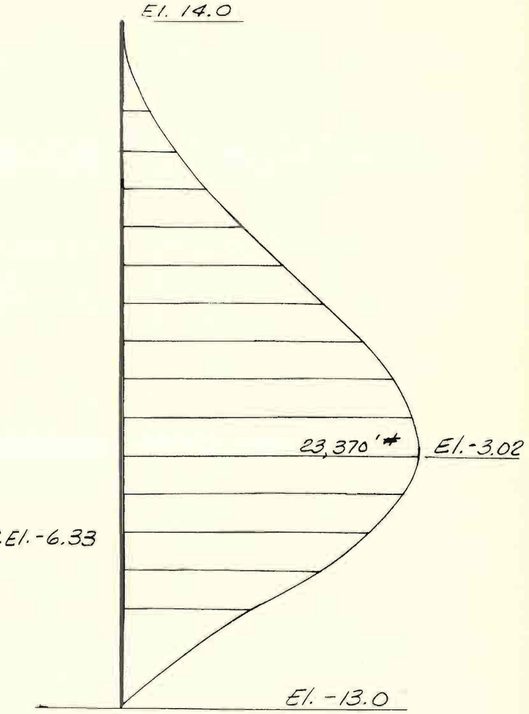
SCALES: 1" = 5'
1" = 1,000 psf

NOTE: With water at El. 14.5, max. moment = 27,260'# at El. -3.59



NET PRESSURE DIAGRAM (F.S. = 1.0)

SCALES: 1" 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 10,000 ft #

STA. 49+68 TO STA. 52+94.75

LAKE PONTCHARTRAIN, LA. AND VICINITY
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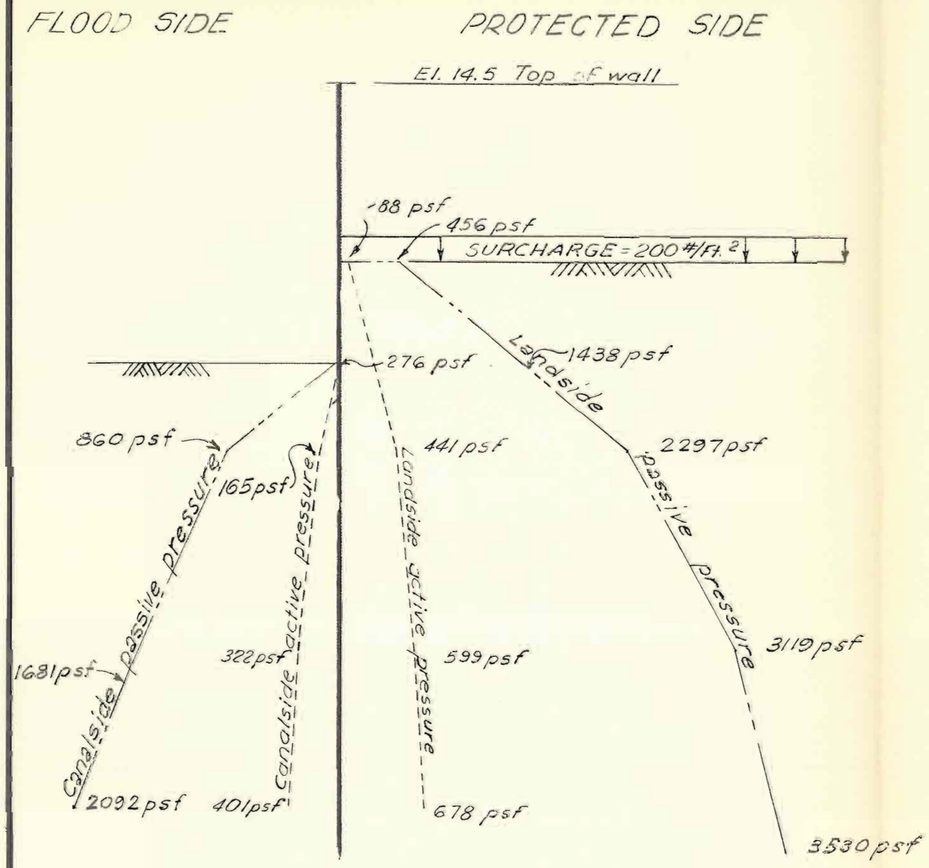
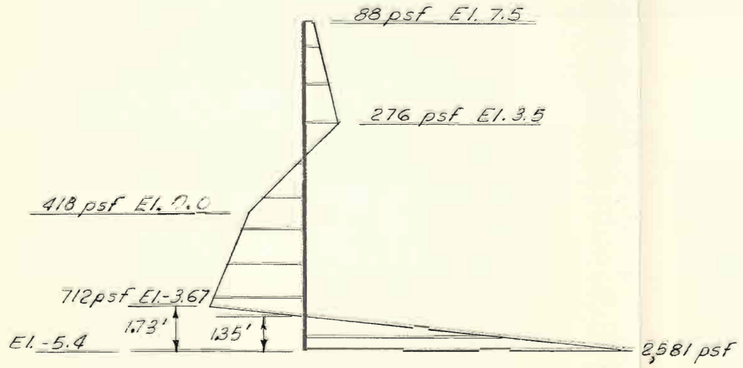
FIGURE 4-2



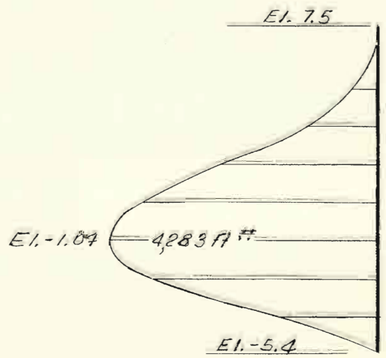
PRESSURE DIAGRAM (F.S. = 1.0)

SCALES: 1" = 5'
1" = 1,000 psf



NET PRESSURE DIAGRAM (F.S. = 1.0)

SCALES: 1" = 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 2,000 ft.#

STA. 49+68 TO STA. 51+07

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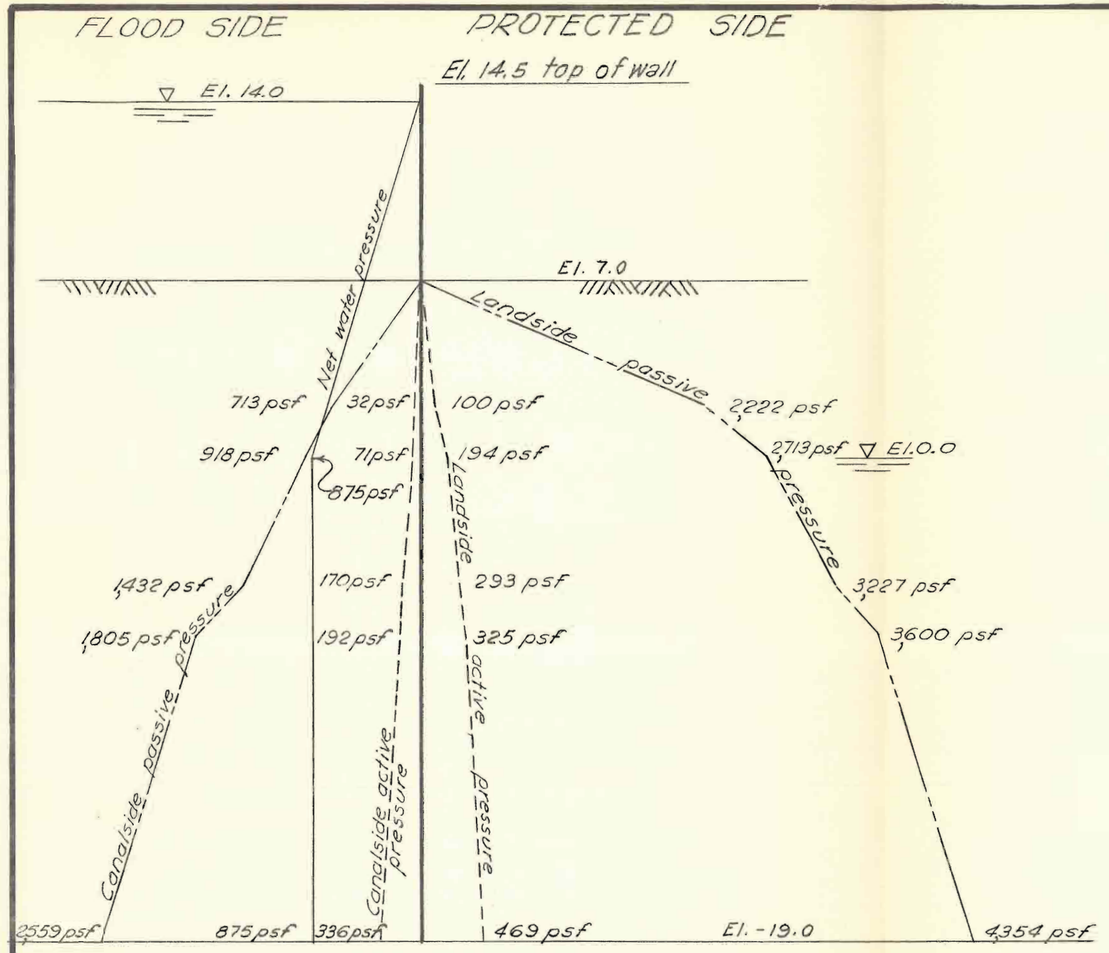
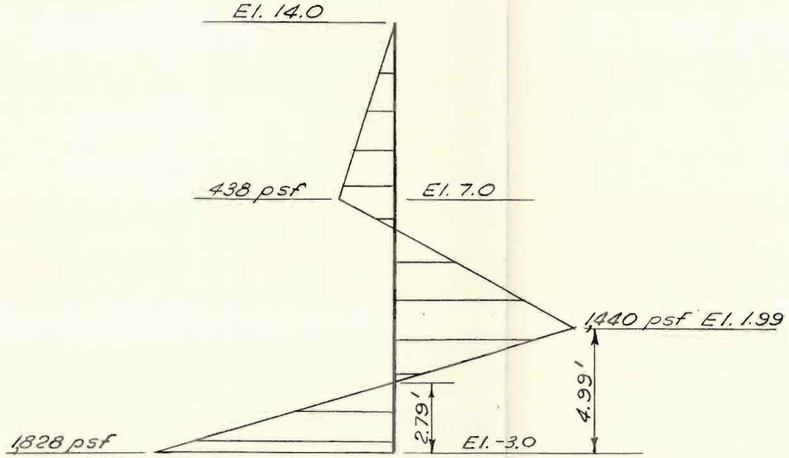
FIGURE 4-3



PRESSURE DIAGRAM (F.S. = 1.0)

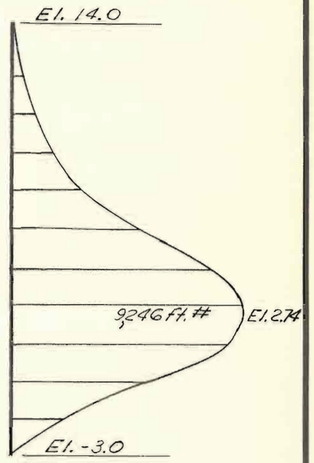
SCALES: 1" = 5'
1" = 1,000 psf

NOTE: With water at El. 14.5, max. moment = 11,362' # at El. +2.44



NET PRESSURE DIAGRAM (F.S. = 1.0)

SCALES: 1" = 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 5,000 ft. #

STA. 53+41 TO STA. 55+74

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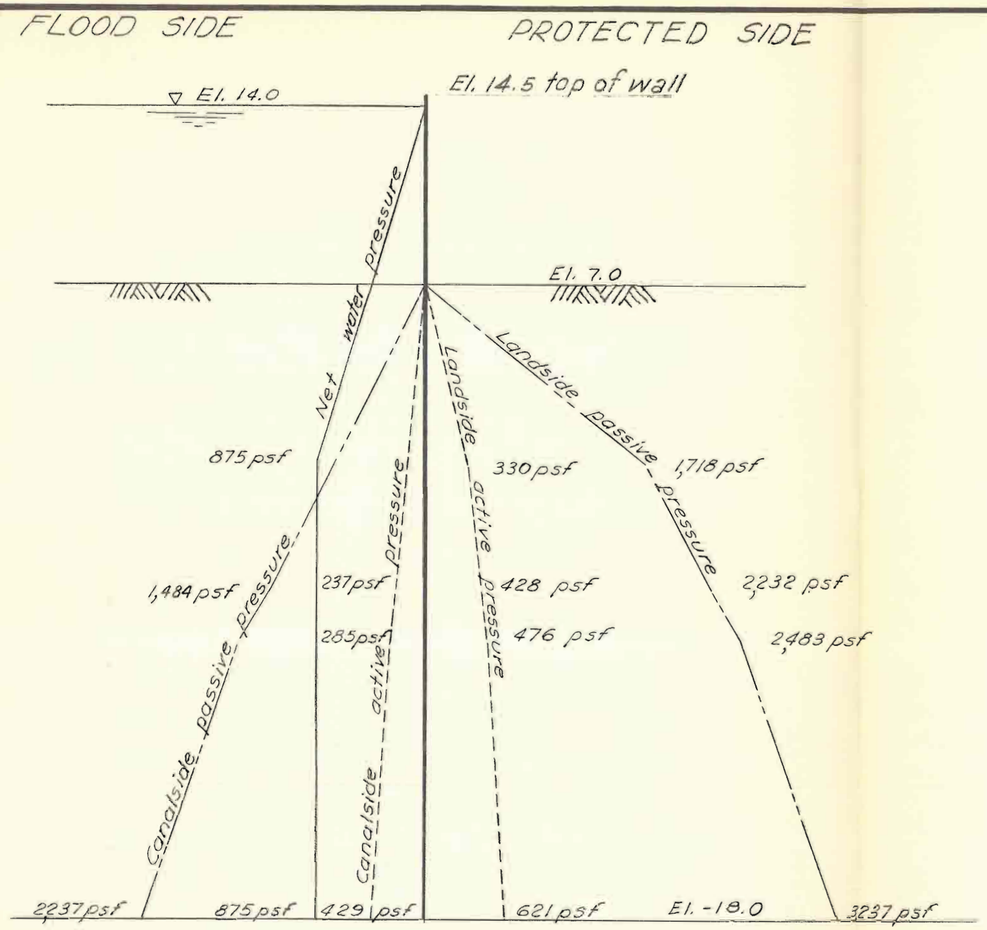
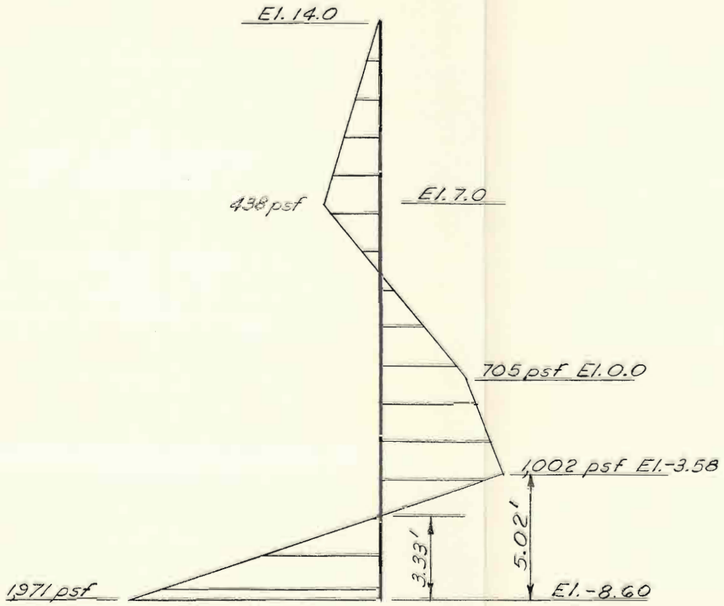
FIGURE 4-4



PRESSURE DIAGRAM (F.S.=1.0)

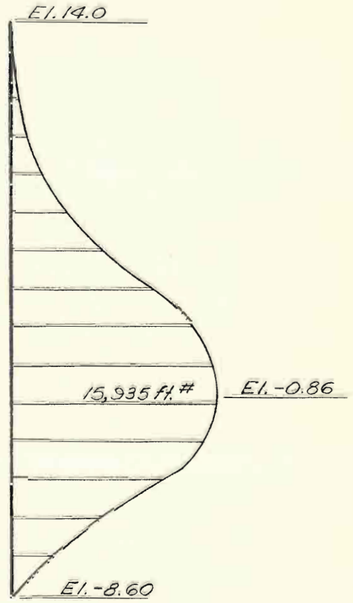
SCALES: 1" = 5'
1" = 1,000 psf

NOTE: With water at El. 14.5, max. moment = 19,631 at El. - 1.42



NET PRESSURE DIAGRAM (F.S.=1.0)

SCALES: 1" = 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 10,000 ft.#

STA. 56+19 TO STA. 56+54
STA. 57+70 TO STA. 58+25.75
STA. 58+69.75 TO STA. 59+30

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FIGURE 4-5

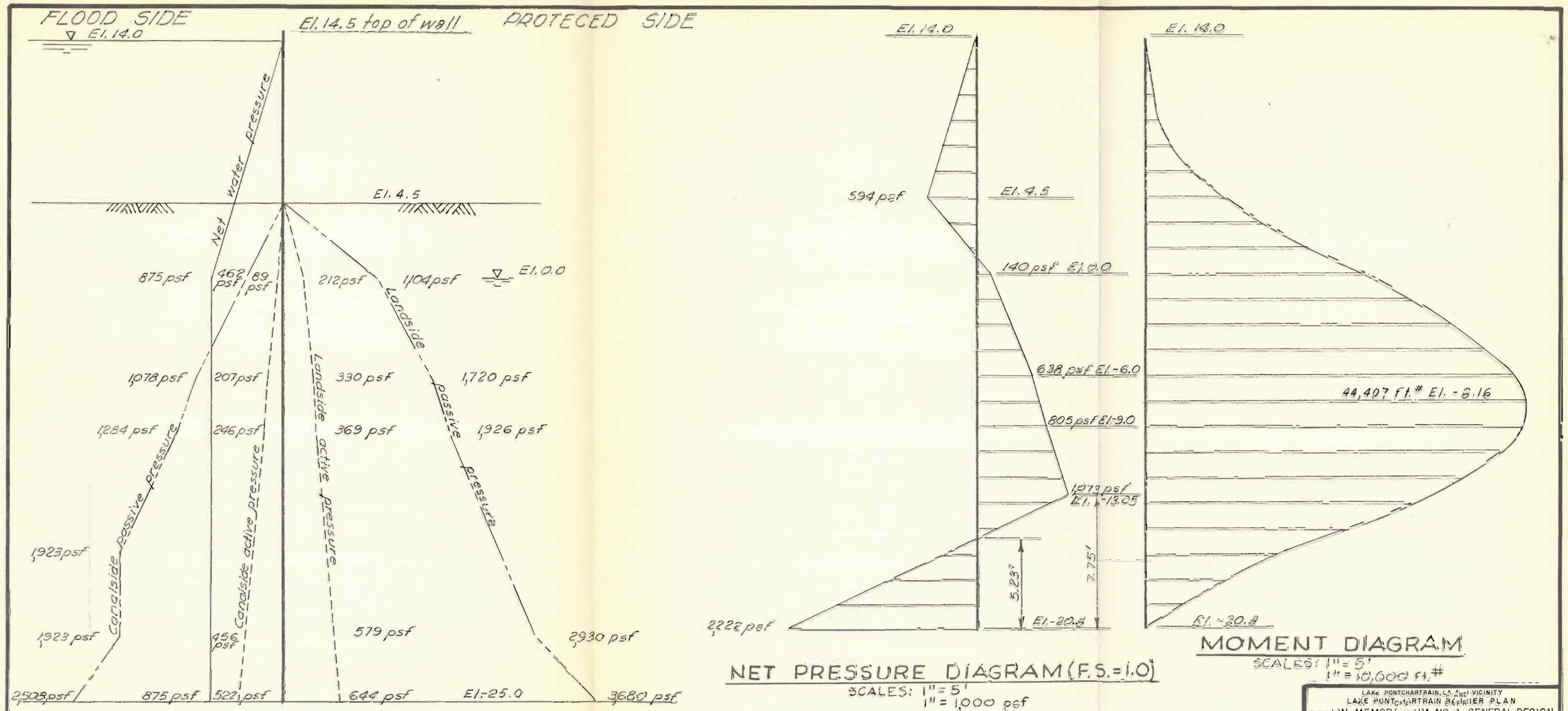
FLOOD SIDE
El. 14.0
El. 14.5 top of wall
PROTECED SIDE
Net water pressure
El. 4.5
875 psf
462 psf
89 psf
212 psf
1,104 psf
El. 0.0
Landside passive pressure
1,078 psf
207 psf
330 psf
1,720 psf
Landside active pressure
1,284 psf
246 psf
369 psf
1,926 psf
Landside passive pressure
Canalside active pressure
1,923 psf
1,923 psf
456 psf
579 psf
2,930 psf
2,508 psf
875 psf
522 psf
644 psf
El. -25.0
3,680 psf
PRESSURE DIAGRAM (F.S. = 1.0)
SCALES: 1" = 5'
1" = 1,000 psf
NOTE: With water at El. 14.5 max. moment = 52,595'# at El. -8.62
El. 14.0
594 psf
El. 4.5
140 psf El. 0.0
638 psf El. -6.0
805 psf El. -9.0
1,072 psf
El. -13.05
5.23'
7.75'
2,222 psf
El. -20.8
NET PRESSURE DIAGRAM (F.S. = 1.0)
SCALES: 1" = 5'
1" = 1,000 psf
El. 14.0
44,407 ft.# El. -6.16
El. -20.8
MOMENT DIAGRAM
SCALES: 1" = 5'
1" = 10,000 ft.#
STA. 59+30 TO STA. 59+49.5
STA. 59+96.75 TO STA. 64+05
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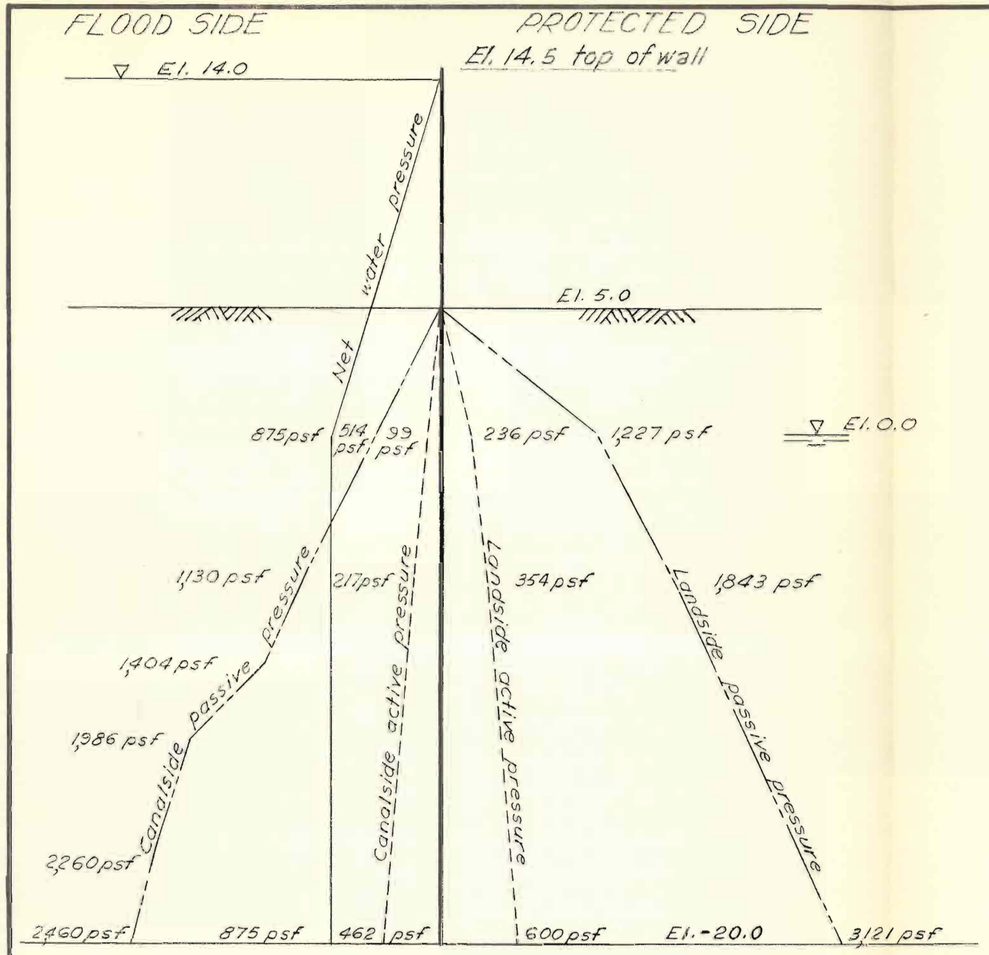
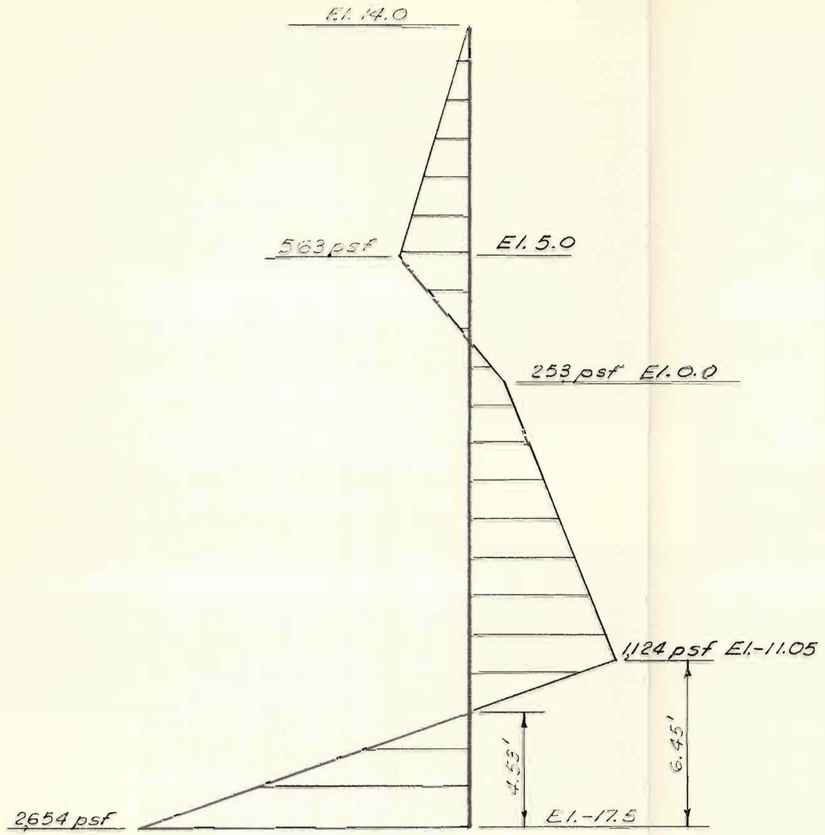
FIGURE 4-6



PRESSURE DIAGRAM (F.S.=1.0)

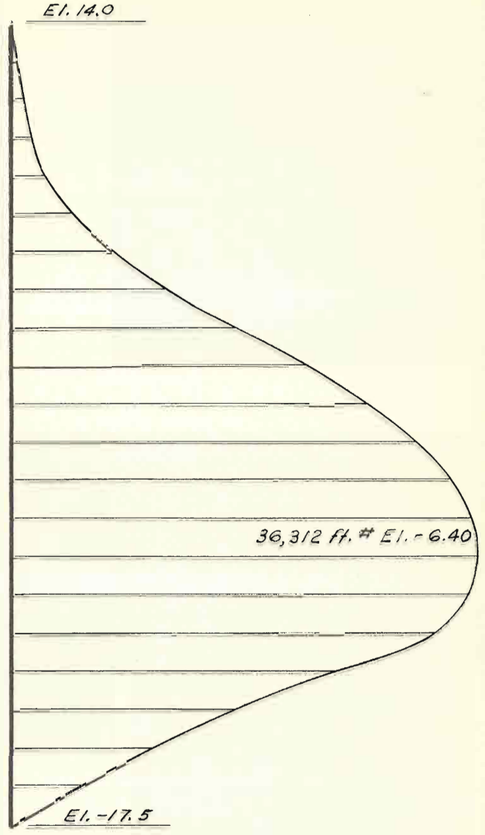
SCALES: 1" = 5'
1" = 1,000 psf

NOTE: With water at El. 14.5, max. moment = 43,166 at El. -7.1



NET PRESSURE DIAGRAM (F.S.=1.0)

SCALES: 1" = 5'
1" = 1,000 psf



MOMENT DIAGRAM

SCALES: 1" = 5'
1" = 10,000 ft.#

STA. 64+05 TO STA. 65+39.25
STA. 106+76.5 TO STA. 107+03.5

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FIGURE 4-7

LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave. to Lock

Sh. 1 of 12
Jan 67
CWR

Inverted T-Type Floodwall

Design Computations - Sta. 99+84 to Sta. 106+10

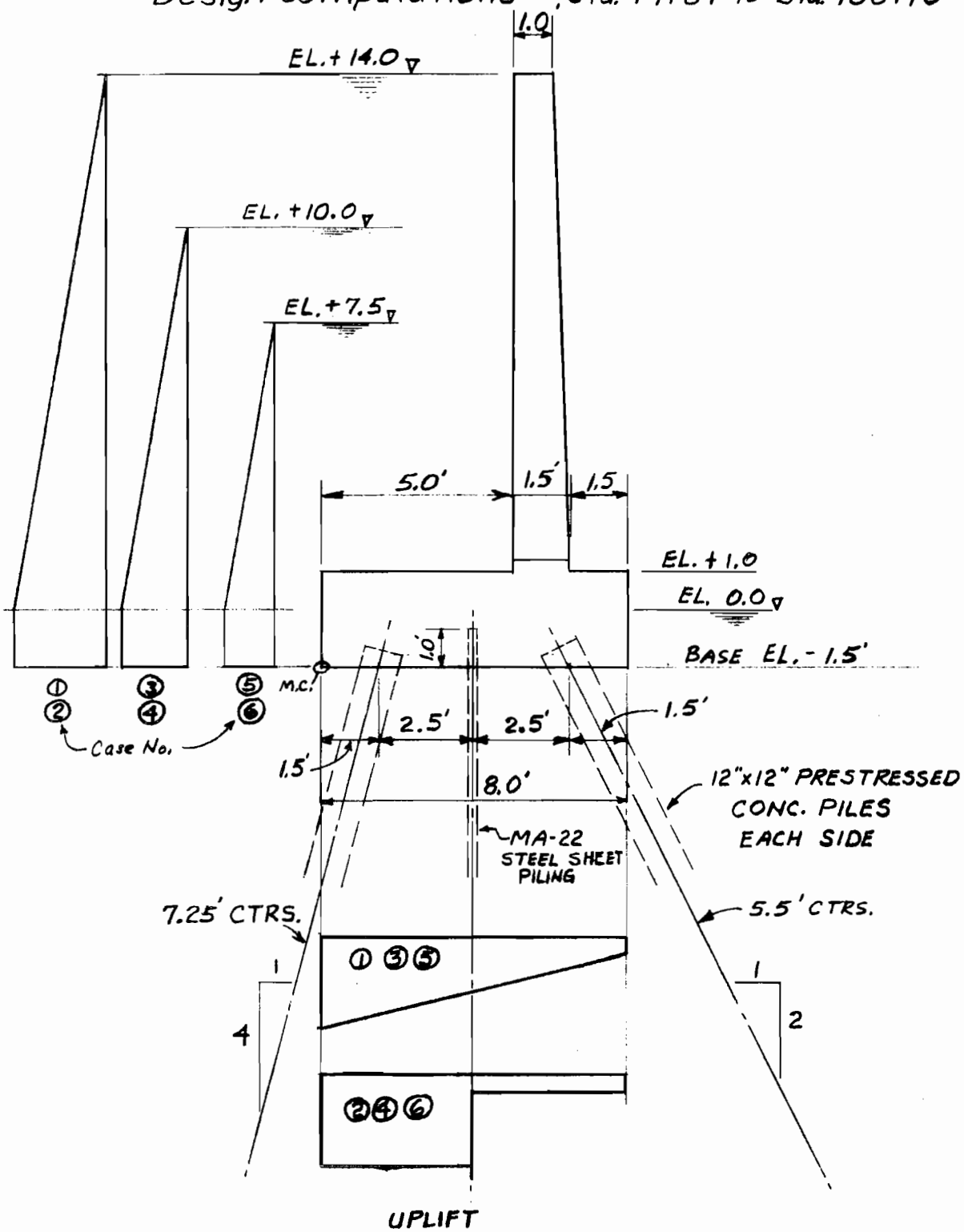


FIG. 4-8

Inverted T-Type Floodwall

Jan. 67

CWR

CASES ① & ②: 1' Unit length of wall

(STA. 99+84 TO STA. 106+10)

ITEM	COMPUTATION	H	V	x	M
Conc. Stem	1 x 13 x .15		1.950^K	5.50'	10.7250$'^K$
" "	½ x .5 x 13 x .15		0.4875	6.1667	3.00625
Conc. Base	2.5 x 8 x .15		3.00	4.00	12.00
Water on Base	5 x 13 x .0625		4.0625	2.50	10.15625
Horiz. Water	½ x 14 x 14 x .0625	6.125^K		3 3/6	37.77083
" "	1.5 x 14 x .0625	1.3125		0.75	0.98437
Sub-totals W/O uplift		7.4375	9.500^K		74.6427
Uplift ①	- 8 x 1.5 x .0625		- 0.750	4.0	- 3.0000
"	- ½ x 8 x 14 x .0625		- 3.500	8/3	- 9.3333
Case ① Totals		7.4375^K	5.250^K		62.3094$'^K$
Uplift ②	- 8 x 1.5 x .0625		- 0.750	4.0	- 3.0000
"	- 4 x 14 x .0625		- 3.500	2.0	- 7.0000
Case ② Totals		7.4375^K	5.250^K		64.6427$'^K$
CASES ③ & ④:					
Conc. Total	(from above)		5.4375	—	25.73125
Water on Base	5 x 9 x .0625		2.8125	2.50	7.03125
Horiz. Water	½ x 10 x 10 x .0625	3.1250^K		2 9/6	15.10417
" "	1.5 x 10 x .0625	0.9375		0.75	0.70313
Part Uplift	- 8 x 1.5 x .0625		- 0.750	4.0	- 3.0000
Sub-totals		4.0625	7.500		45.5698
Uplift ③	- ½ x 8 x 10 x .0625		- 2.500	8/3	- 6.66667
Case ③ Totals		4.0625^K	5.000^K		38.90313$'^K$
Uplift ④	- 4 x 10 x .0625		- 2.500	2.0	- 5.0000
Case ④ Totals		4.0625^K	5.000^K		40.5698$'^K$
CASES ⑤ & ⑥:					
Conc. Total	(from above)		5.4375	—	25.73125
Water on Base	5 x 6.5 x .0625		2.03125	2.50	5.07813
Horiz. Water	½ x 7.5 x 7.5 x .0625	1.75781		4.0	7.03125
" "	1.5 x 7.5 x .0625	0.70313		0.75	0.52734
Part uplift	- 8 x 1.5 x .0625		- 0.750	4.0	- 3.0000
Sub-totals		2.46094	6.71875		35.36797
Uplift ⑤	- ½ x 8 x 7.5 x .0625		- 1.8750	8/3	- 5.000
Case ⑤ Totals		2.46094	4.84375		30.36797
Uplift ⑥	- 4 x 7.5 x .0625		- 1.8750	2.0	- 3.75
Case ⑥ Totals		2.46094	4.84375		31.61797

FIG. 4-9

LAKE PONTCHARTRAIN, LA. & VICINITY

IHNC - West Side - Fla. Ave. to Lock — Sh. 3 of 12

Inverted T-Type Floodwall — Jan. 67

(STA. 99+84 TO STA. 106+10) — CWR

Moments and forces for 159.5' of wall:

Case	Moment	Vert.	Horiz.
1	9,938.349'k	837.375k	1,186.281k
2	10,310.512	837.375	1,186.281
3	6,205.049	797.50	647.969
4	6,470.883	797.50	647.969
5	4,843.691	772.578	392.52
6	5,043.066	772.578	392.52

For tension piles at 7.25' ctrs. and compression piles at 5.5' ctrs, a wall length of 159.5' was used in order to compute pile loads on a whole number of 22 tension and 29 compression piles. Pile loads were computed by the Hrennikoff Method * of analysis of pile foundations with batter piles, utilizing a G.E. 225 data processing system and program no. 41-G1-Z5-002 with minor modifications.

Design Data for 12"x12" prestressed concrete piles:

Tension Load = 46k Compression Load = 70k
Min. penetration to El. -64.0
Assumed pinned end and friction type piles
Modulus of subgrade soil reaction (K) = 135 psi

PILE LOADS

CASE NO.	TENSION			COMPRESSION		
	AXIAL FORCE	TRANSVERSE FORCE	TRANSVERSE DEFL.	AXIAL FORCE	TRANSVERSE FORCE	TRANSVERSE DEFL.
1	- 42.55k	- 1.68k	0.205"	67.80k	- 1.70	0.207"
2	- 45.79k	- 0.72k	0.087"	70.14k	- 0.65k	0.079"
3	- 10.00k	- 1.71k	0.208"	39.50k	- 1.76k	0.214"
4	- 12.32k	- 1.02k	0.125"	41.18k	- 1.01k	0.123"
5	+ 1.51k	- 0.62k	0.075"	28.71k	- 0.58k	0.071"
6	- 0.22k	- 0.10k	0.012"	29.96k	- 0.02k	0.003"

* Paper No. 2401 of A.S.C.E. Transactions - "Analysis of Pile Foundations with Batter Piles" by A. Hrennikoff.

FIG 4-10

DETERMINE ALLOWABLE TRANSVERSE LOAD ON PILE HEAD:

12" x 12" Std. Prestressed Conc. Piles $f_c' = 5{,}000$ psi
$f_c = .35 f_c' = 1{,}750$ psi $w = 150$ #/c.f. Max. Pile Loads = 70^k Comp.
& 46^k Ten.

Ref: 1. A.C.I. Bldg. Code (ACI 318-63)
2. Std. Prestressed Conc. Piles (10" to 24" Square) Design Sheet by Joint Committe of AASHO & PCI
3. Journal #3509, May '63, of Soil Mechanics and Fdns. Div. of A.S.C.E. "Laterally Loaded Piles in a Layered Soil System" by M.T. Davisson & H.L. Gill.

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$ modulus of subgrade soil reaction, $k = 135$ psi
Min. Prestress after losses = 700 psi
Eff. Prestress after losses = 840 psi } Ref. 2

$A = 12 \times 12 = 144 \square''$ $I = \frac{Ad^2}{12} = \frac{144(12)^2}{12} = 1{,}728$ in.^{4}

$E = (w)^{3/2}\, 33\sqrt{f_c'} = (150)^{3/2}(33)\sqrt{5{,}000} = 4.29 \times 10^6$ psi (Ref. 1)

$R = \sqrt[4]{\frac{EI}{k}} = \left(\frac{4.29 \times 10^6 \times 1728}{135}\right)^{1/4} = 86.0829''$

Ten. Piles:
$f_a = \frac{P_T}{A} = \frac{46{,}000}{144} = 319.44$ psi
$F_a = 700$ psi

Comp. Piles
$f_a = \frac{P_c}{A} = \frac{70{,}000}{144} = 486.11$ psi
$F_a = 1750 - 840 = 910$ psi

Max. Mom. Coef. = 0.50 (Ref. 3)
$M = .50\,RQ = .5 \times 86.0829\,Q$ "$^k \times 1000 = 43{,}041.45Q$"#
Q = Max. Transverse Load applied at pile head in kips
$S = \frac{bd^2}{6} = \frac{12(12)^2}{6} = 288$ in.^{3}
$f_b = \pm \frac{M}{S} = \frac{\pm 43{,}041.45Q}{288} = 149.4495Q$ psi

$F_b = 700$ psi $F_b = 910$ psi

$\frac{319.44 + 149.4495Q}{700} \leq 1$ $\frac{486.11 + 149.4495Q}{910} \leq 1$

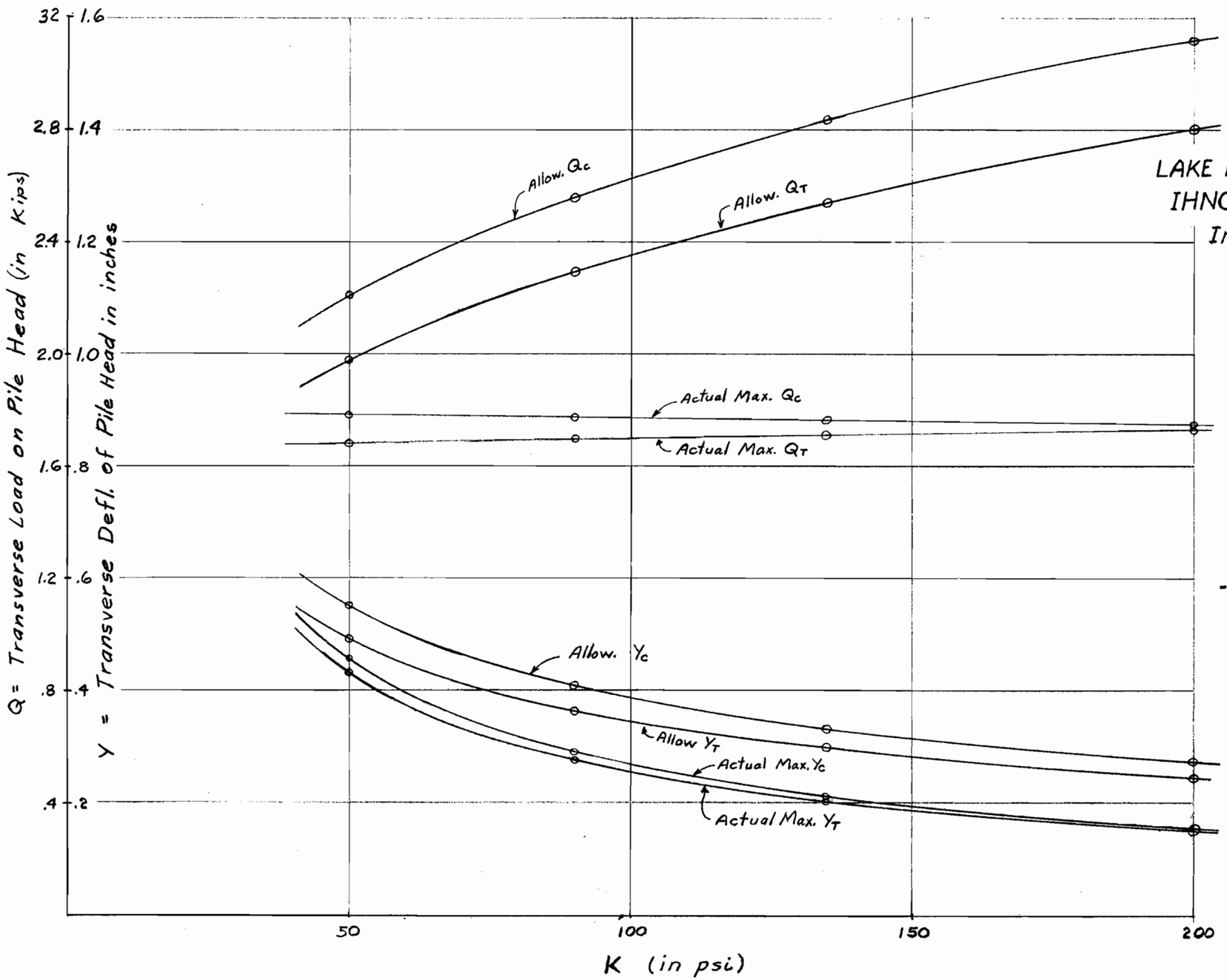
$Q_T = \frac{700 - 319.44}{149.4495} = \underline{2.546^k}$ $Q_c = \frac{910 - 486.11}{149.4495} = \underline{2.836^k}$

Defl. of Pile Head $Y = \frac{1.375\,QR^3}{EI}$

$Y = \frac{1.375(86.0829)^3 Q}{4.29 \times 1728 \times 10^3} = 118.3184 \times 10^{-3}\,Q$ (Ref. 3)

$Y_T = \underline{0.301''}$ $Y_c = \underline{0.336''}$

FIG 4-11



LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave. to Lock
Inverted T-Type Floodwall
(STA. 99+84 TO STA. 106+10)

Curves showing allowable and actual transverse loads and deflections for pile heads vs. modulus of subgrade soil reaction (K).

Note: Case III produced the maximum actual transverse loads and de--flections.

FIG. 4-12

LAKE PONTCHARTRAIN, LA. & VICINITY

IHNC - West Side - Fla. Ave. to Lock — Sh. 6 of 12

Inverted T-Type Floodwall — Jan. 67

(STA. 99+84 TO STA 106+10)

8' Base with 2 pile rows.
Approximate graphic analysis of pile loads by Vetter's method.

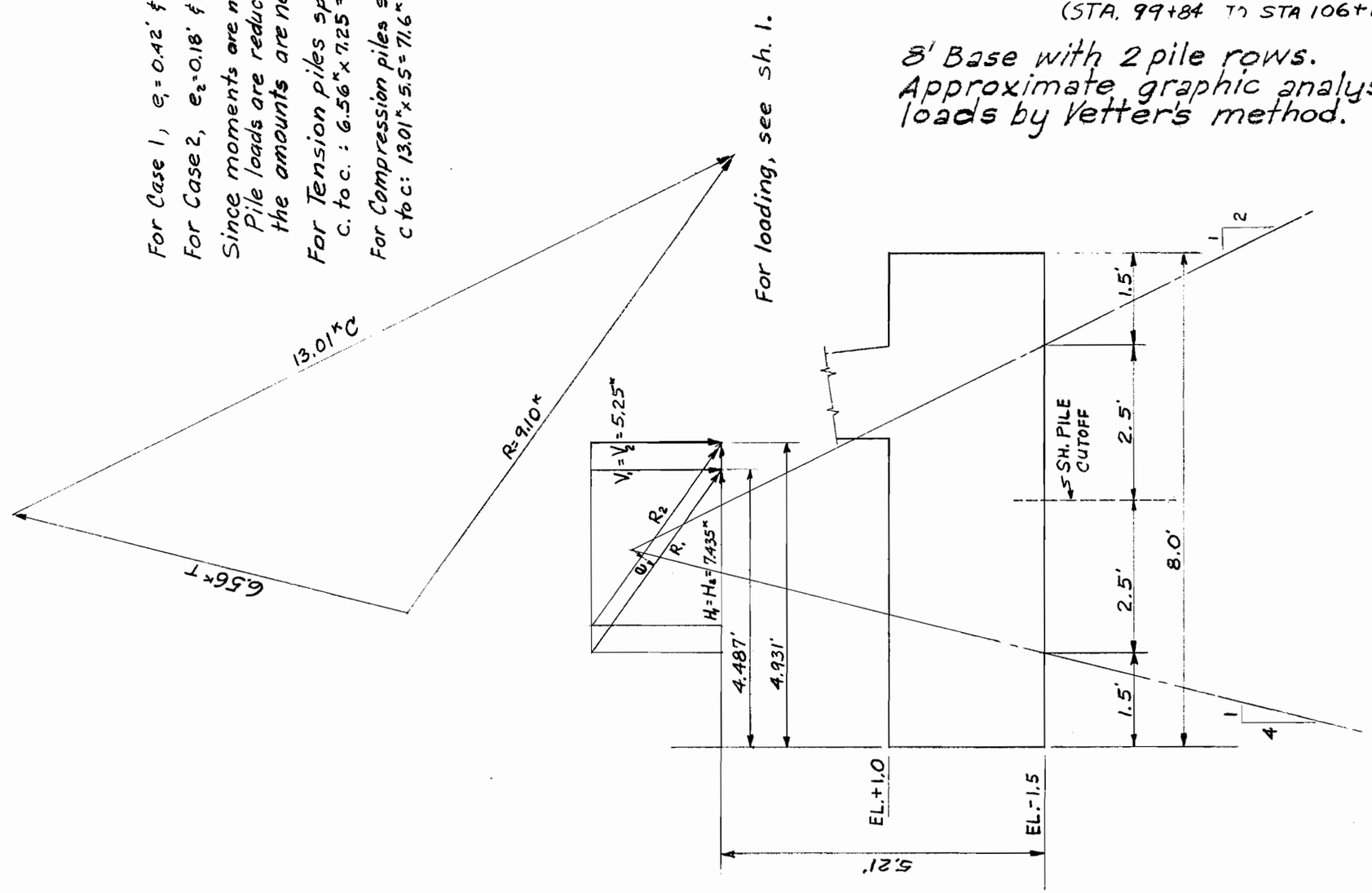
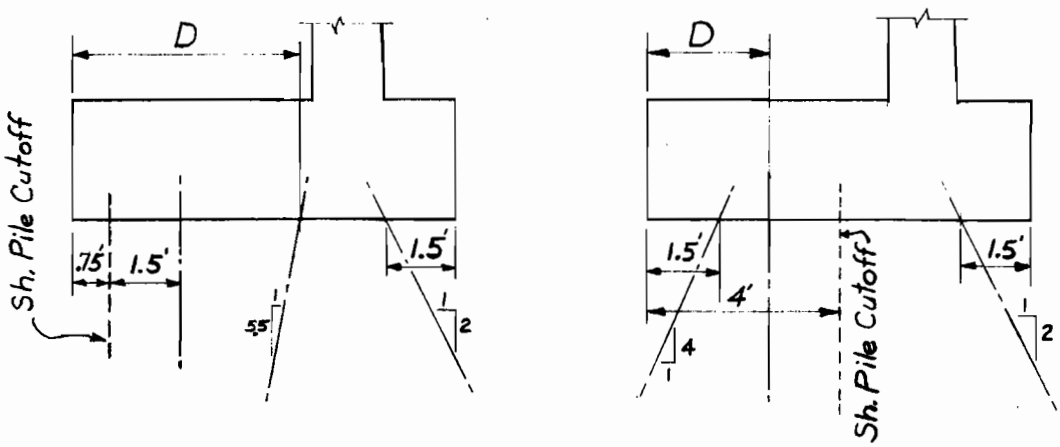


FIG. 4-13

LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave. to Lock
Inverted T-Type Floodwall

Alternate 8' Base With 3 Pile Rows:
(STA. 99+84 TO STA. 106+10) K = 135 psi



Arrangement A Arrangement B

Allow. Ten.: P = 46^k Q = 2.546^k Y = 0.301"
Allow. Comp.: P = 70^k Q = 2.836^k Y = 0.336"

	RUN NO.	D	CASE NO.	MAX. PILE LOADS/LF WALL ROW 1	ROW 2	ROW 3	MAX. PILE SPACING ROW 1	ROW 2	ROW 3
Arrangement A	1.	4.75'	1	+10.16^k	-13.48^k		6.88'	3.41'	
			2			+10.57^k			6.62'
	2.	5.5'	1		-9.84^k		(13.29)	4.68'	
			2	+4.98^k		+12.24^k	14.06'		5.72'
	3.	6.5'	1		-9.02^k		(14.93')	5.10'	
			2	+3.92^k		+12.65^k	17.88'		5.53'
Arrangement B	1.	2.5'	1	-6.25^k			7.37'	(18.00')	
			2			+12.37^k			5.66'
			3		+1.04^k			67.22'	
	2.	5.5'	1	-6.42^k			7.16'		
			2			+12.35^k		(17.94')	5.67'
			3		+1.67^k			41.79'	

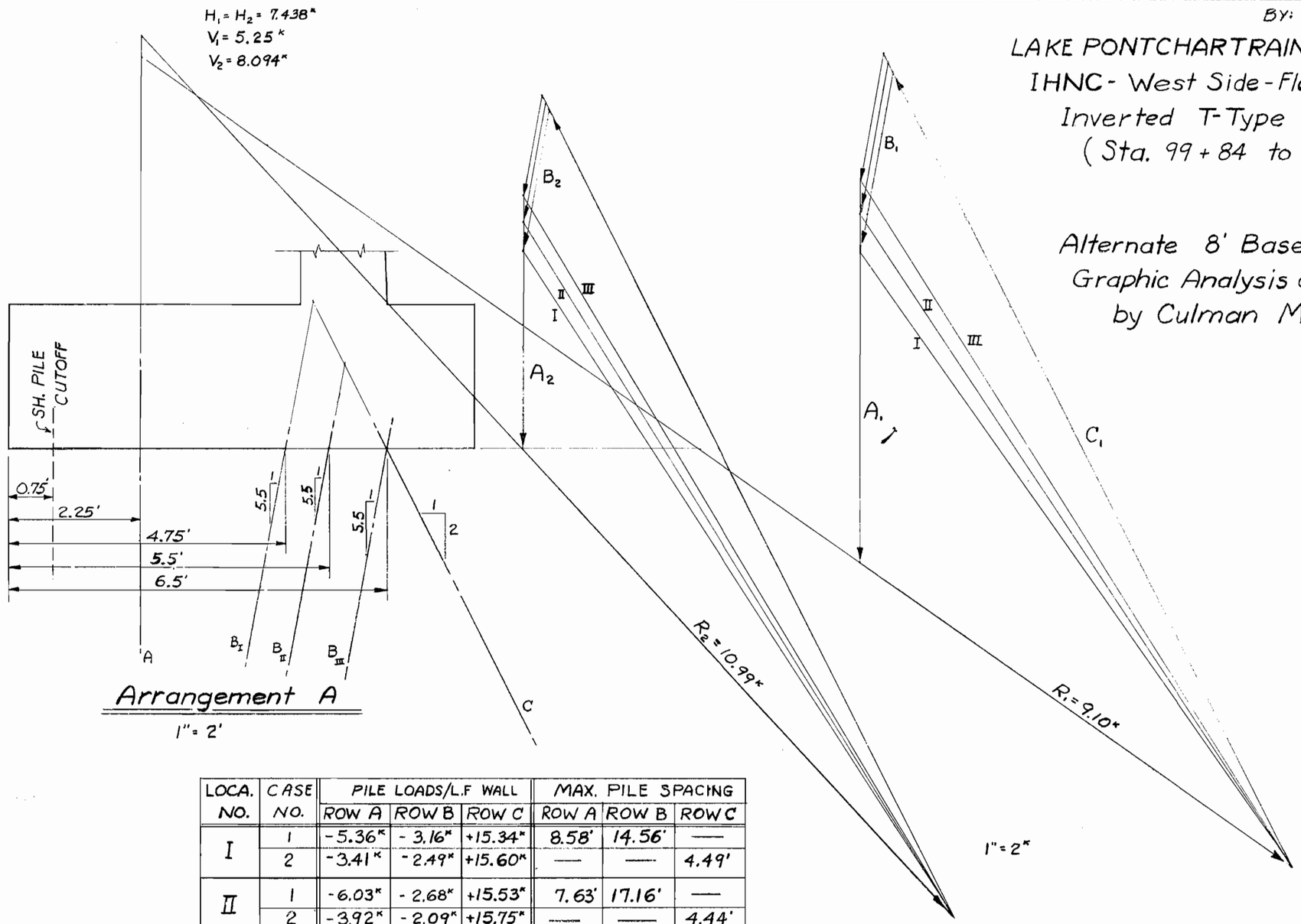
Note: Pile spacings in parentheses above were determined by max. allow. transverse defl. of pile head. All other spacings were determined by max. allow. axial loads on piles.

FIG. 4-14

BY: CWR Sh. 8 of 12

LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave. to Lock
Inverted T-Type Floodwall
(Sta. 99 + 84 to Sta. 106+10)
Jan. 67

Alternate 8' Base With 3 Pile Rows
Graphic Analysis of Pile Loads
by Culman Method



LOCA. NO.	CASE NO.	PILE LOADS/L.F WALL			MAX. PILE SPACING		
		ROW A	ROW B	ROW C	ROW A	ROW B	ROW C
I	1	-5.36ᵏ	-3.16ᵏ	+15.34ᵏ	8.58'	14.56'	—
	2	-3.41ᵏ	-2.49ᵏ	+15.60ᵏ	—	—	4.49'
II	1	-6.03ᵏ	-2.68ᵏ	+15.53ᵏ	7.63'	17.16'	—
	2	-3.92ᵏ	-2.09ᵏ	+15.75ᵏ	—	—	4.44'
III	1	-6.60ᵏ	-2.22ᵏ	+15.52ᵏ	6.97'	20.72'	—
	2	-4.38ᵏ	-1.87ᵏ	+15.90ᵏ	—	—	4.40'

FIG. 4-15

BY: C.W.R. Sh. 9 of 12

LAKE PONTCHARTRAIN, LA. & VICINITY

IHNC - West Side - Fla. Ave. to Lock

Inverted T-Type Floodwall

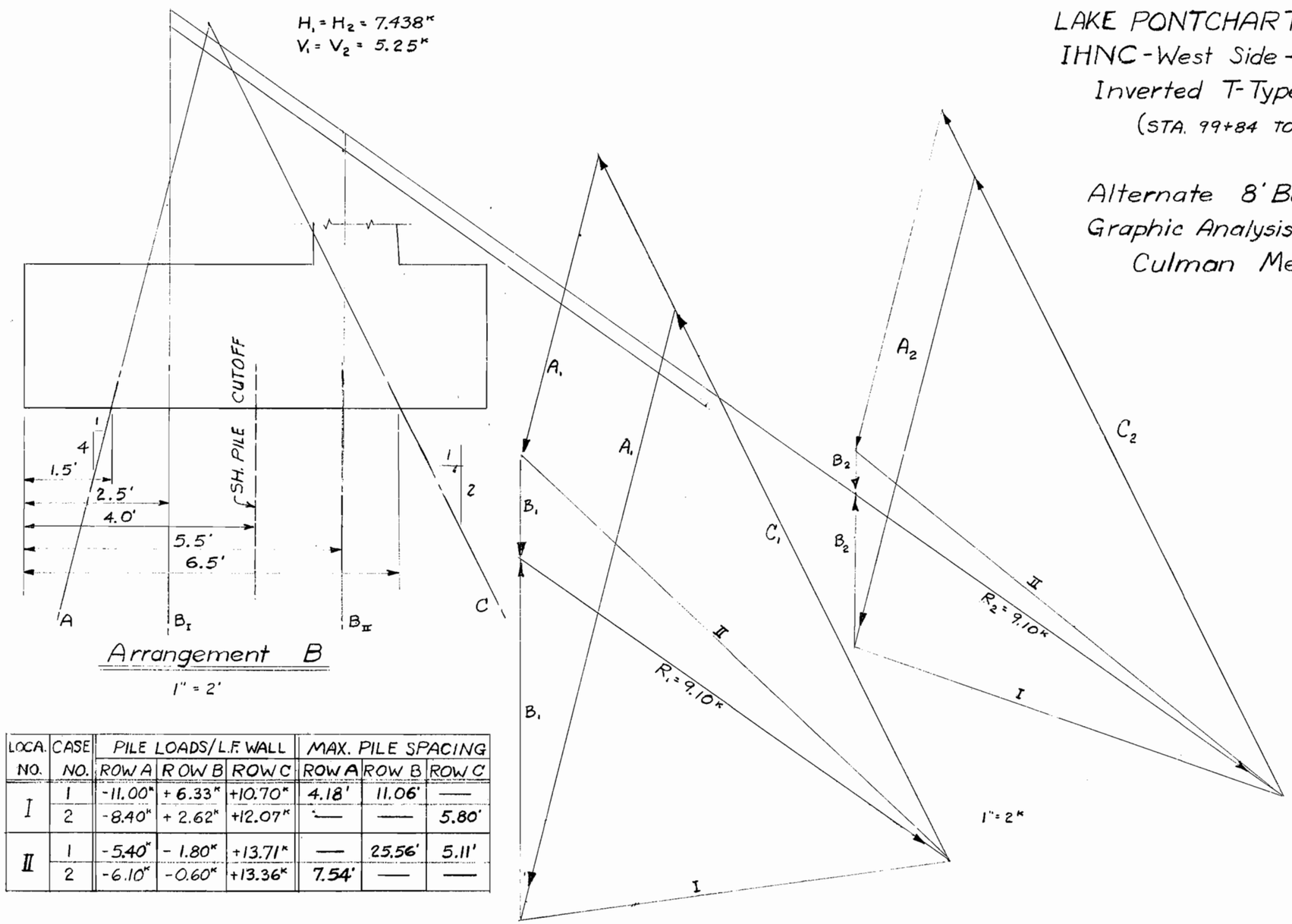
(STA. 99+84 TO STA. 106+10)

Jan. 67

Alternate 8' Base With 3 Pile Rows

Graphic Analysis of Pile Loads by

Culman Method



Arrangement B

1" = 2'

LOCA. NO.	CASE NO.	PILE LOADS/L.F. WALL			MAX. PILE SPACING		
		ROW A	ROW B	ROW C	ROW A	ROW B	ROW C
I	1	-11.00^k	+ 6.33^k	+10.70^k	4.18'	11.06'	—
	2	-8.40^k	+ 2.62^k	+12.07^k	—	—	5.80'
II	1	-5.40^k	- 1.80^k	+13.71^k	—	25.56'	5.11'
	2	-6.10^k	-0.60^k	+13.36^k	7.54'	—	—

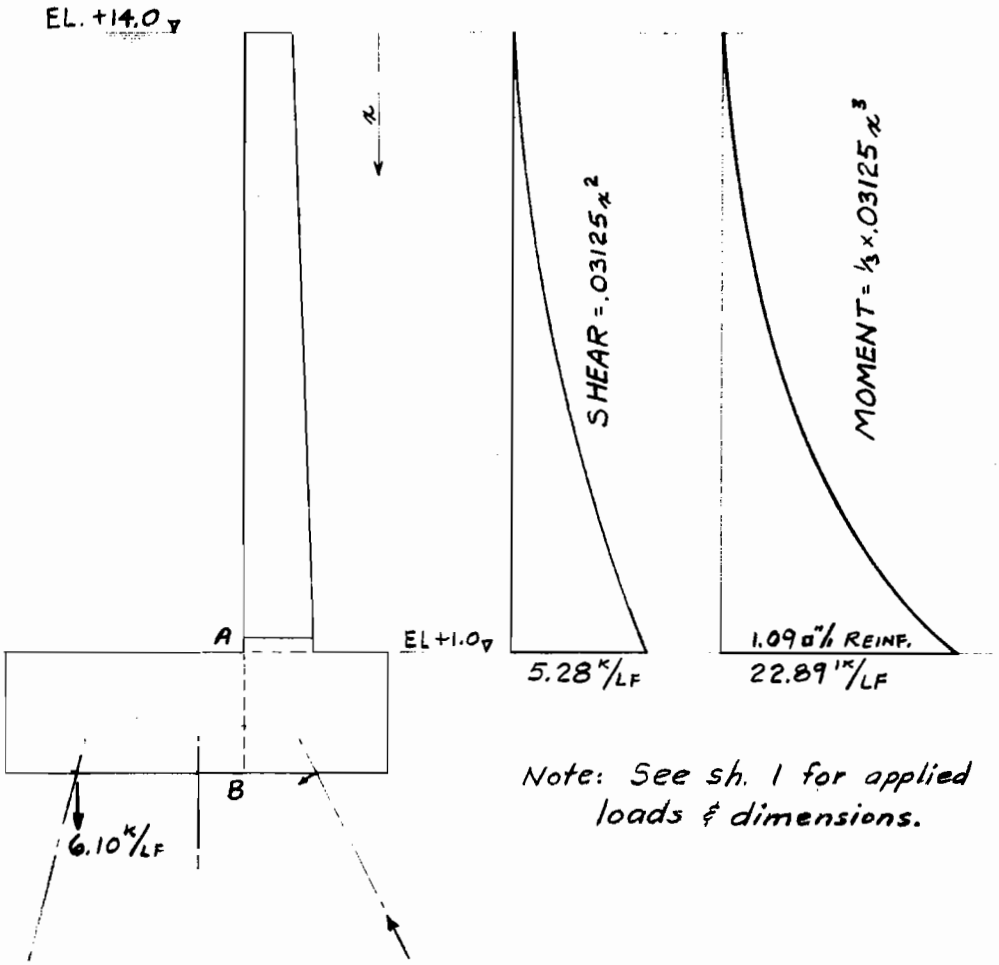
FIG. 4-16

LAKE PONTCHARAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave to Lock

CWR
Sh. 10 of 12
Jan. 67

Inverted T-Type Floodwall

Conc. Design: (Sta. 99+84 to Sta. 106+10)



Note: See sh. 1 for applied loads & dimensions.

SHEAR & MOMENT AT SEC. A-B FOR 1 L.F. OF WALL:

ITEM	COMPUTATION	V	x	M
PILE	NET VERT. COMP./7.25'	6.10K	3.5	21.36'K
WATER	5x13x.0625	4.06	2.5	10.16
CONC.	5x2.5x.15	1.88	2.5	4.69
	SUB-TOTALS W/O UPLIFT	12.04K		36.21'K
UPLIFT ①	-6.75 x 5 x .0625	-2.11	2.5	-5.27
"	-½ x 8.75 x 5 x .0625	-1.37	10/3	-4.56
	TOTALS W/UPLIFT ①	8.56K↓		26.38'K
UPLIFT ②	-4 x 14 x .0625	-3.50	3.0	-10.50
"	-5 x 1.5 x .0625	0.47	2.5	-1.17
	TOTALS W/UPLIFT ②	8.07K↓		24.54'K

0.70 □" REINF. PER L.F.

FIG. 4-17

(STA. 99+84 TO STA. 106+10)

Conc. Design (Cont.): $f_c' = 3{,}000$ psi $f_c = 0.35 f_c' = 1050$ psi
$f_s = 20$ ksi $n = 9.2$

Vert. Stem Reinf.

At Top of Base, $d = 15''$ $V = 5.28^k/'$ $M = 22.89'^k/'$ $b = 12''$

Shear $v = \frac{V}{bd} = 29.3$ psi < 60 O.K.

Reinf: #9 @ 12" $A_s = 1.00$ □"/' $\Sigma o = 3.5$"/'

$m = \frac{nA_s}{bd} = 0.0511$ $k = \sqrt{m^2 + 2m} - m = 0.273$
$j = 0.909$

$f_s = \frac{12M}{A_s jd} = 20.145$ ksi $\cong 20$ O.K.

$f_c = \frac{f_s k \times 1000}{n(1-k)} = 821$ psi < 1050 O.K.

$u = \frac{V}{\Sigma o jd} = 111$ psi < 233 O.K.

At 1'-2" Above Top of Base, $d = 14.46''$ $V = 4.38^k/'$ $M = 17.26'^k/'$

Shear $v = \frac{V}{bd} = 25.2$ psi < 60 O.K.

Reinf: #9 @ 24" & #6 @ 12" $A_s = 0.94$ □"/' $\Sigma o = 4.13$"/'

$m = \frac{nA_s}{bd} = 0.0498$ $k = \sqrt{m^2 + 2m} - m = 0.27$ $j = 0.91$

$f_s = \frac{12M}{A_s jd} = 16.7$ ksi < 20 O.K.

$f_c = \frac{f_s \times k \times 1000}{n(1-k)} = 672$ psi < 1050 O.K.

$u = \frac{V}{\Sigma o jd} = 80.6$ psi < 233 O.K.

∴ Add #6 @ 12 above constr. joint & cut off half of #9 bars @ 2'-9" above base (6" + 2'-3" for splice)

At 3'-6" Above Top of Base, $d = 13.38''$ $V = 2.82^k/'$ $M = 8.93'^k/'$

Shear $v = \frac{V}{bd} = 17.6$ psi < 60 O.K.

Reinf: #6 @ 12" $A_s = 0.44$ □"/' $\Sigma o = 2.36$"/'

$m = \frac{A_s n}{bd} = 0.0252$ $k = \sqrt{m^2 + 2m} - m = 0.201$ $j = 0.933$

$f_s = \frac{12M}{A_s jd} = 19.51$ ksi < 20 O.K.

$f_c = \frac{f_s k \times 1000}{n(1-k)}$ 533 psi < 1050 O.K.

$u = \frac{V}{\Sigma o jd} = 95.7$ psi < 351 O.K.

∴ Cut off Balance of #9 bars @ 4'-8" above base (3'-6" + 1'-2")

For Vert. Bars in land face Min. $A_s = .002bd$
at base, $d = 15''$ Min. $A_s = 0.36$ □"/'
Use #6 @ 12" $A_s = 0.44$ □"/'

For Horiz. Bars Temp. Steel $= .002bD$
at base, $d = 18''$ $A_s = 0.432$ □"/'
Use #4 @ 12" ea. face

FIG. 4-18

Inverted T-Type Floodwall
(STA. 99+84 TO STA 106+10)

Conc. Design (Cont.): Base Reinf.

At sec. A-B: $V = 8.56^{k}/'$ $M = 26.38'^{k}/'$

$d = 25''$ Try #6 @ 7½" $A_s = 0.70\,\square''/'$ $\Sigma_o = 3.8''/'$

$m = \frac{nA_s}{bd}$ $\frac{9.2 \times 0.70}{12 \times 25} = 0.021467$ $k = 0.1869$ $j = 0.9377$

$f_s = \frac{12M}{A_s jd} =$ 19.3 ksi < 20 O.K.

$f_c = \frac{f_s}{n} \times \frac{k}{(1-k)} \times 1000 =$ 482 psi < 1050 O.K.

Shear $v = \frac{V}{bd} =$ 28.5 psi < 60 O.K.

Bond $u = \frac{V}{\Sigma_o jd} =$ 96.1 psi < 248 O.K.

∴ Use #6 @ 7½" ctrs.

Longitudinal Reinf. (Ten. Piles @ 7.25' ctrs.)

Max. $V = 8.56 \times \frac{1}{2} \times 7.25 = 31.03^{k}$
Max. $M = \frac{1}{12} \times 8.56\,(7.25)^2 = 37.49'^{k}$

$d = 25.5''$ Assume $b = 36''$ & $j = .92$

Req'd. $A_s = \frac{12M}{f_s jd} = 0.96\,\square''$

Per ACI Code 318-63, Sec. 911, increase A_s

$A_s = 0.96 \times \frac{4}{3} = 1.28\,\square''$

Use #6 @ 12" ctrs. top & bottom

Ten. Pile Anchorage: Max. Pile Load in Tension = 46^{k}

Req'd. $A_s = 46/20 = 2.30\,\square''$

Use 4 - #7 Bars w/ Std. Hooks $A_s = 2.40\,\square''$

Compression Pile Anchorage:

Use 4 - #7 Bars w/o Hooks

FIG. 4-19

LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - West Side - Fla. Ave. to Lock
Inverted T-Type Floodwall

CWR
Sh. 1 of 2
Jan. 67

Allowable Transverse Loads and Deflections for Pile Heads:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad f_a = \frac{P_a}{A} \qquad f_b = \frac{M}{S}$$

$$Y = \frac{1.375\,Q\,R^3}{EI}$$

Constant k for Sta. 42+00 to Sta. 99+50

$K = 90\,psi \quad A = 144\,\square'' \quad I = 1{,}728\ in.^4 \quad E = 4.29 \times 10^6\,psi$

$$R = \sqrt[4]{\frac{EI}{K}} = \left(\frac{4.29 \times 1{,}728 \times 10^6}{90}\right)^{1/4} = 95.2664''$$

$M = 0.5RQ \qquad S = 288\ in.^3 \qquad f_b = \pm \frac{M}{S} = \frac{.5 \times 95.2664\,Q}{288} \times 10^3$

$f_b = \pm\ 165.393\ Q \qquad Y = \frac{1.375(95.2664)^3}{4290 \times 1728} Q = 160.3692 \times 10^{-3} Q$

$F_a = F_b = 700\,psi$ for Tension Piles

$F_a = F_b = 910\,psi$ for Compression Piles

For Sta. 42+00 to Sta. 59+00

Max. Allow. Pile Loads: 37ᵏ Tension & 76ᵏ Compression

Ten. Piles:

$f_a = \frac{37{,}000}{144} = 256.94\,psi$

$\frac{256.94 + 165.393\,Q}{700} \leq 1$

$Q_T \leq \frac{700 - 256.94}{165.393} = 2.679^k$

$Y_T \leq 0.430''$

Comp. Piles:

$f_a = \frac{76{,}000}{144} = 527.778\,psi$

$\frac{527.778 + 165.393\,Q}{910} \leq 1$

$Q_c \leq \frac{910 - 527.778}{165.393} = 2.311^k$

$Y_c = 0.371''$

For Sta. 66+00 to Sta 99+50

Max. Allow. Pile Loads: 44ᵏ Tension & 70ᵏ Compression

Ten. Piles

$f_a = \frac{44{,}000}{144} = 305.56\,psi$

$\frac{305.56 + 165.393\,Q}{700} \leq 1$

$Q_T \leq \frac{700 - 305.56}{165.393} = 2.385^k$

$Y_T = 0.382''$

Comp. Piles

$f_a = \frac{70{,}000}{144} = 486.11\,psi$

$\frac{486.11 + 165.393\,Q}{910} \leq 1$

$Q_c \leq \frac{910 - 486.11}{165.393} = 2.563^k$

$Y_c = 0.411''$

FIG. 4-20

LAKE PONTCHARTRAIN, LA. & VICINITY CWR

IHNC.- West Side - Fla. Ave. to Lock Sh. 2 of 2

Inverted T-Type Floodwall Jan. 67

Allow. Loads & Transverse Deflections for Pile Heads:

Stations	Tension			Compression		
	P_T	Q_T	Y_T	P_c	Q_c	Y_c
42+00 to 59+00	37^k	2.679^k	0.430"	76^k	2.311^k	0.371"
66+00 to 99+50	44^k	2.385^k	0.382"	70^k	2.563^k	0.411"
99+50 to 108+00	46^k	2.546^k	0.301"	70^k	2.836^k	0.336"

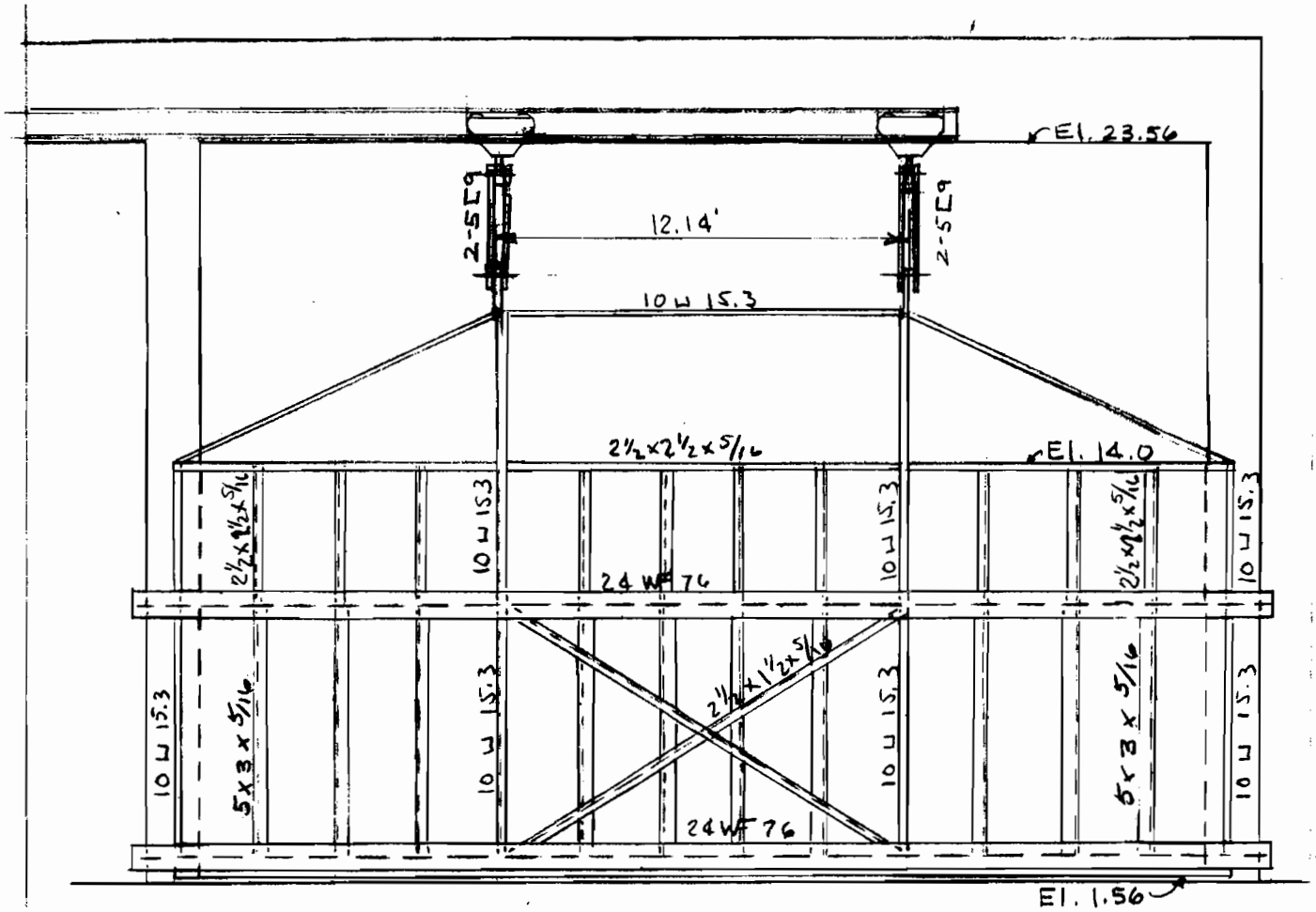
Pile Spacing and Computed Max. Pile Loads

STATION LIMITS	TOP BASE EL.	PILE SPACING		CRIT. CASE NO.	MAX. PILE LOADS					
					TENSION			COMPRESSION		
		TEN.	COMP.		P	Q	Y	P	Q	Y
44+12 TO 49+68	+2.75	7.5'	6.0'	1		-1.67^k	.276"		-1.71^k	.282"
				2	-30.75^k			58.51^k		
65+39.25 TO 88+19.25	+1.0	7.0'	5.5'	2	-44.21^k			70.14^k		
				3		-1.67^k	.276"		-1.75^k	.288"
89+06 TO 95+30.25	+1.0	7.75'	5.0'	2	-43.43^k			68.81^k		
				3		-0.78^k	.129"		-0.77^k	.127"
95+67.25 TO 99+39	+1.0	7.0'	5.5'	2	-44.21^k			70.14^k		
				3		-1.67^k	.276"		-1.75^k	.288"
99+84 TO 106+10	+1.0	7.25'	5.5'	2	-45.79^k			70.14^k		
				3		-1.71^k	.208"		-1.76^k	.214"
106+55 TO 106+77	+2.0	7.25'	5.5'	2	-45.79^k			70.14^k		
				3		-1.71^k	.208"		-1.76^k	.214"

FIG 4-21

LAKE PONTCHARTRAIN LA & VIC
IHNC WEST SIDE FLA AVE TO LOCK
GATE #10

Sheet 1 of 11
Computed by HLB
Jan 67
Checked by [illegible]

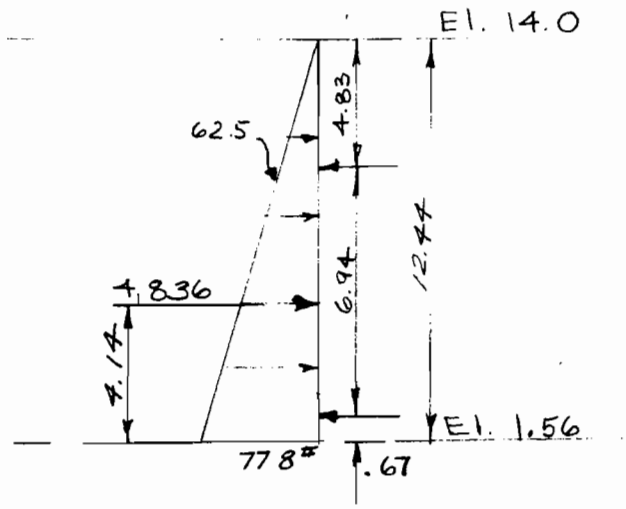


ELEVATION AT STA. 73+60
Scale: 3/16" = 1'-0"

Figure 4-22

LAKE PONTCHARTRAIN LA & VIC
IHNC WEST SIDE FLA AVE TO LOCK
GATE #10

Computed by HCB
Jan. 6 70
Checked by WM.



$R_T = \frac{4,836 \times 3.47}{6.94}$

$= 2,418^{\#}$

$R_B = \frac{4,836 \times 3.47}{6.94}$

$= 2,418$

Allow T Bending 20,000#/□"

Top & Bottom Girders

SPAN 31.08 LOAD 2,418#/' or 2.418k/'

$M_{max} = \frac{1}{8} WL^2$

$M_{max} = \frac{2.418\,(31.08)^2}{8}$

$= 291.96'^k$

$S = \frac{291.96 \times 12}{20} = 175.18$

USE 24 WF 76 , I = 2096.4

$\Delta = \frac{.013\,(2.418 \times 31.08)\,373^3}{(30 \times 10^3)\,2096.4}$

$\Delta = .81$ in

Skinplate:

Use 3/8" PLATE

I = .053 , S = .28

Figure 4-23

LAKE PONTCHARTRAIN LA & VIC
IHNC WEST SIDE, FLA AVE TO LOCK
GATE #10

Sheet 8 of 11
Computed by HIB
Jan. 69
Checked by WM

Skinplate:

Allow T Bending 20,000#/□"

Max Load = (12.44 − .67) 62.5 = 736 #/□'

$M_{max\ int.} = \frac{1}{12}(736)L^2 = \frac{.28\,(20{,}000)}{12}$

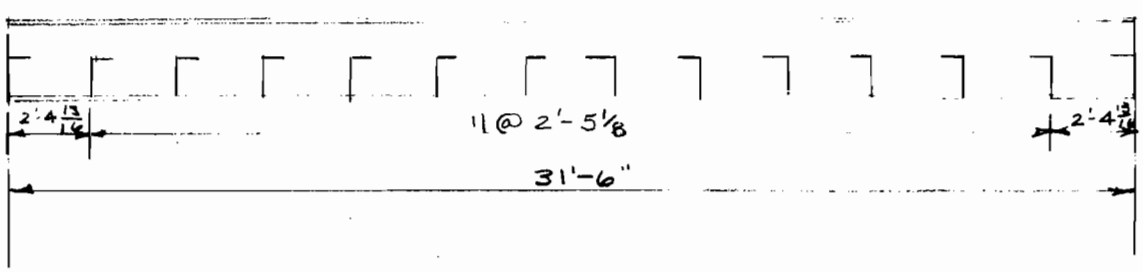
$L^2 = 7.66$

$L = 2.77$

$M_{max\ ext.} = \frac{1}{10}(736)L^2 = \frac{.28\,(20{,}000)}{12}$

$L^2 = 6.38$

$L = 2.53$



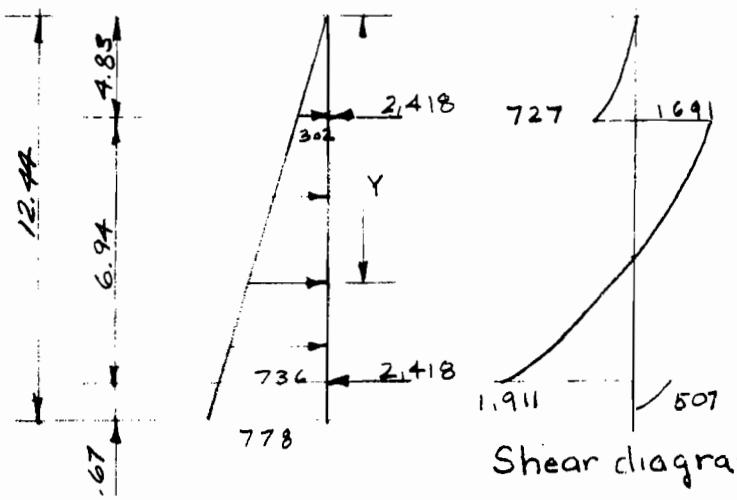
INT MOM. = $\frac{736(2.4271)^2}{12} = 361$ $f_b = \frac{361\,(12)}{.28} = 15{,}471$

END MOM = $\frac{736(2.401)^2}{10} = 424$ $f_b = \frac{424\,(12)}{.28} = 18{,}171$

Figure 4-24

LAKE PONTCHARTRAIN LA & VIC.
IHNC WEST SIDE, FLA AVE TO LOCK
GATE # 10

Computed by HLB
Jan. 67
Checked by LDM.



Pt of Zero Shear

$\frac{62.5\,Y^2}{2} = 2{,}418^{\#}$

$Y^2 = 77.3$

$Y = 8.8$

$M_{max} = 1{,}691\,(3.97).667 - 727\,\frac{(4.83)}{3}$

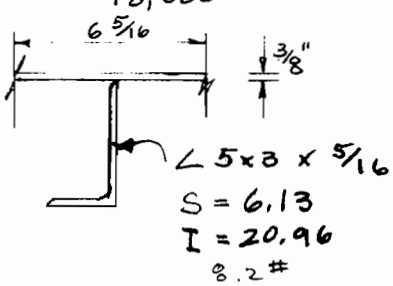
$= 4{,}478 - 1{,}170$

$= 3{,}308$

SPACING OF LOWER VERTICALS @ 2.43

$M_{MAX} = 3{,}308\,(2.43) = 8{,}038'^{\#}$ Allow $\sigma = 18{,}000\,\#/\square''$

$S = \frac{8{,}038 \times 12}{18{,}000} = 5.36$



$fb = \frac{8{,}038 \times 12}{6.13} = 15{,}735$

$\Delta = \frac{.013 \times 3602 \times 2.43 \times 83.3^3}{30 \times 10^6 \times 20.96}$

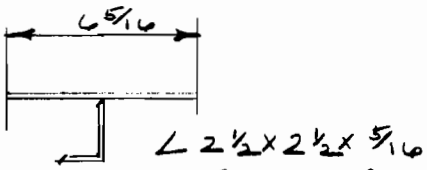
$\Delta = .104\,in$

SPACING OF UPPER VERTICALS @ 2.43

$M = 302 \times \frac{4.83}{2} \times \frac{4.83}{3} = 1{,}174'^{\#}$ $M_T = 1{,}174\,(2.43)$

$= 2{,}853'^{\#}$

$S = \frac{2853 \times 12}{18{,}000} = 1.90\,in^3$ Allow $\sigma = 18{,}000\,\#/\square''$



$fb = \frac{2853 \times 12}{2.21} = 15{,}491\,psi$

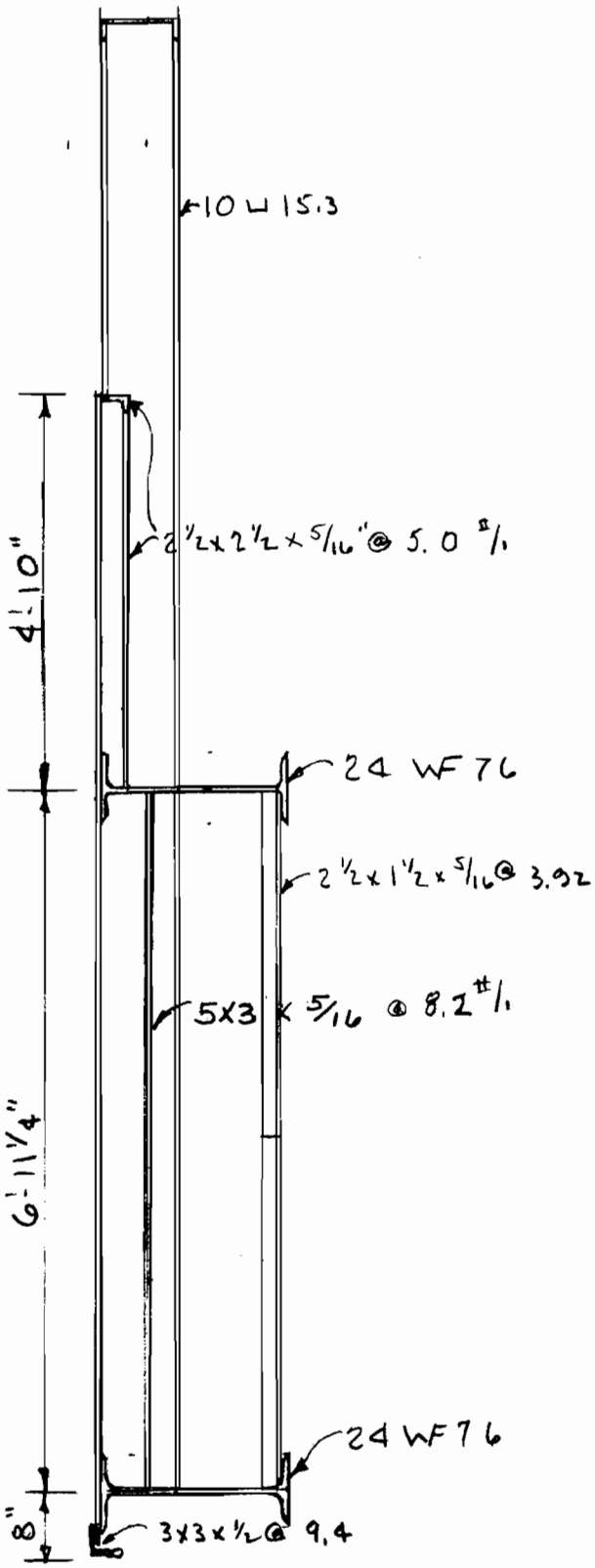
$\Delta = \frac{.013 \times 1772 \times 58^3}{30 \times 10^6 \times 4.29}$

$\Delta = .035\,in$

Figure 4-25

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE #10

Sheet 5 of 11
Computed by HLB
Jan. 64
Checked by [illegible]



(68.31) 15.3 = 1,045.1
10 (5.0) 4.83 = 241.5
10 (8.2) 6.94 = 569.1
2 (3.92) 13.5 = 105.8
2 (76) 33.75 = 5,130.0
(180) 31.5 = 5,670.0
(5.0) 31.5 = 157.5
(9.4) 31.5 = 296.1
13,215.1

1,045.1 (5.38) = 5,622.6
241.5 (1.12) = 270.5
569.1 (3.20) = 1,821.1
105.8 (22.66) = 2,397.4
5,130.0 (12.31) = 63,150.3
5,670.0 (.19) = 1,077.3
157.5 (1.12) = 176.4
296.1 (.56) = 165.8
74,681.4

$$\bar{X} = \frac{74,681.4}{13,215.1} = 5.65 \text{ in.}$$

IMPACT LOAD
13.2 x 1.25 = 16.5^{k}

Figure 4-26

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE #10.

Sheet 6 of 11
Computed by HIB
Jan. 64
Ckd by A.R.

HANGGERS :

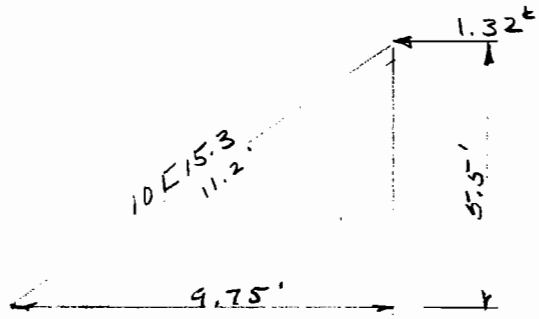
TOTAL LOAD OF GATE + IMPACT = 16.5^k
2-10 [15.3 HANGGERS

AREA OF STEEL = 2 X 4.47 = 8.94 □"

$\sigma = \frac{16.5}{8.94} = \underline{1.85 \text{ ksi}}$ ok allow σ = 18,000 #/□"

CHECK DIAGONAL STRUTS :

SIDESWAY DUE TO IMPACT = 10% of 13.2^k
= 1.32^k



$P = \frac{11.2}{9.75} \times 1.32$

$P = 1.52^k$

$\frac{L}{r} = \frac{11.2 \times 12}{.72}$

$\frac{L}{r} = 187$ ALLOWABLE σ = 4.27 ksi

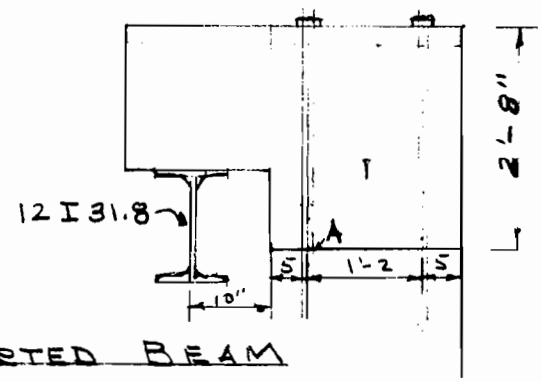
$\frac{P}{A} = \frac{1.52}{4.47} = \underline{.340 \text{ ksi} \quad \text{o.k.}}$

Figure 4-27

LAKE PONTCHARTRAIN LA & VIC
IHNC WEST SIDE FLA AVE TO LOCK
GATE 10

Sheet 7 of 11
Computed by HLB
Jan. 69
ckd by A.H.

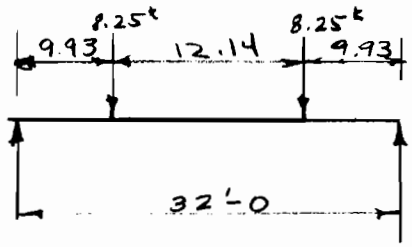
DESIGN OF GANTRY GIRDER :



Area Conc.
2.67 x 2 = 5.34
1.5 x 1.5 = 2.25
7.59 □'

SIMPLY SUPPORTED BEAM

SPAN = 32'-0



M = 8.25k x 9.93
M = 81.92'k

DL = 7.59 x .15 = 1.14 k/'
+ I Beam = .03
1.17 k/'

DL Mom = $\frac{1}{8}$ x 1.17 x $(32)^2$ = 149.76 k'
+ 81.92
Tot. Mom. = 231.68'k

$M = Kbd^2$

b = 24"
K = 152

$d^2 = \frac{231.68 \times 12{,}000}{152 \times 24}$

$d^2 = 762$

d = 27.6" o.k. Use d = 28"

FOR BAL. BEAM

Figure 4-28

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE 10

Sheet 8 of 11
Computed by
Jan 67
Ckd by A.R.

ANCHOR BOLTS

ΣM @ Pt. A.

$13.37^k \times 15'' = 200.55^{k''}$

$\frac{11.80}{2} \times 15 = 88.50$

$-\frac{25.6}{2} \times 7'' = -89.6$

Sub Tot. $199.45^{''k}$

WIND $\frac{4.27}{2} \times 16 = 34.24^{''k}$

Tot $= 233.69^{''k}$

Tension on Bolt:

$F \times 14'' = 233.69$

$F = 16.7^k$

Max reaction due to crane when one trolley, directly under support.

$P = 8.25^k + \left(\frac{32.0 - 12.14}{32}\right) \times 8.25$

$8.25 + \frac{19.86}{32} \times 8.25$

$P = 13.37^k$

Conc. $= 2.67 \times 2 \times 32 \times .15 = 25.6^k$

$1.5 \times 1.5 \times 32 \times .15 = 10.8^k$

I beam $= 32 \times .0318 = 1.0$

WIND $= .05^{k/\square'} \times 2.67 \times 32 = 4.27^k$

Try $1\frac{1}{2}''\phi$ Bolt Area $= 1.767\,\square''$

$\sigma = \frac{16.7}{1.767} = 9.45^{k/\square''}$ ok.

Embeddment:

$u = \frac{1.7\sqrt{f'_c}}{1.5}$

$u = \frac{1.7 \times \sqrt{3,000}}{1.5} = \frac{1.7 \times 54.7}{1.5}$

$u = 62.0$ psi < 160

$\Sigma o = 1.5\pi = 4.71''$

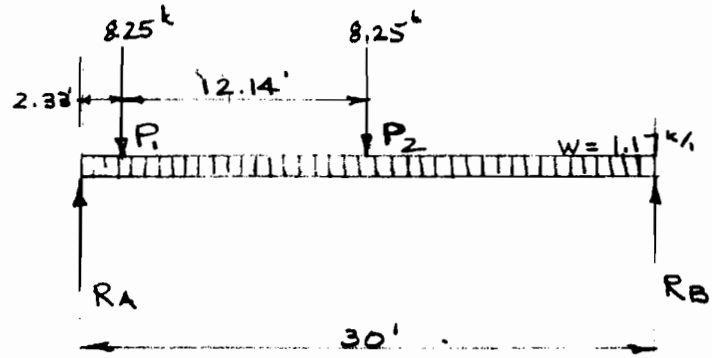
$L = \frac{16.7^k}{4.71 \times .062}$

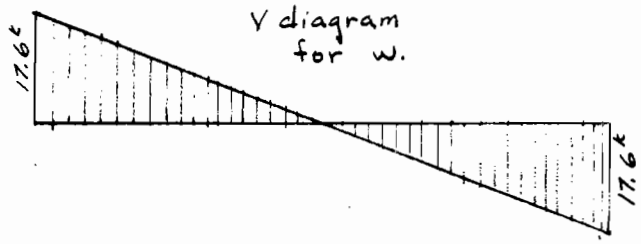
$L = 57.2$ in. Use 5'-0

Figure 4-29

$$w = \begin{cases} 2.67 \times 2 \times .15 = .80 \\ +1.5 \times 1.5 \times .15 = .34 \\ + \quad 0.03 \end{cases}$$

$w = 1.17^{k}/_{'}$



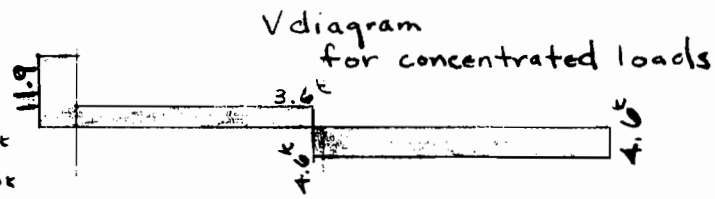


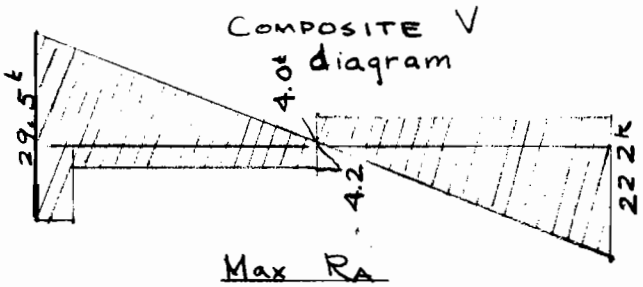
TORSION:

MOMENT

$.37^{k} \times 30 \times 22'' = 244.2''^{k}$

$16.5^{k} \times 22'' = 363.0''^{k}$





V@A due to Torsion:

$$\frac{244.2}{2} = 122.1$$

$$+ \frac{21.60}{30} \times 363 = 261.4$$

$$383.5''^{k}$$

$$v_a = \frac{T}{k_2 b^2 h}$$

$$V_A = \frac{383{,}500''\#}{.21 \times 24^2 \times 32} = 99.1\ \#/\square''$$

Figure 4-30

LAKE PONTCHARTRAIN LA & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE #10

TORSION @ P_1

$M = 122.1 + 261.4 - 19.0$ $\qquad .37 \times 2.33 \times 22 = 19.0''^k$

$M = 364.5''^k$

$V_{P_1} = \dfrac{364{,}500''^{\#}}{.21 \times 24^2 \times 32} = 94.2\ \#/\square''$

TORSION @ P_2 = Max Tors @ Center w/ P_2 @ Center

$M = 363 \times \dfrac{8.93}{30}$

$M = 108.1''^k$

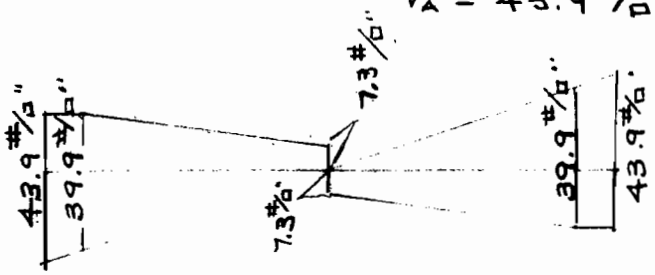
$V_{P_2} = \dfrac{108{,}100}{.21 \times 24^2 \times 32}$

$V_{P_2} = 27.9\ \#/\square''$

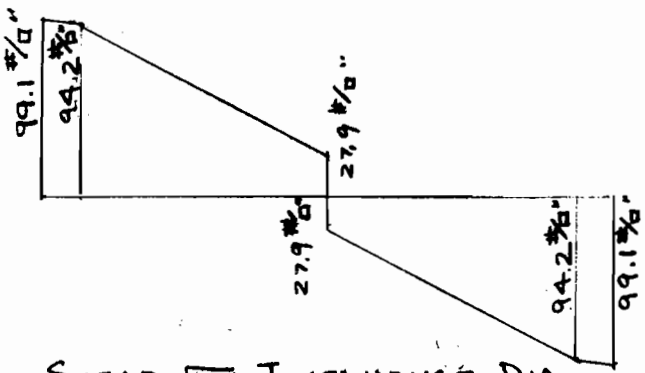
V_A due to shear

$V_A = \dfrac{29{,}500}{24 \times 28}$

$V_A = 43.9\ \#/\square''$



SHEAR ↓ INFLUENCE DIA.
DUE TO MOVING VERT LOADS
AND DEAD LOAD



SHEAR ↓ INFLUENCE DIA.
DUE TO TORSION OF MOVING
VERT. LOADS + DEAD LOAD

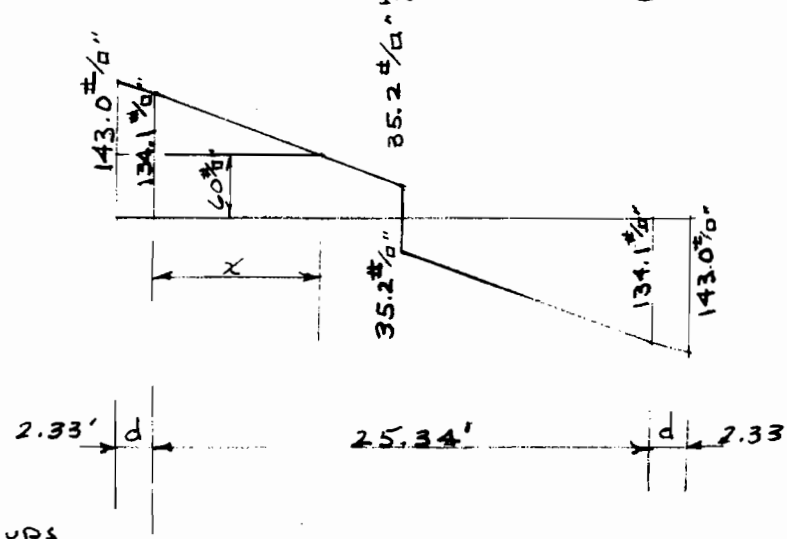
Figure 4-31

Computed by [illegible]
Ckd by A.R.

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE #10

DESIGN OF VERTICAL STIRRUPS

COMPOSITE SHEAR-TORSION INFLUENCE DIA.



$$\frac{98.9}{12.67} = \frac{74.1}{x}$$

$$x = \frac{74.1 \times 12.67}{98.9}$$

$$x = 9.49'$$

USE #4 stirrups

$$\frac{v'b}{B\,A_v f_v} = \frac{74.1 \times 24}{1 \times 8{,}000} = 0.222$$

$S_1 = 9.49'$ From ACI Handbook

$$\text{Max } s = \frac{A_v}{.0015\,b} = \frac{2 \times .20}{.0015 \times 24} = 11.1''$$

say 10"

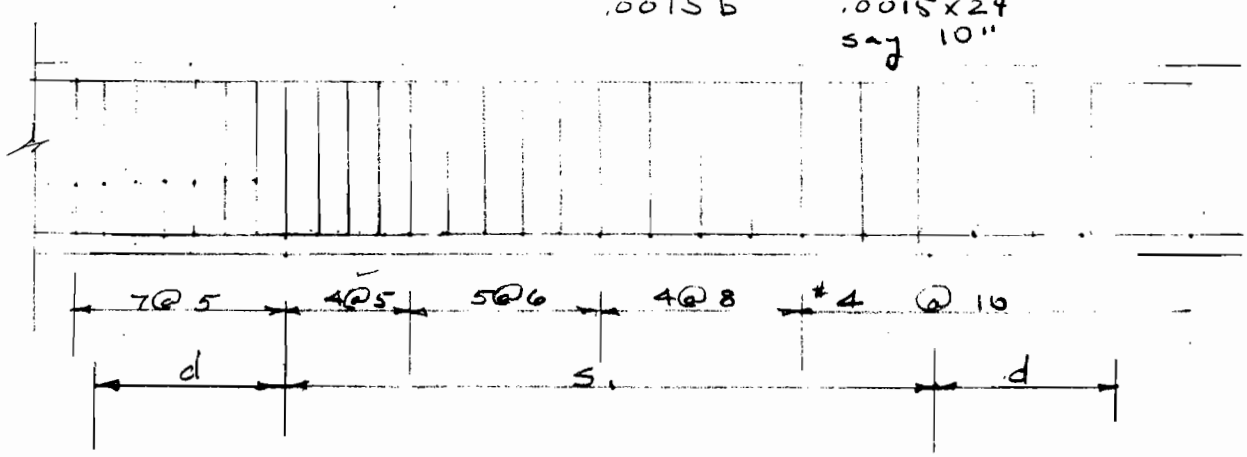
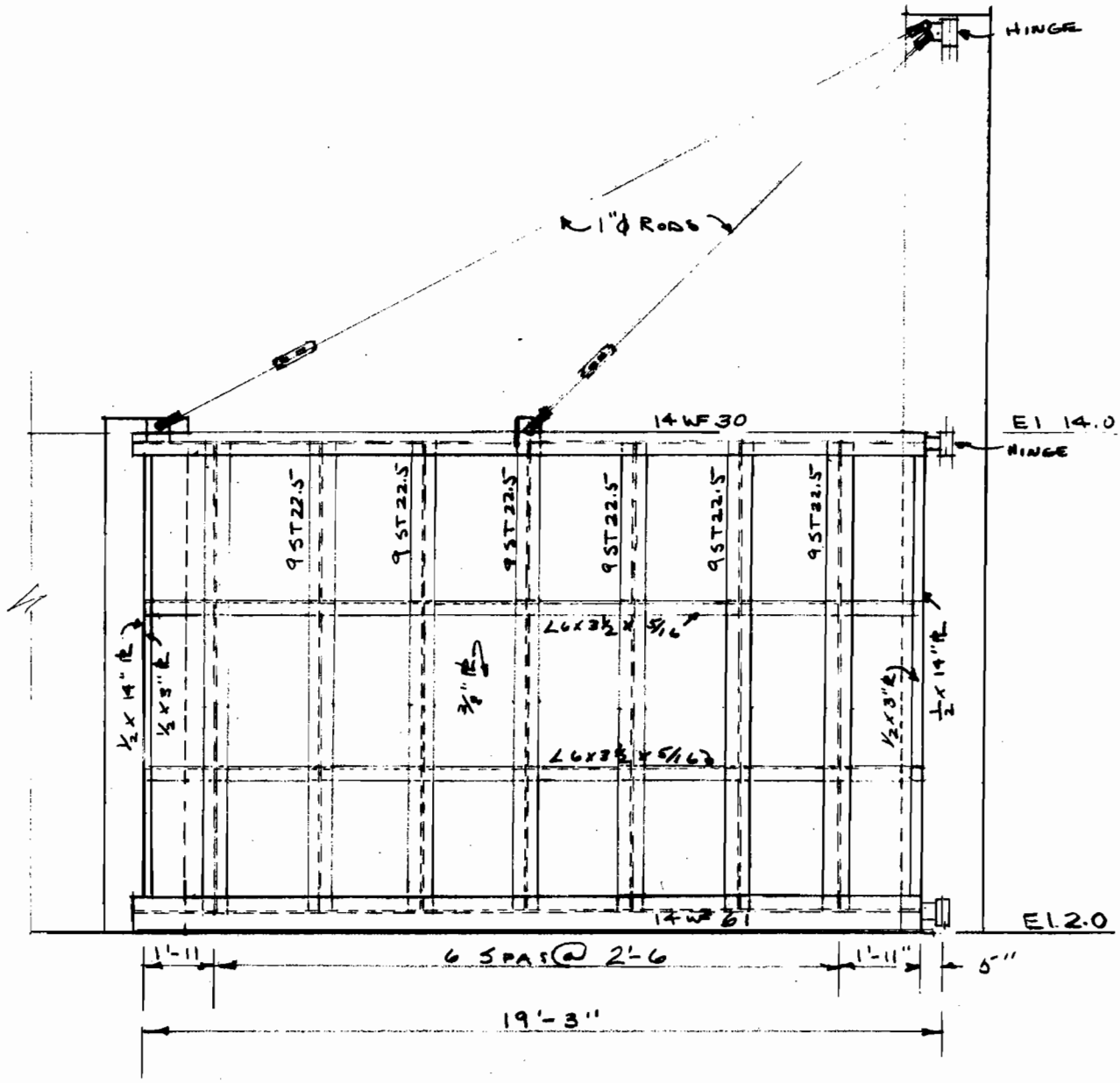


Figure 4-32

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE, FLA. AVE TO LOCK
GATE #11 SWING GATE

Sheet 1 of 6
Computed by WAM
Ckd by A.K



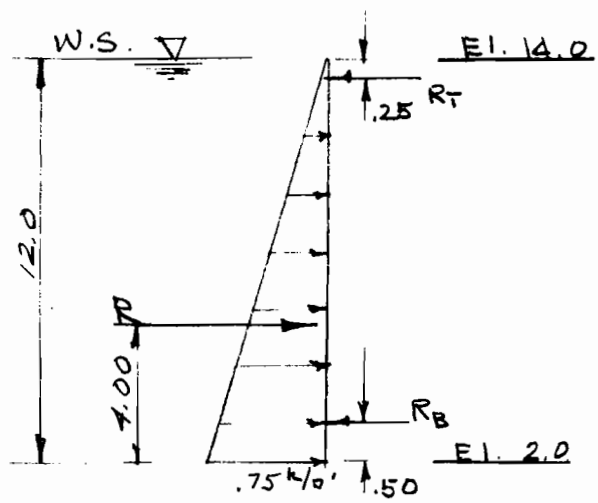
FLOODSIDE ELEVATION

Scale ¼" = 1'-0

Figure 4-33

LAKE PONTCHARTRAIN LA. & VIC
IHNC WEST SIDE, FLA AVE TO LOCK
GATE # 11 SWING GATE

Sheet 2 of 6
Computed by D.G.M
1/30/67
Ckd by A.R.



$R = \frac{1}{2} \times 0.75 \times 12.0 = \underline{4.5\ k/'}$

$R_T = \frac{3.5}{11.25} \times 4.5 = \underline{1.4\ k/'}$

$R_B = \frac{7.75}{11.25} \times 4.5 = \underline{3.1\ k/'}$

UPPER GIRDER

SPAN = 19'-0" ±

$M = \frac{1}{8} \times 1.4 \times 19^2 = 63.2'^k$

$S = 63.2 \times \frac{12}{20} = 37.9\ in^3$

USE 14WF30 $I = 289.6$

BENDING
ALLOW σ = 20,000 psi

$\Delta = \frac{.013 \times 26.6 \times 228^3}{30 \times 10^3 \times 289.6}$

$\Delta = .47$ in

LOWER GIRDER

$M = \frac{1}{8} \times 3.1 \times 19^2 = 140'^k$

$S = 140 \times \frac{12}{20} = 84\ in^3$

USE 14WF61 $I = 641.5$

$\Delta = \frac{.013 \times 58.9 \times 228^3}{30 \times 10^3 \times 641.5}$

$\Delta = .47$ in.

SKINPLATE 3/8" ℄

MAX PRESSURE = .75 k/□'

$I = \frac{1}{12} \times 12 \times 0.375^3$

$I = 0.053\ in^4$

$S = \frac{0.053}{0.1875} = 0.283\ in^2$

BENDING
ALLOW σ = 20,000 psi

Figure 4-34

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE, FLA. AVE TO LOCK
GATE #11 SWING GATE

Sheet 3 of 6
Computed by [illegible]
1/30/67
Ckd by A.K.

SKINPLATE

$M_{max} = 20{,}000 \times 0.283 = 5{,}660$ in-lbs $= 471.7$ ft lbs

$M = \frac{1}{12} wL^2 \qquad L^2 = \frac{12 \times 471.7}{750} = 7.55$

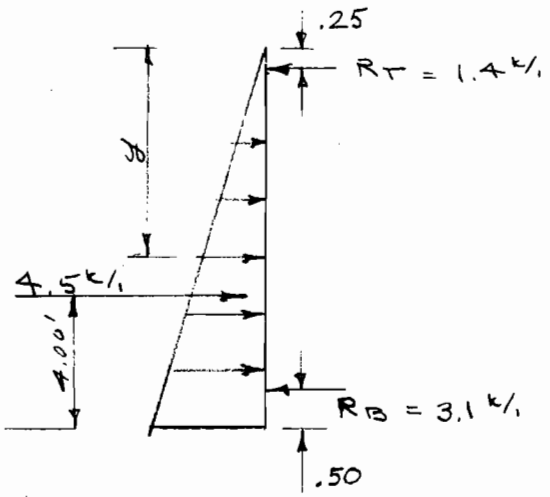
$L = 2.75'$ USE 2'-6" for vertical spacing

$M = \frac{1}{10} wL^2$ for ext. span

$L^2 = \frac{10}{12} \times 7.55 = 6.29$

$L = 2.51$ 2'-6" Spacing o.k.

VERTICAL STIFFENERS



PT. OF ZERO SHEAR

$1.4^k = \frac{.0625}{2} y^2$

$y^2 = 44.8$

$y = 6.69'$

$M_{max} = 1.4 \times 6.44 - 1.4 \times \frac{6.69}{3}$

$= 1.4 \times 4.21$

$M_{max} = 5.89'^k$

FOR 2'-6" SPACING

ALLOW $\sigma = 18{,}000$ psi
For Unsymmetrical member

$M = 2.5 \times 5.89$

$M = 14.73$

$S_{req'd} = 14.73 \times 12/18 = 9.82 \text{ in}^3$

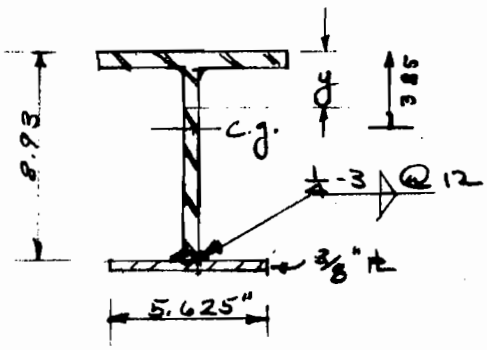
Figure 4-35

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE, FLA. AVE TO LOCK
GATE #11 SWING GATE

Sheet 4 of 6
Computed by D.G.M.
1/30/67
Ckd by A.R.

STIFFENERS:

Try ST 9-22.5



Tee: Area = 6.62 □"
y = 2.17"
d = 8.93"
I = 48.70

Area of PL = 5.625 x .375
= 2.11 □"

cg: 6.62 x 2.17 = 14.37
2.11 x 9.12 = 19.24
8.73 x 3.85 = 33.61

I = 48.70
+ 6.62 x 1.68² = 18.68
+ 2.11 x 5.27² = 58.60
Total = 125.98

S = 125.98 / 5.46

S = 23.07 in³ O.K. Req'd S = 9.82 in³

HOR. SHEAR

Max V = 3.1 x 2.5
V = 7.75^k

hor. shear = VQ/I = 7.75 x 2.11 x 5.27 / 125.98 = .684^k/"

Use 1/4" - 3 @ 12

Figure 4-36

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE, FLA. AVE TO LOCK
GATE # 11 SWING GATE

Sheet 5 of 6
Computed by D.M.
1/30/69
Ckd by A.K.

WEIGHT OF GATE

3/8" Skinplate	19 x 12.0 x 15.3#	= 3,488	x 0.1875	= 654.
14 WF 30	19 x 30	= 570	x 7.375	= 4,204
14 WF 61	19 x 61	= 1159	x 7.375	= 8,548
ST 9-22.5	7 x 11.25 x 22.5	= 1,772	x 5.28	= 9,356.
1/2" x 3" PL	2 x 0.25' x 11.25' x 20.4#	= 115	x 14.125	= 1,624.
1/2 x 14" PL	2 x 1.17' x 12.0' x 20.4#	= 573	x 7.375	= 4,226.
∠ 6 x 3 1/2 x 5/16	18 x 2 x 9.8#	= 353	x 4.365	= 1,541.
		8,030#	x 3.84	30,307

SUPPORT RODS & HINGES

$V_1 = \frac{7}{16} WL - \frac{1}{16} WL = \frac{3}{8} WL$, $W = \frac{8,030}{19}$

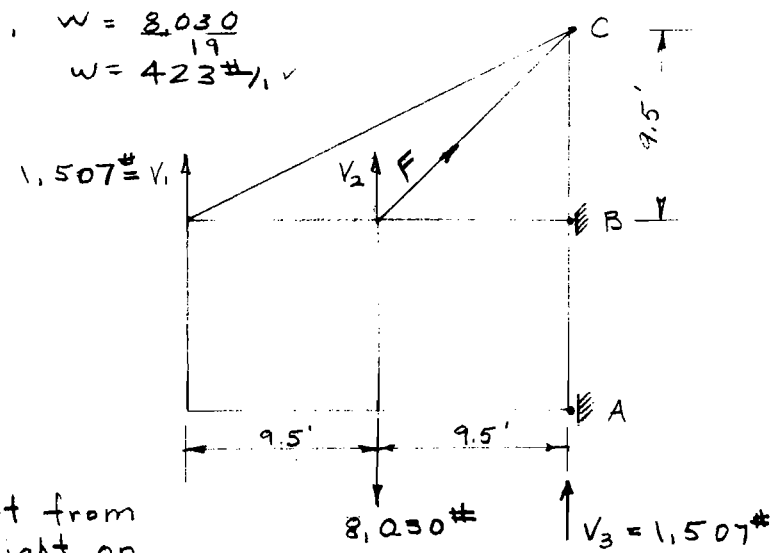
$V_1 = \frac{3}{8} \times 423 \times 9.5$ $\qquad W = 423\#/'$

$V_1 = 1,507\#$

$V_1 = V_3$

$V_2 = 8,030 - 2 \times 1,507$

$V_2 = 5,016\#$



support rods:

Assume no support from outer rod, all weight on center rod:

$F = \sqrt{2} \times 8,030\#$

$F = 11,354\#$

As Req'd $= \frac{11,354\#}{18,000} = .631\square''$ - USE 1" ϕ RODS

ALLOW $\bar{T}$ = 18,000 psi Tension

Figure 4-37

Computed by D.G.M.
1/30/67
Ckd by A.R.

LAKE PONTCHARTRAIN LA. & VIC.
IHNC WEST SIDE FLA. AVE TO LOCK
GATE # 11 SWING GATE

SUPPORT ROD PAD EYES

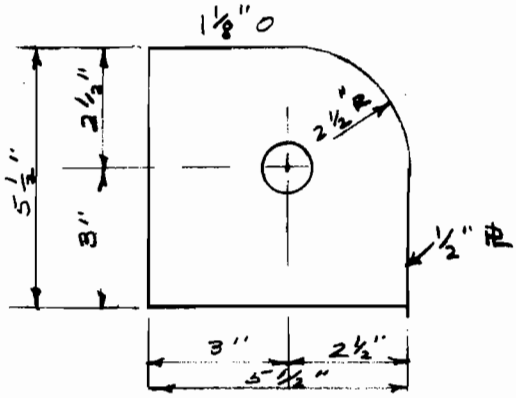
F = 11,354# - 1" ϕ pin

Bearing Area Req'd = $\frac{11,354}{27,000}$ = .421 □"

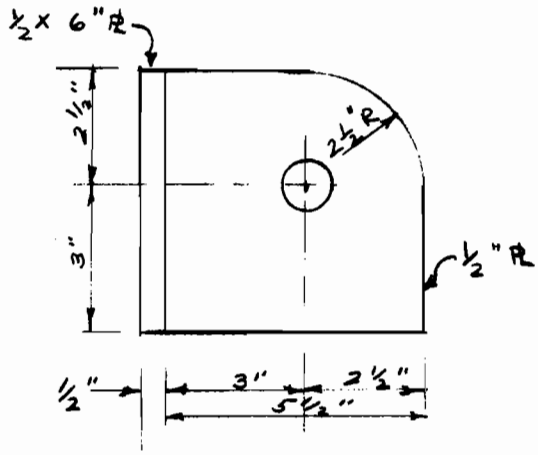
Thickness Req'd = $\frac{.421}{1.0}$ = .421" Use ½" ℞

Tension Area Req'd = $\frac{11,354}{13,500}$ = .841 □"

Width Req'd = $\frac{.841}{.5}$ = 1.69" Use 2"



OUTER PAD EYE



CENTER PAD EYE

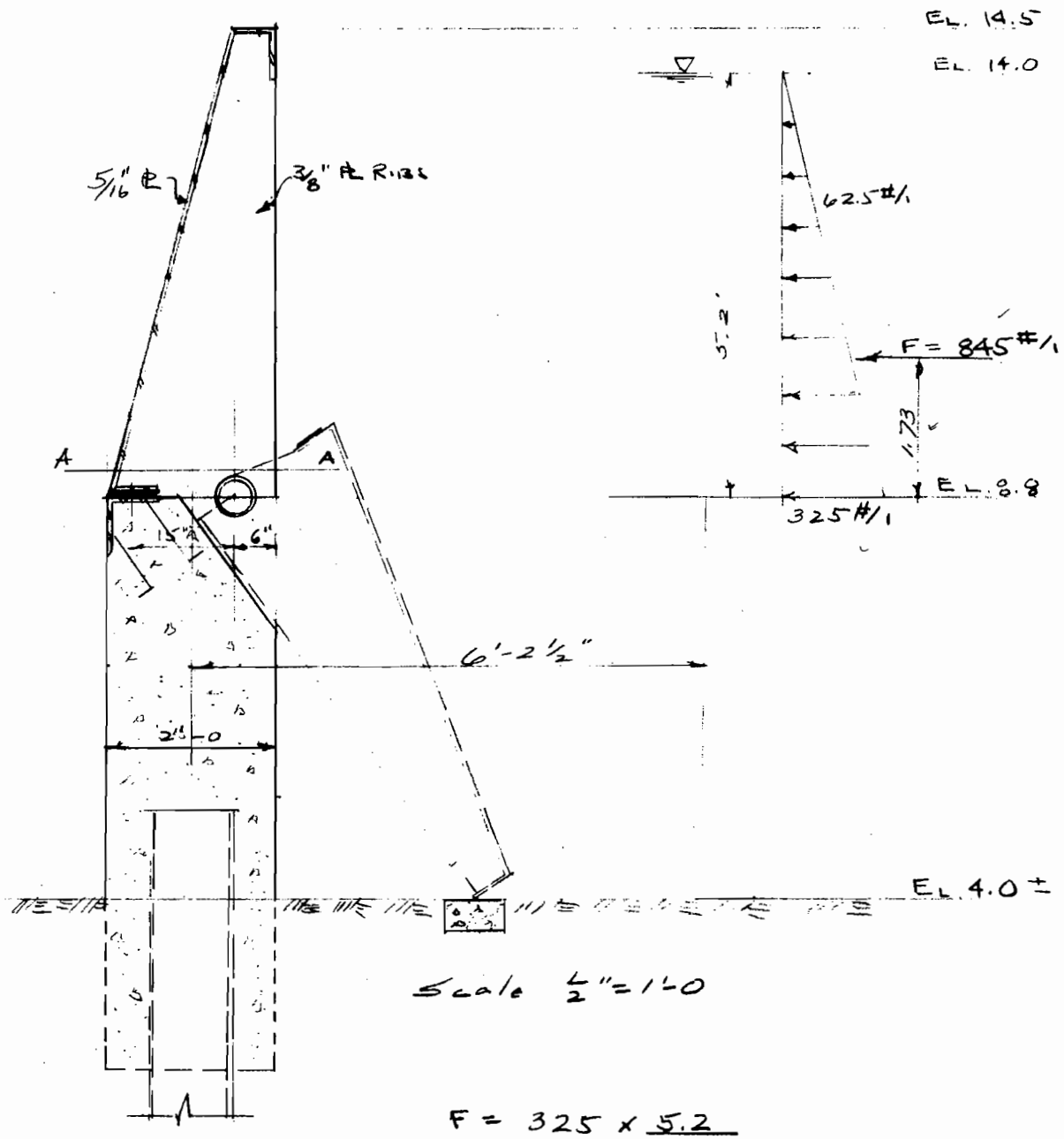
Figure 4-38

LAKE PONTCHARTRAIN, LA. & VICINITY
IHNC - WEST SIDE - FLA AVE TO LOCK

Sheet 1 of 3
D.M
Chkd by A.R.

HINGED FLOOD FLAP DESIGN

GATE #9 J&L WAREHOUSE



Scale 1/2" = 1'-0

$$F = 325 \times \frac{5.2}{2}$$

$$F = 845 \#/1$$

Figure 4-39

LAKE PONTCHARTRAIN LA & VIC
IHNC - WEST SIDE - FLA AVE TO LOCK

DESIGN OF SKINPLATE: Allow T = 20,000 #/□"

Max load = 325 #/□'

Use 5/16" skinplate $S = \frac{1}{6} \times 12 \times \left(\frac{5}{16}\right)^2 = 0.195 \text{ in}^3$

$M_{max} = \frac{325(L)^2}{12}$ $T = \frac{M}{S}$

Substituting $\frac{325(L)^2 \times 12}{12 \times 0.195} = 20{,}000$ psi

$L^2 = 12.0$ FT^2

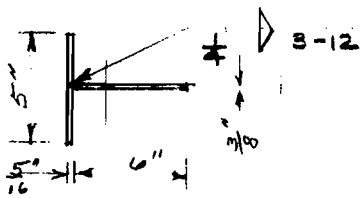
$L = 3.47$ FT

∴ Use 24 spaces @ 3'-2½" = 77'-0

Section A-A Allow T = 18,000 #/□"

$M_{max} = (845 \times 1.73)(3.21)$

$M_{max} = 4{,}693'^{\#}$



c.g. 1.55□" × 0.155 = 0.24
2.25□" × 3.31 = 7.42
3.80 × 2.02 = 7.66

$I = 1.55\square' \times 1.86^2 = 5.37$
$\frac{1}{12} \times 6^3 \times 3/8 = 6.75$
$2.25 \times 1.29^2 = 3.75$
15.87 in^4

$S = \frac{15.87}{4.29}$

$S = 3.7 \text{ in.}^3$

$F_b = \frac{4{,}693'^{\#} \times 12}{3.7}$

$F_b = 15{,}219$ psi. ok

Max Shear on Section = 3.21 × 845# = 2712#

Hor. Sh. $= \frac{2712 \times 1.55 \times 1.86}{15.87}$

= 493 #/"

Use ¼" - 3" @ 12

Figure 4-40

LAKE PONTCHARTRAIN, LA. & VIC. Sheet 3 of 3 WM

IHNC – WEST SIDE – FLA. AVE. TO LOCK Ckd by A.R.

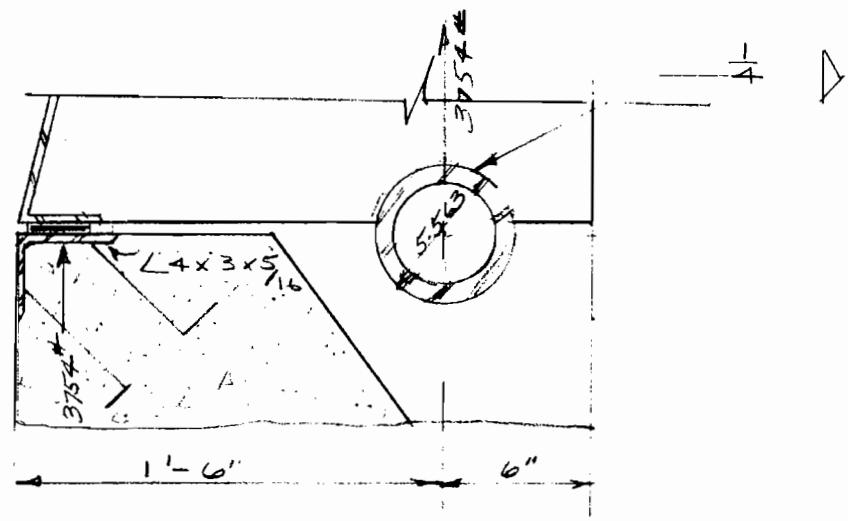
Load on Hinge:

$\leftarrow 845 \times 3.21 = 2712^{\#}$ ✓

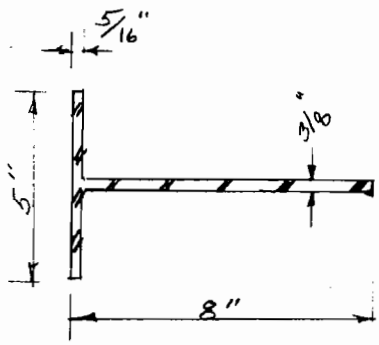
$\frac{845 \times 1.73 \times 3.21}{1.25} = 3754^{\#}$ ✓

Result = $4,640^{\#}$

Weld size $\frac{4,640^{\#}}{8''} = 580^{\#}/''$



Column: (A.C.I. – Sect. 1.9.1)



$d = \frac{4,000 \times 3/8}{\sqrt{F_y}} = \frac{4,000 \times 3/8}{\sqrt{36,000}}$

$d = 8''$

$I_{2-2} = \frac{1}{12} \times \frac{5}{16} \times 5^3 = 3.25 \text{ in}^4$

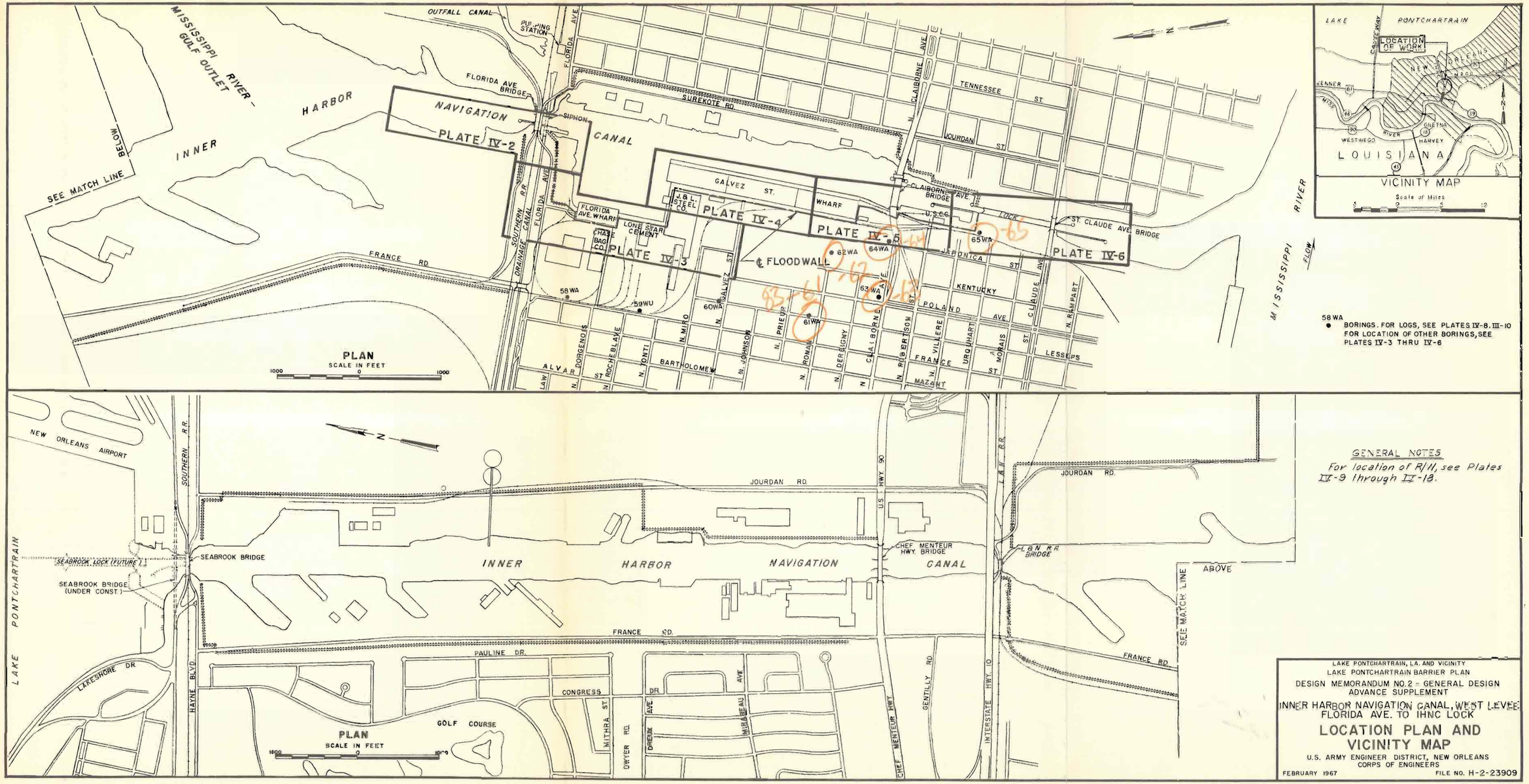
$A = 4.43$ ✓

$r = \sqrt{\frac{3.25}{4.43}} = 0.85$ ✓

$\frac{L}{r} = \frac{125}{.85} = 147$ Allow $\sigma = 5.73$ ksi

Tot. Col allow ld. = 25.4^k ok.

Figure 4-41



GENERAL NOTES

For location of R/11, see Plates IV-9 through IV-18.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

LOCATION PLAN AND VICINITY MAP

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-1

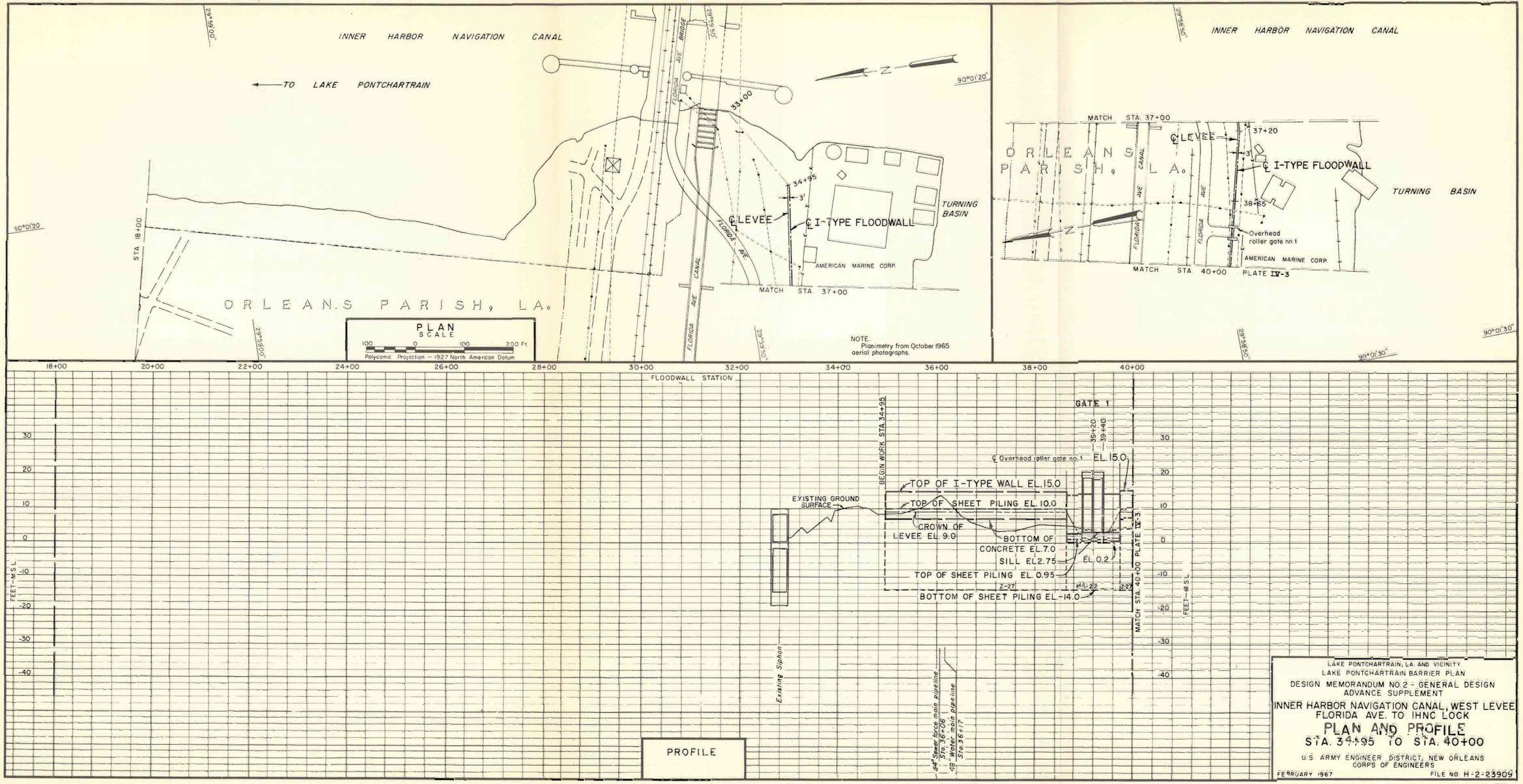
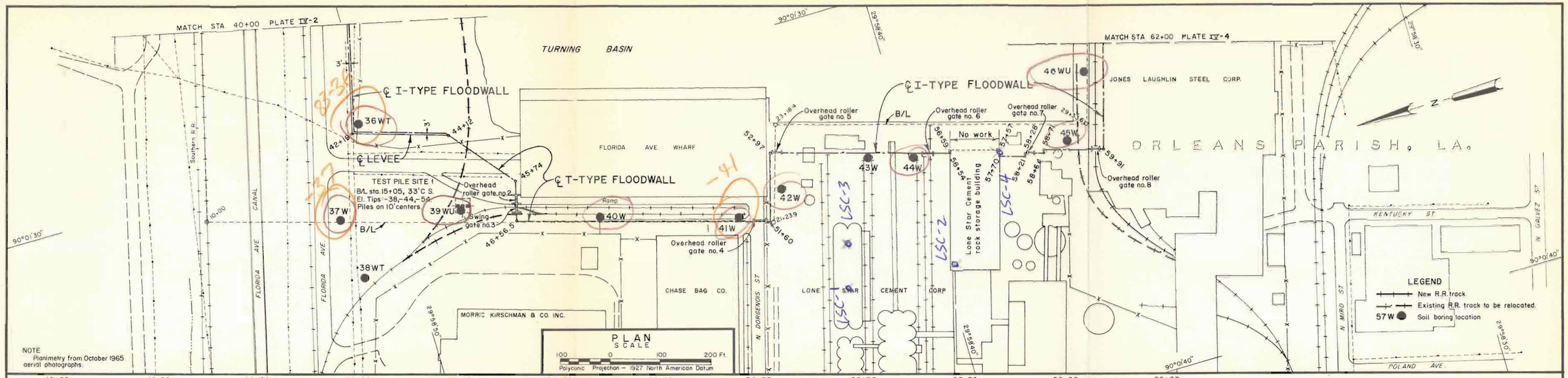
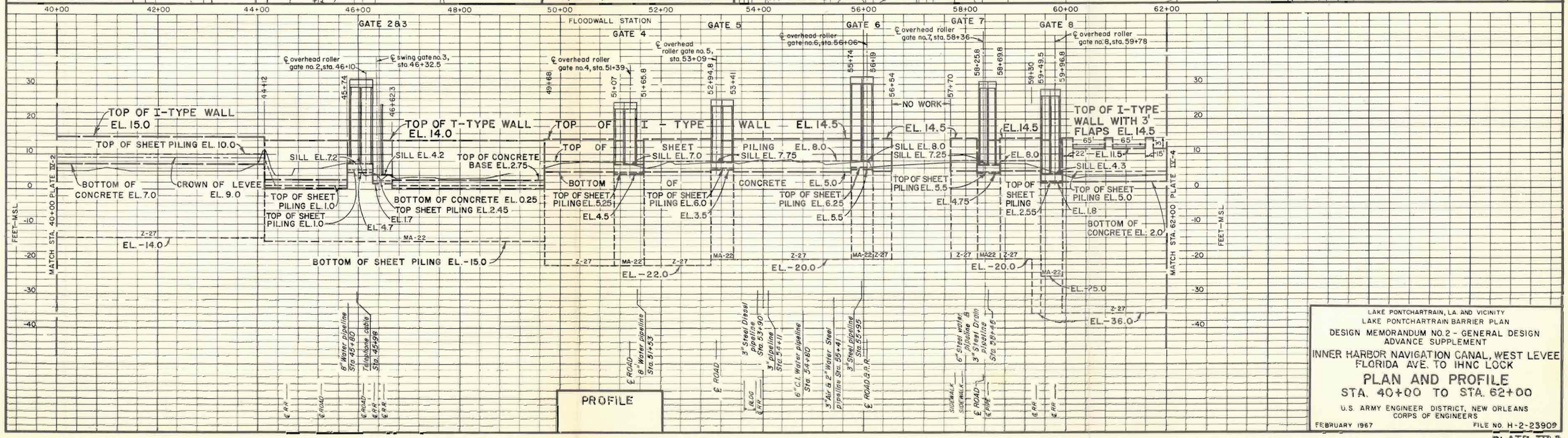

INNER HARBOR NAVIGATION CANAL
TO LAKE PONTCHARTRAIN
FLORIDA AVE BRIDGE
FLORIDA AVE CANAL
33+00
34+95
℄ LEVEE
℄ I-TYPE FLOODWALL
TURNING BASIN
AMERICAN MARINE CORP.
MATCH STA. 37+00
STA 18+00
ORLEANS PARISH, LA.
PLAN
SCALE
100 0 100 200 Ft
Polyconic Projection – 1927 North American Datum
NOTE:
Planimetry from October 1965 aerial photographs.
MATCH STA. 37+00
37+20
38+65
Overhead roller gate no. 1
MATCH STA. 40+00
PLATE IV-3
FLOODWALL STATION
18+00
20+00
22+00
24+00
26+00
28+00
30+00
32+00
34+00
36+00
38+00
40+00
FEET-M.S.L.
GATE 1
BEGIN WORK STA. 34+95
℄ Overhead roller gate no. 1
EL. 15.0
TOP OF I-TYPE WALL EL. 15.0
TOP OF SHEET PILING EL. 10.0
EXISTING GROUND SURFACE
CROWN OF LEVEE EL. 9.0
BOTTOM OF CONCRETE EL. 7.0
SILL EL. 2.75
EL. 0.2
TOP OF SHEET PILING EL. 0.95
BOTTOM OF SHEET PILING EL. -14.0
MATCH STA. 40+00 PLATE IV-3
Existing Siphon
54" Sewer force main pipeline Sta. 36+06
48" Water main pipeline Sta. 36+17
PROFILE
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
PLAN AND PROFILE
STA. 34+95 TO STA. 40+00
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

PLATE IV-2





LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

PLAN AND PROFILE
STA. 40+00 TO STA. 62+00

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-3

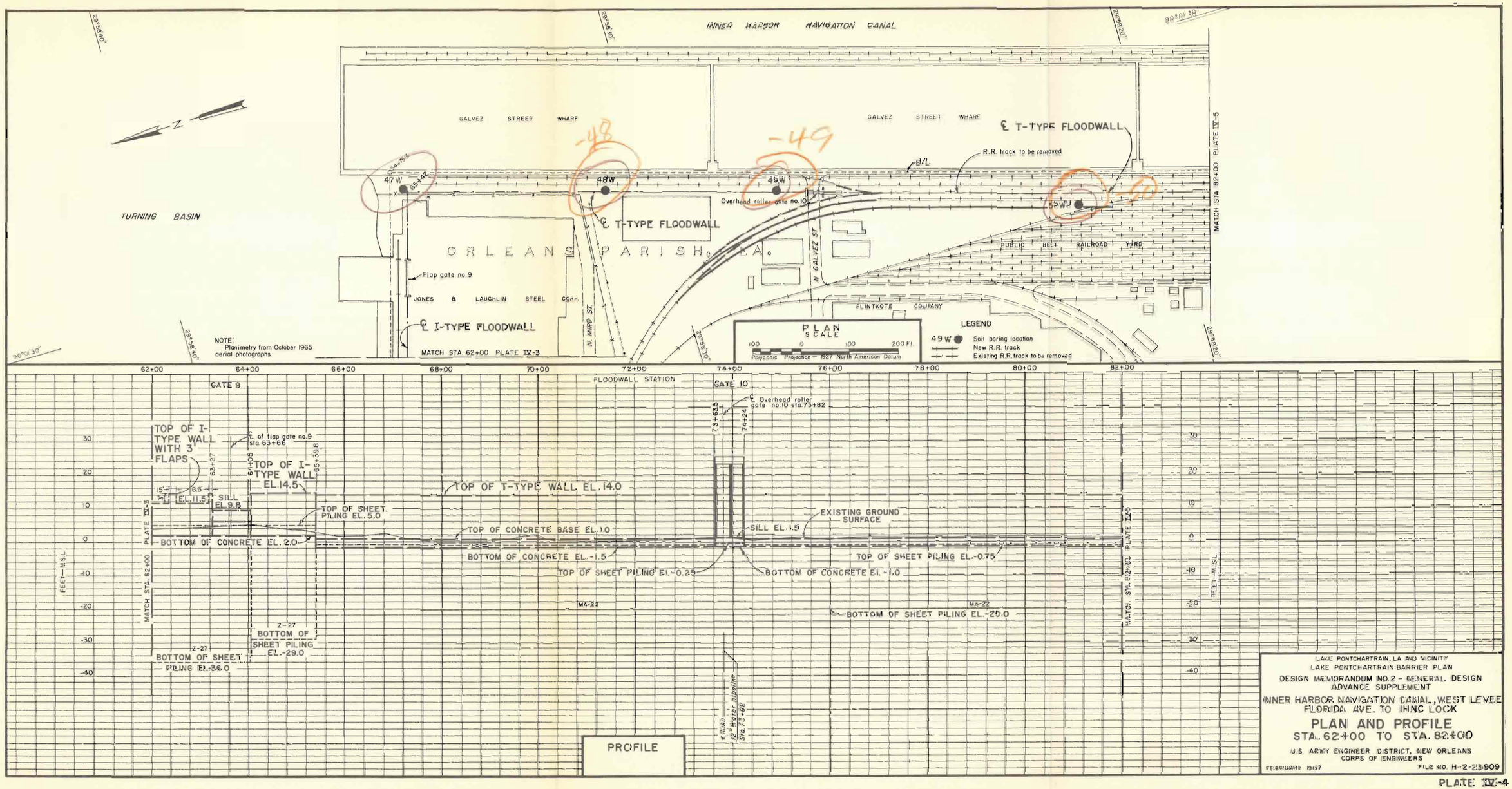
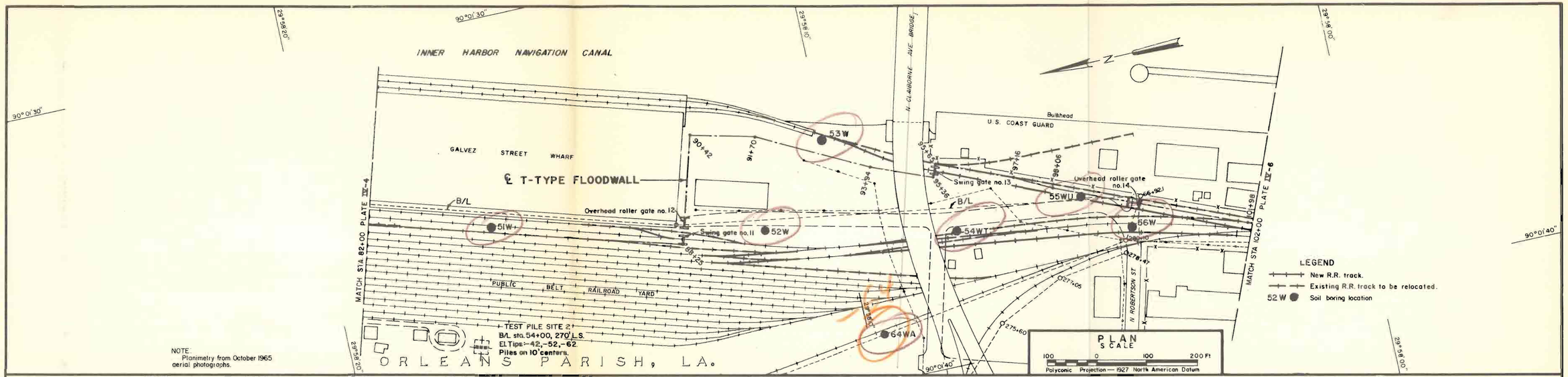
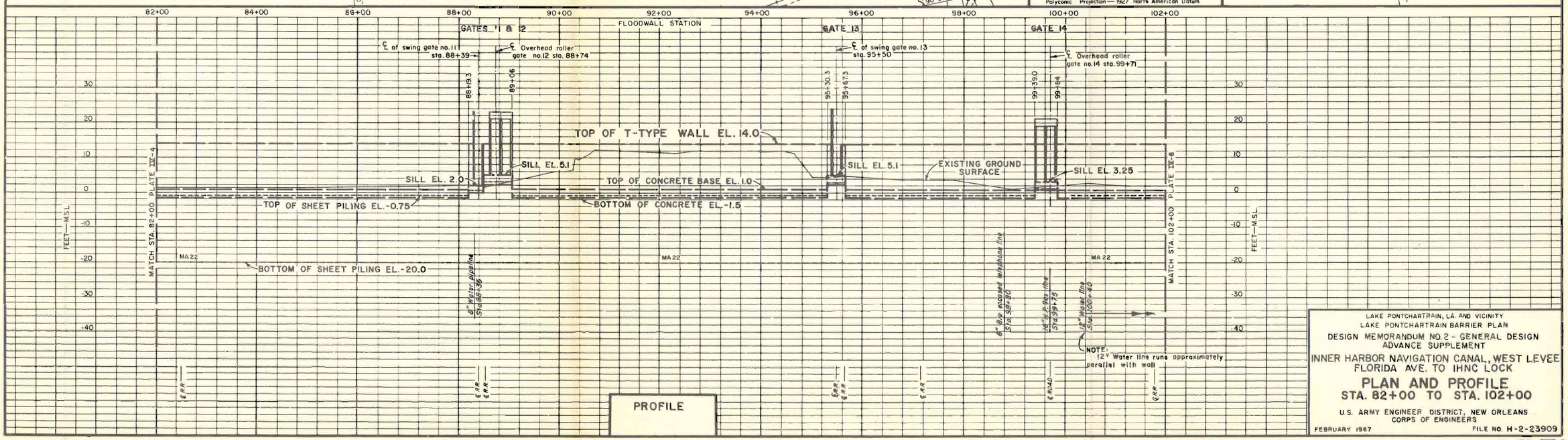
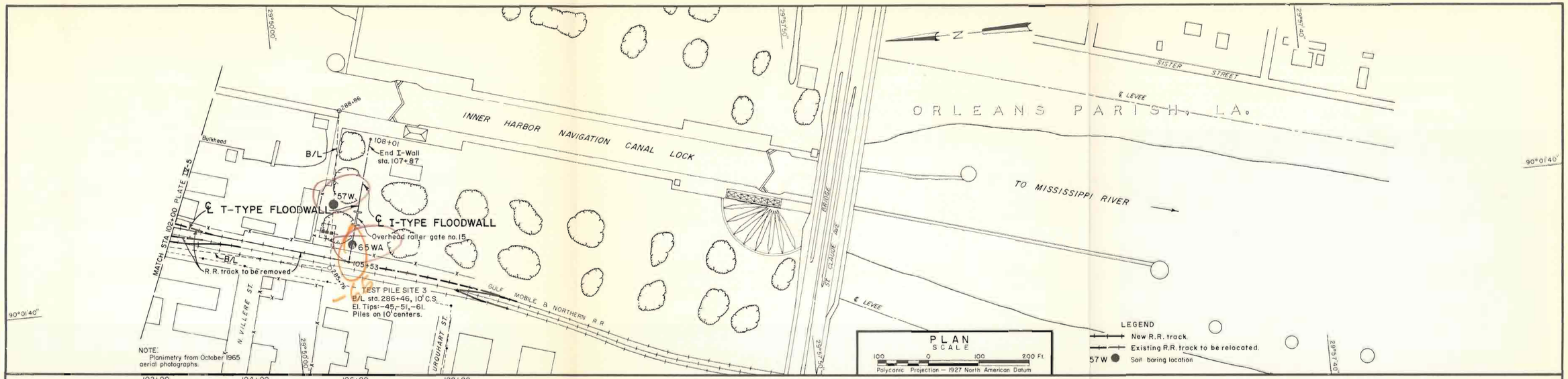
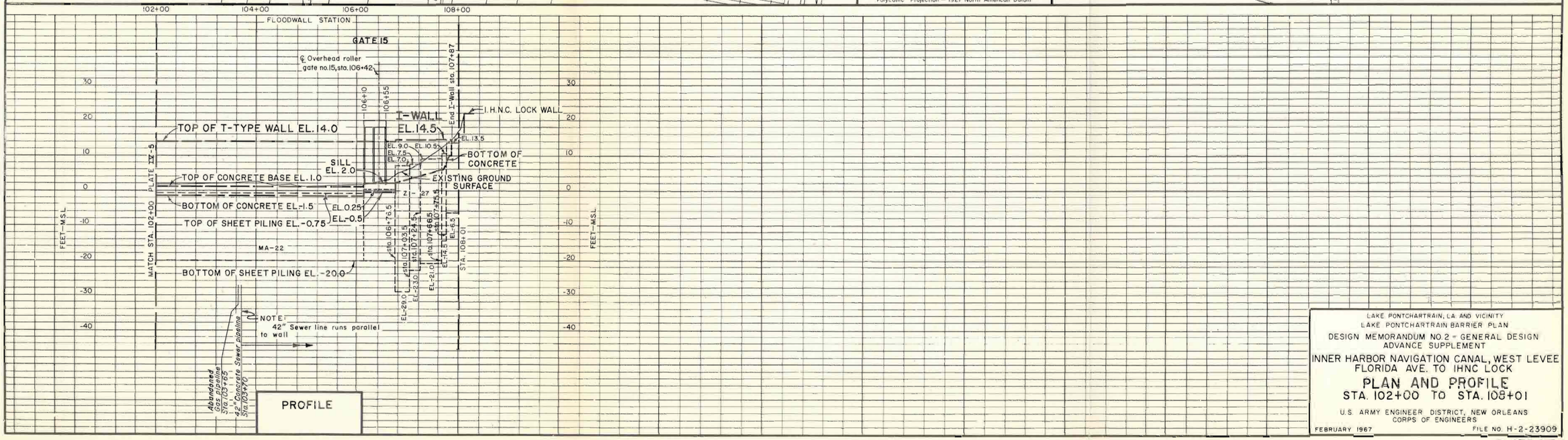

INNER HARBOR NAVIGATION CANAL
GALVEZ STREET WHARF
GALVEZ STREET WHARF
℄ T-TYPE FLOODWALL
R.R. track to be removed
B/L
TURNING BASIN
47W
48W
49W
55W
Overhead roller gate no.10
℄ T-TYPE FLOODWALL
ORLEANS PARISH, LA.
PUBLIC BELT RAILROAD YARD
N. GALVEZ ST.
N. MIRO ST.
Flap gate no.9
JONES & LAUGHLIN STEEL CORP.
FLINTKOTE COMPANY
℄ I-TYPE FLOODWALL
MATCH STA. 62+00 PLATE IV-3
MATCH STA. 82+00 PLATE IV-5
NOTE:
Planimetry from October 1965 aerial photographs.
PLAN
SCALE
100 0 100 200 Ft.
Polyconic Projection — 1927 North American Datum
LEGEND
49W Soil boring location
New R.R. track
Existing R.R. track to be removed
FLOODWALL STATION
GATE 9
GATE 10
℄ Overhead roller gate no.10 sta.73+82
℄ of flap gate no.9 sta.63+66
TOP OF I-TYPE WALL WITH 3' FLAPS
TOP OF I-TYPE WALL EL.14.5
TOP OF T-TYPE WALL EL.14.0
EL.11.5
SILL EL.9.8
TOP OF SHEET PILING EL.5.0
TOP OF CONCRETE BASE EL.1.0
SILL EL.1.5
EXISTING GROUND SURFACE
BOTTOM OF CONCRETE EL.2.0
BOTTOM OF CONCRETE EL.-1.5
TOP OF SHEET PILING EL.-0.25
BOTTOM OF CONCRETE EL.-1.0
TOP OF SHEET PILING EL.-0.75
BOTTOM OF SHEET PILING EL.-20.0
BOTTOM OF SHEET PILING EL.-29.0
BOTTOM OF SHEET PILING EL.-36.0
MA-22
Z-27
FEET—M.S.L.
PROFILE
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
PLAN AND PROFILE
STA. 62+00 TO STA. 82+00
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

PLATE IV-4


INNER HARBOR NAVIGATION CANAL
GALVEZ STREET WHARF
℄ T-TYPE FLOODWALL
B/L
Overhead roller gate no.12
Swing gate no.11
51W
52W
53W
54WT
55WU
56W
64WA
Swing gate no.13
Overhead roller gate no.14
U.S. COAST GUARD
Bulkhead
N. CLAIBORNE AVE. BRIDGE
N. ROBERTSON ST.
PUBLIC BELT RAILROAD YARD
TEST PILE SITE 2
B/L sta. 54+00, 270' L.S.
El Tips:-42,-52,-62.
Piles on 10' centers.
ORLEANS PARISH, LA.
MATCH STA. 82+00 PLATE IV-4
MATCH STA. 102+00 PLATE IV-6
NOTE:
Planimetry from October 1965 aerial photographs.
LEGEND
New R.R. track.
Existing R.R. track to be relocated.
52W Soil boring location
PLAN
SCALE
100 0 100 200 Ft
Polyconic Projection — 1927 North American Datum


FLOODWALL STATION
GATES 11 & 12
GATE 13
GATE 14
℄ of swing gate no.11 sta. 88+39
℄ Overhead roller gate no.12 sta. 88+74
℄ of swing gate no.13 sta. 95+50
℄ Overhead roller gate no.14 sta. 99+71
TOP OF T-TYPE WALL EL. 14.0
SILL EL. 5.1
SILL EL. 2.0
TOP OF CONCRETE BASE EL. 1.0
EXISTING GROUND SURFACE
SILL EL. 3.25
TOP OF SHEET PILING EL.-0.75
BOTTOM OF CONCRETE EL.-1.5
BOTTOM OF SHEET PILING EL.-20.0
FEET—M.S.L.
MA 22
NOTE:
12" Water line runs approximately parallel with wall
PROFILE
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
PLAN AND PROFILE
STA. 82+00 TO STA. 102+00
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909





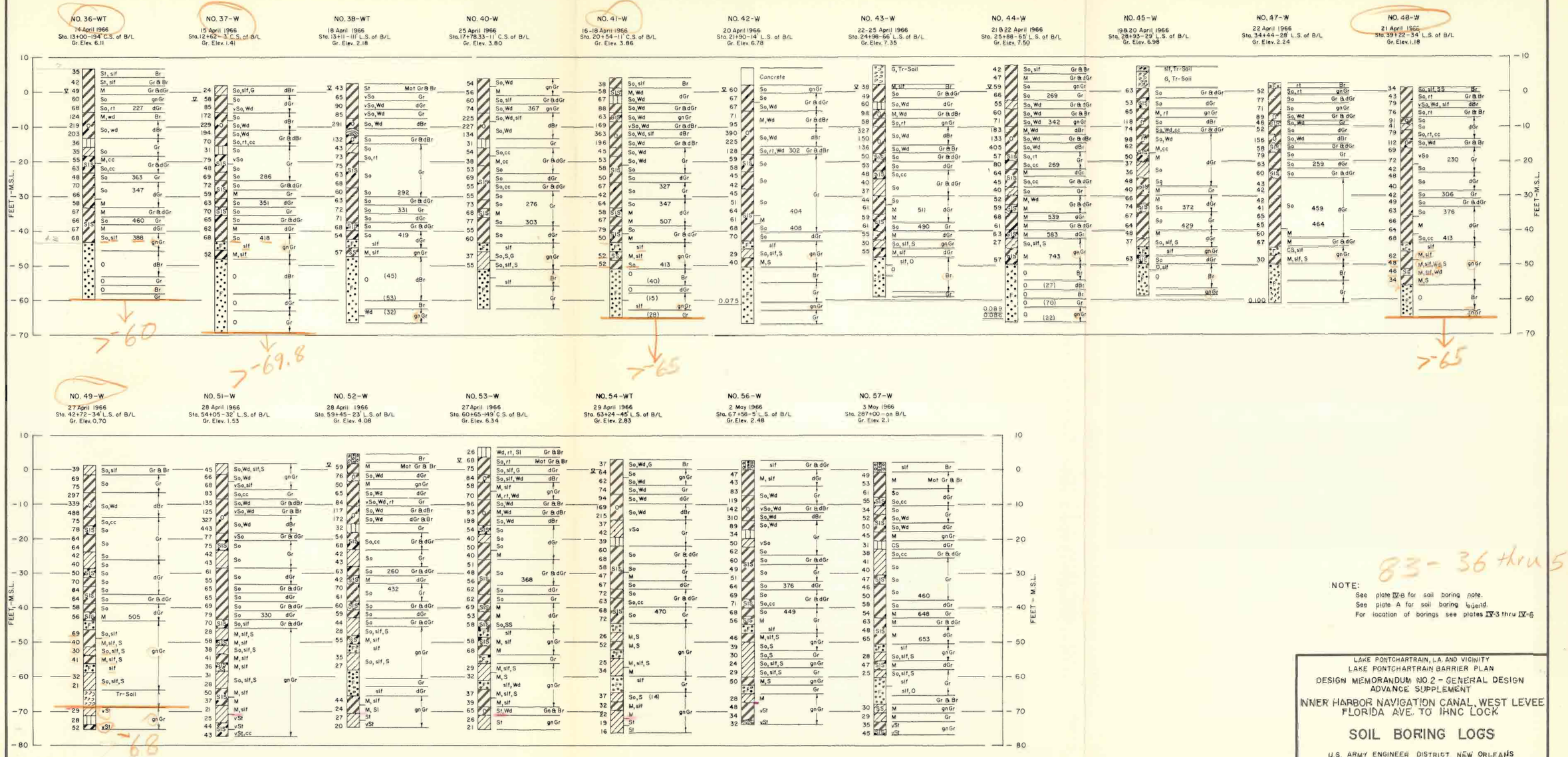
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

PLAN AND PROFILE
STA. 102+00 TO STA. 108+01

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

FEBRUARY 1967 FILE NO. H-2-23909



83-36 thru 57

NOTE:
See plate IV-8 for soil boring note.
See plate A for soil boring legend.
For location of borings see plates IV-3 thru IV-6

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

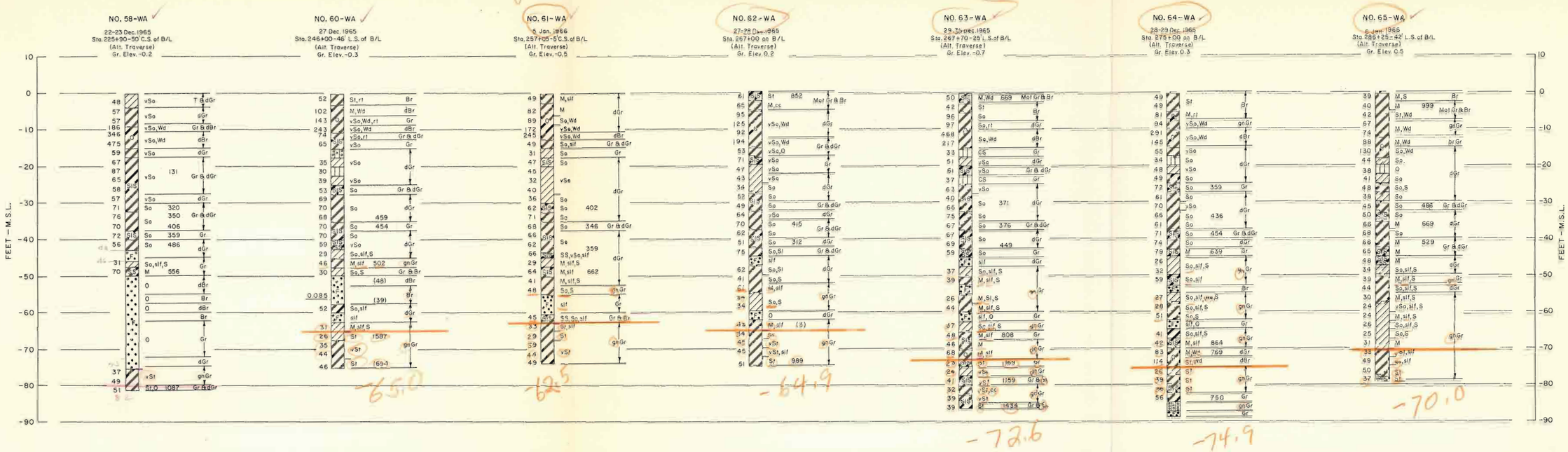
SOIL BORING LOGS

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-7

83-36,37,41,4849



PERTINENT NOTES

WATER CONTENTS shown are based on weight of oven dry soils.

STRATUM CHANGES are assumed to occur at an elevation halfway between elevations of samples shown, unless otherwise indicated.

Surface soil samples for borings no. 36 WT, 38 WT, 40 W, 43 W, 45 W, 47 W, 49 W, 51 W, 54 WT, 56 W, and 57 W were taken with a 4" Dia post hole auger.

Soil samples in sand stratum borings no. 41 W, 42 W 44 W and 60 WA, 63 WA were taken with a 1 3/8" I.D., 2" O.D. split spoon sampler using a 140 lb. hammer and a 30" drop

All other samples for borings no. 36 WT, 38 WT, 40 W, 45 W, 47 W, 48 W, 49 W, 51 W, 54 W, 56 W, 58 WA, and 60 WA, 65 WA were taken with a 1 7/8" core barrel sampler.

See plate A for soil boring legend.

For location of borings see plates IV-1, IV-5 & IV-6

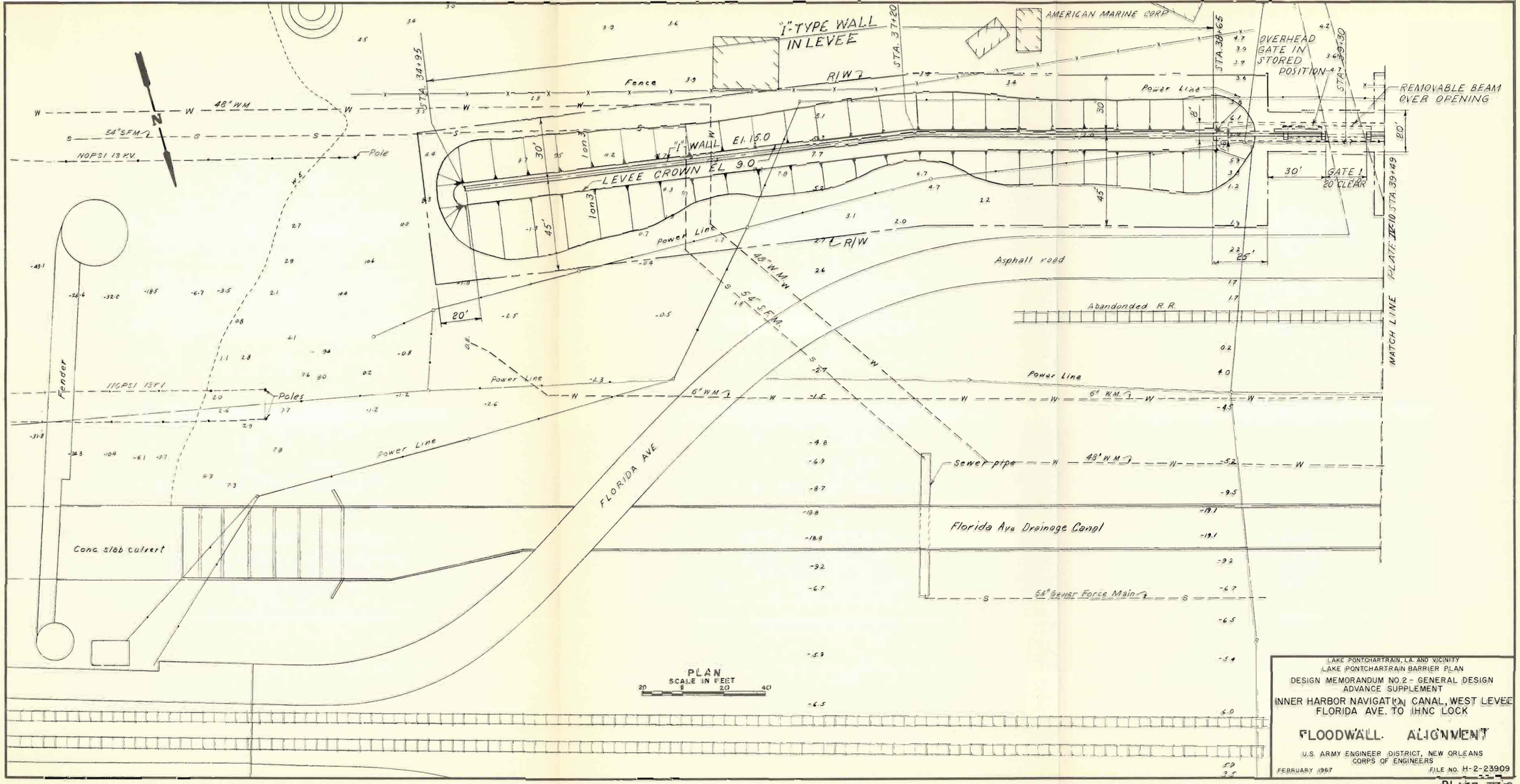
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

SOIL BORING LOGS

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-8



LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

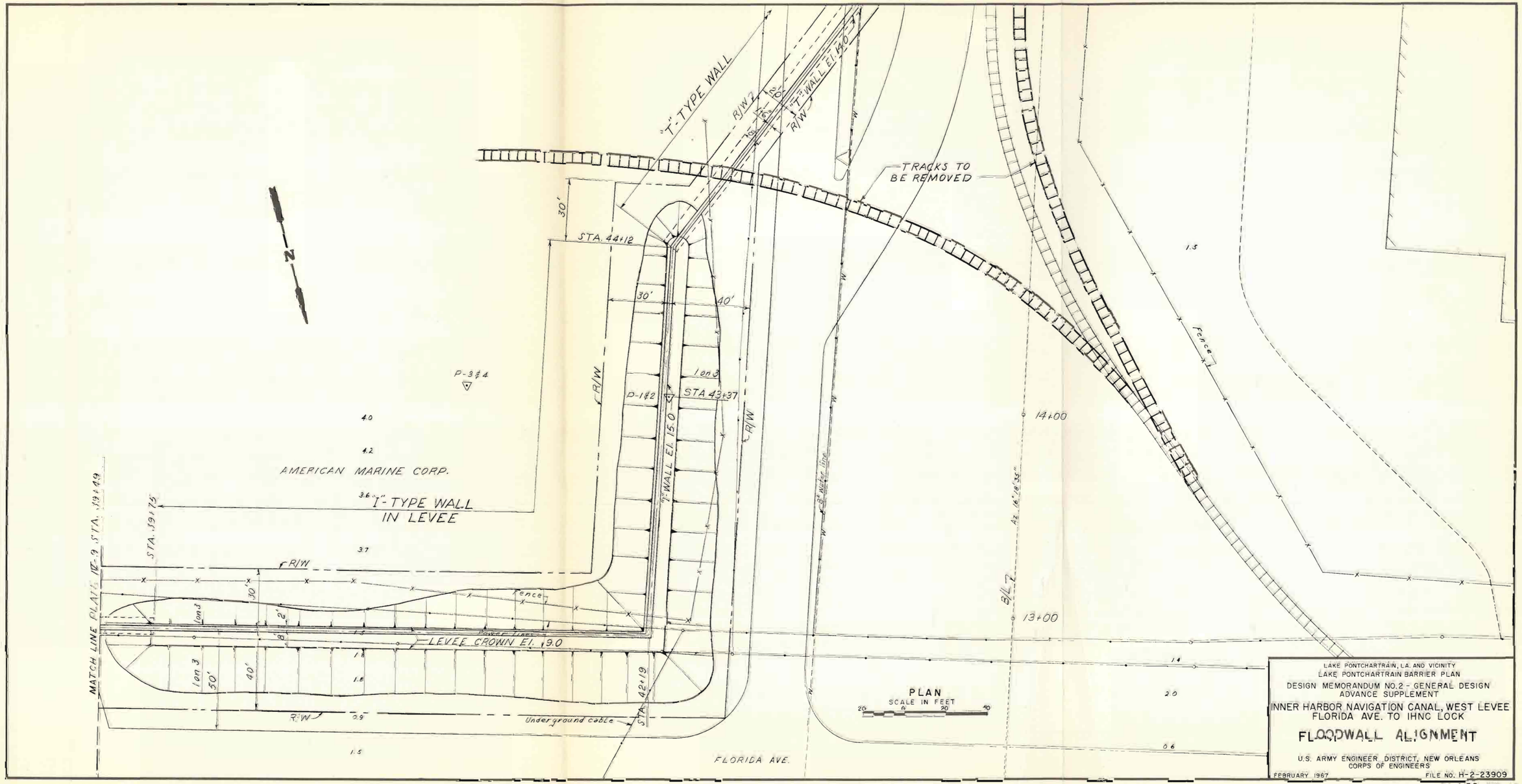
FLOODWALL ALIGNMENT

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-9

THIS AREA OVERLAPS PLATE IV-11



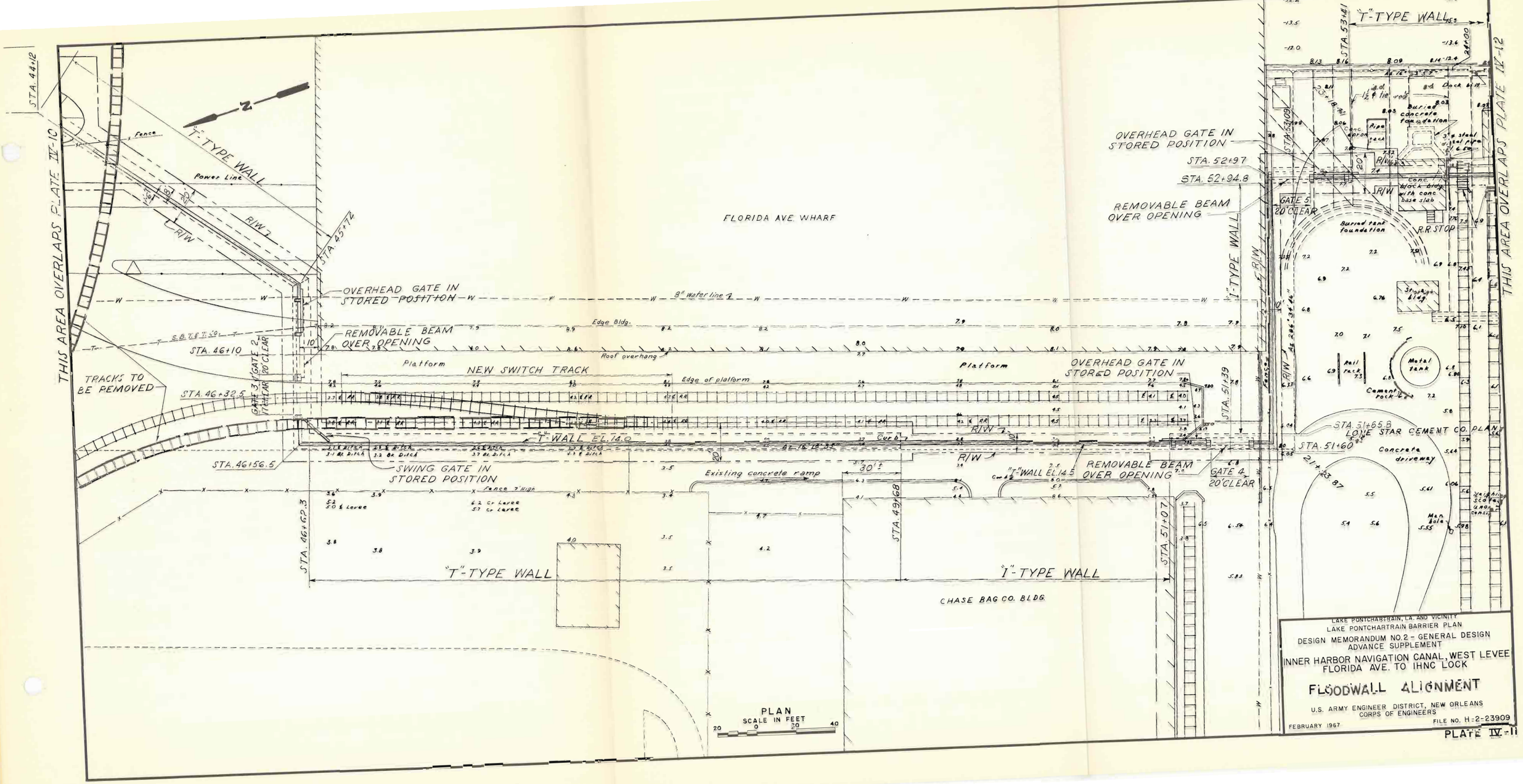
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

FLOODWALL ALIGNMENT

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

FEBRUARY 1967 FILE NO. H-2-23909



LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

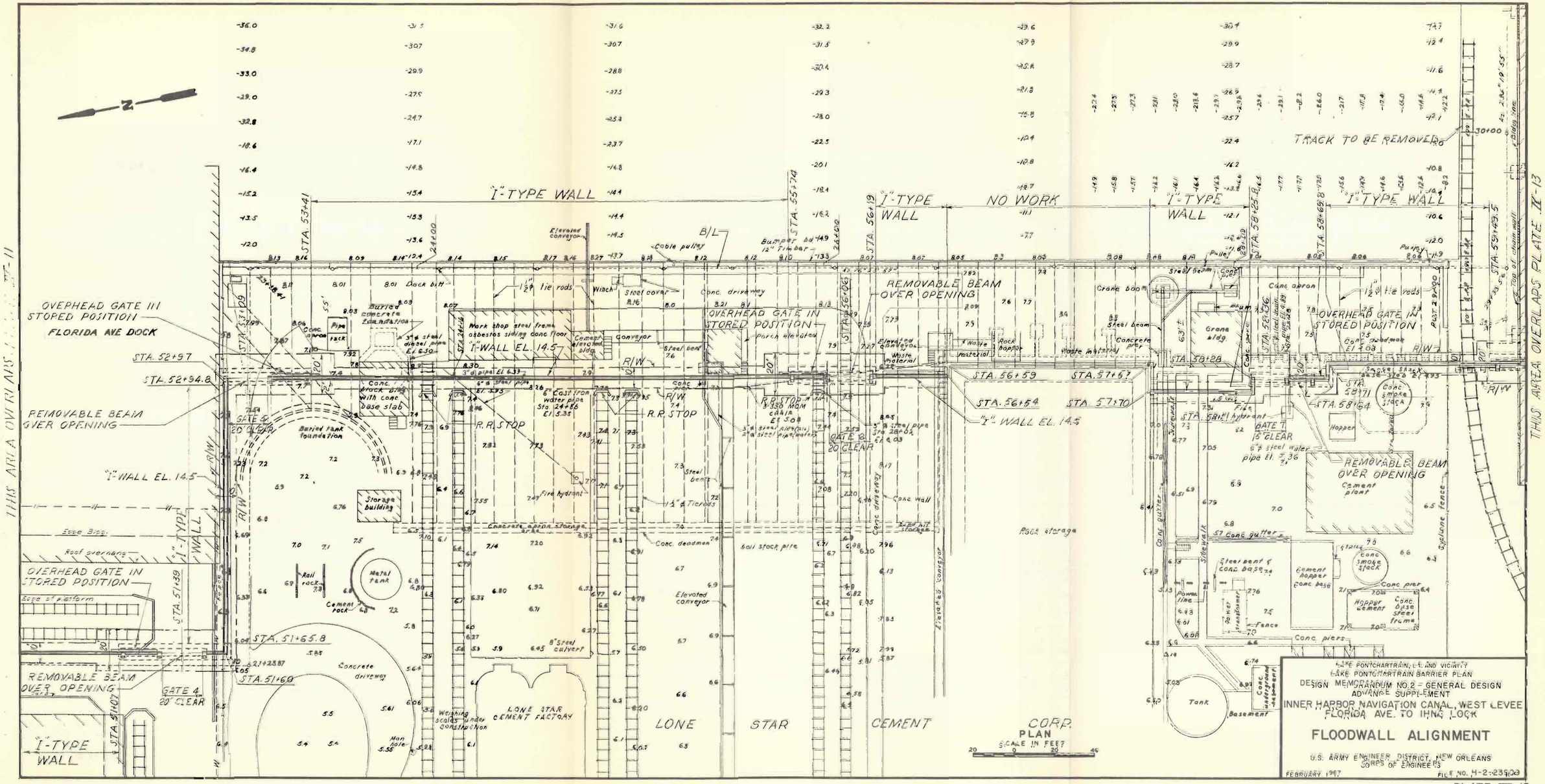
FLOODWALL ALIGNMENT

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

FEBRUARY 1967

FILE NO. H-2-23909

PLATE IV-11

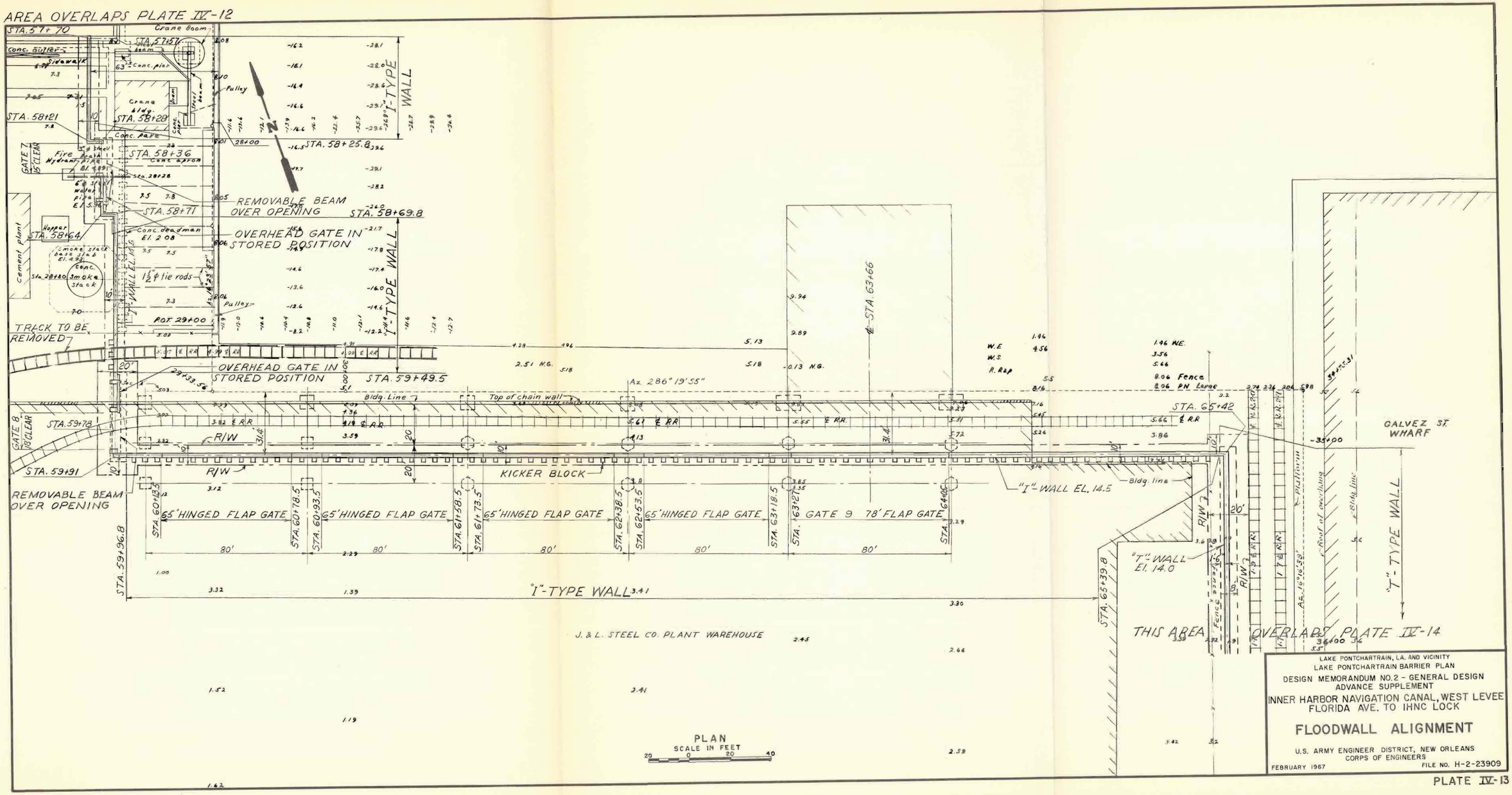


LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

FLOODWALL ALIGNMENT

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

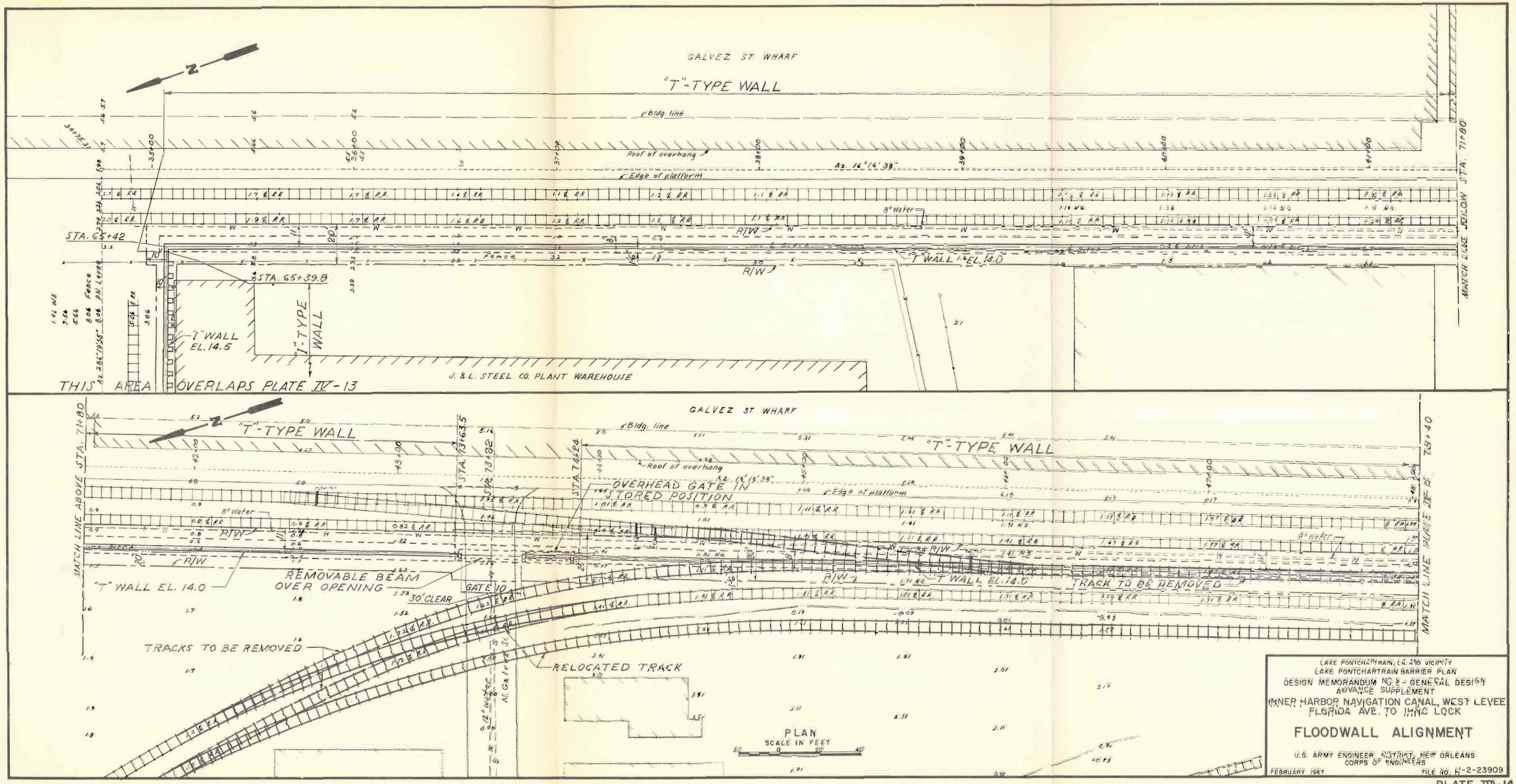
FEBRUARY 1967 FILE NO. H-2-23903



LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

FLOODWALL ALIGNMENT

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909



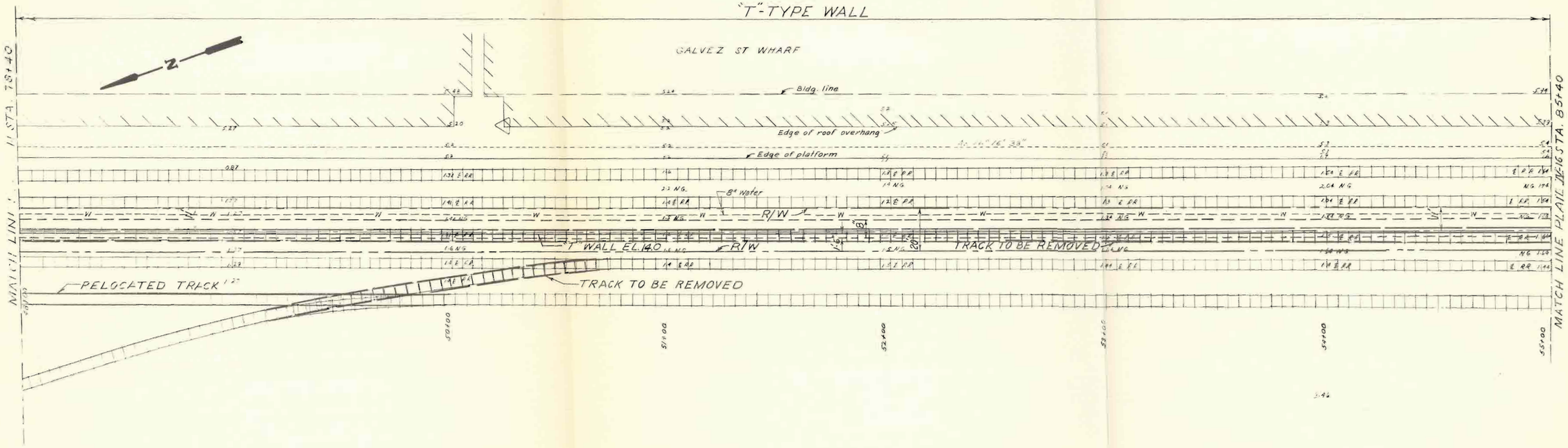
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 5 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

FLOODWALL ALIGNMENT

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

FEBRUARY 1967 FILE NO. H-2-23909

PLATE IV-14



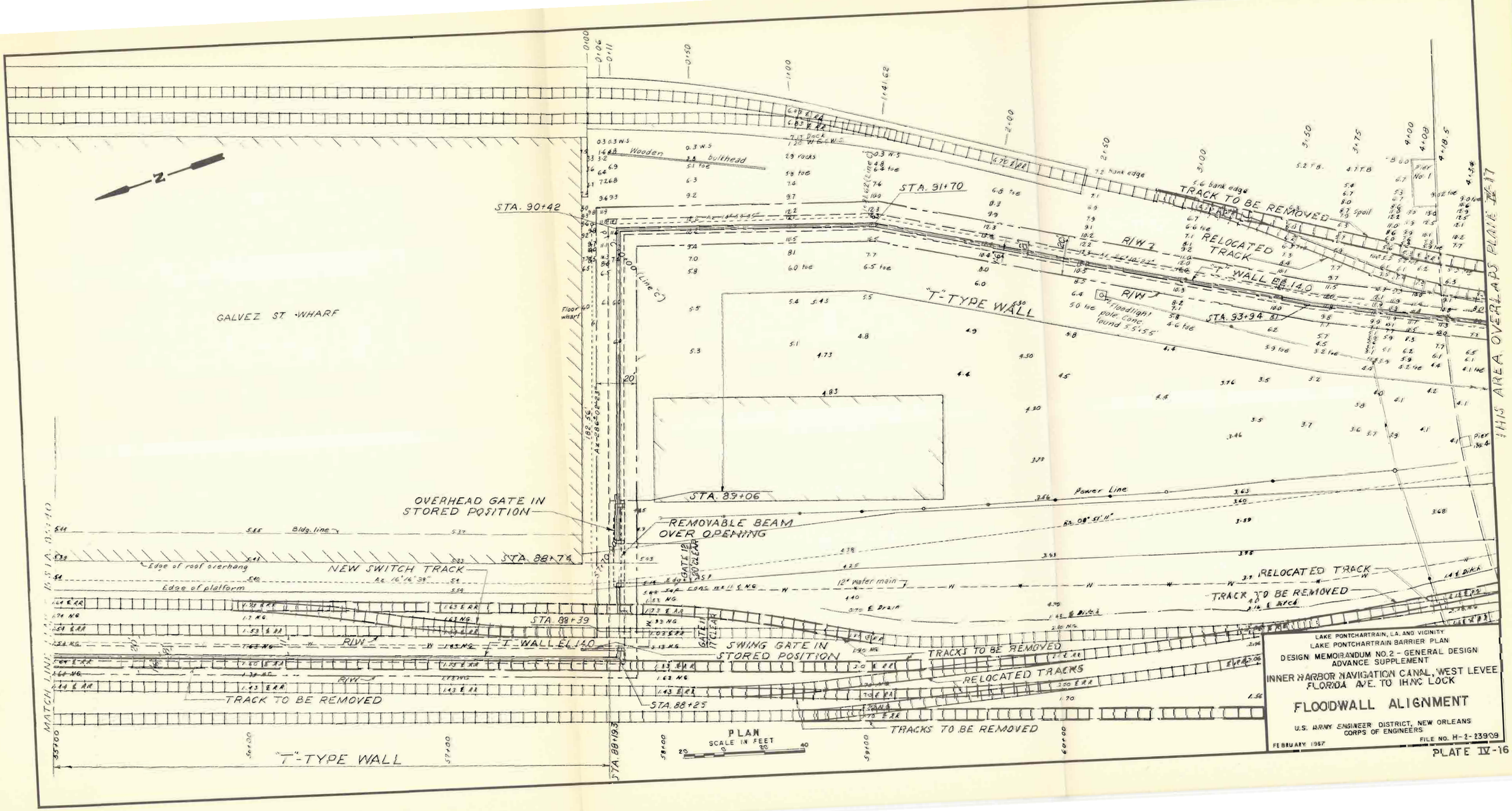
PLAN
SCALE IN FEET
20 0 20 40

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

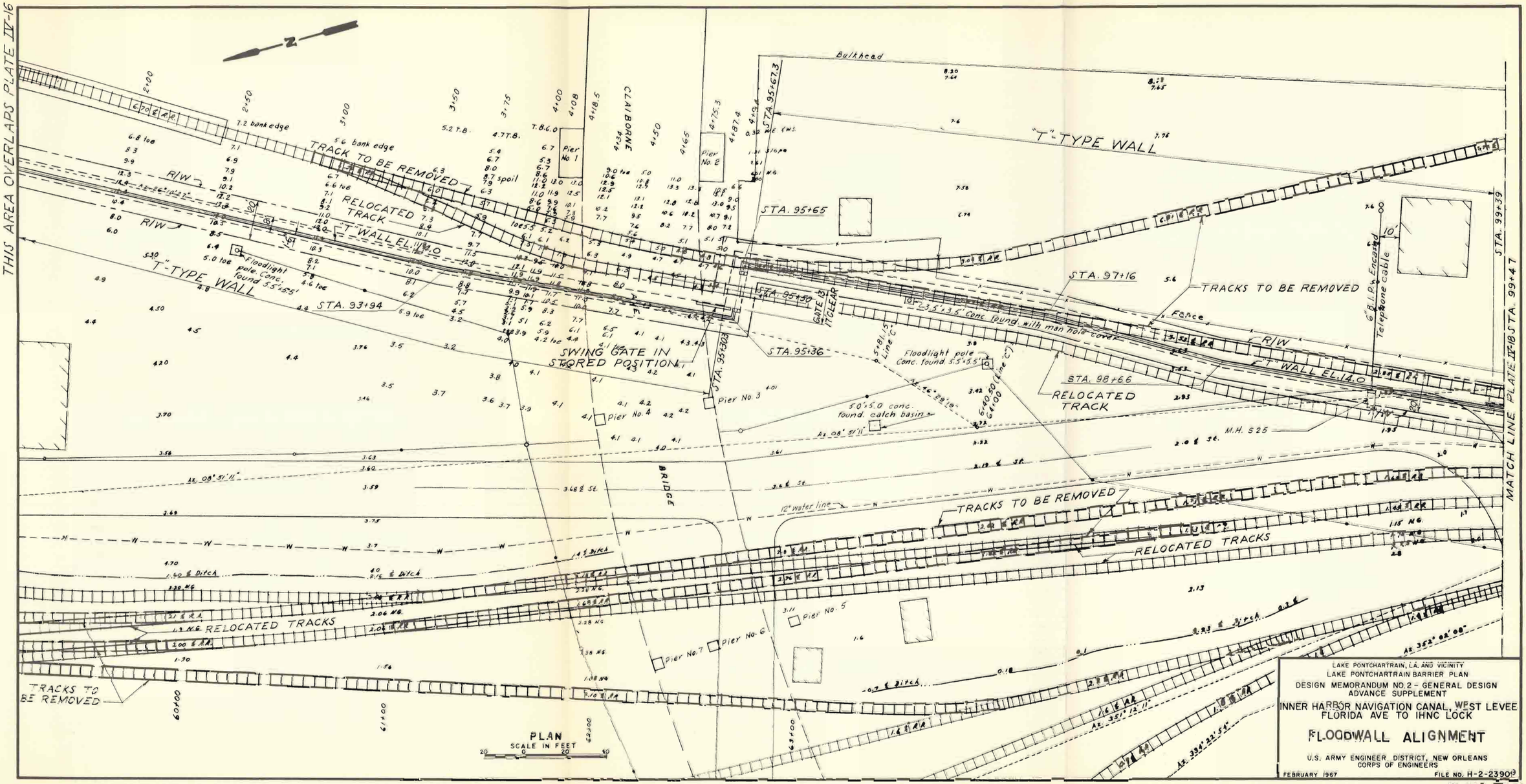
FLOODWALL ALIGNMENT

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909

PLATE IV-15


GALVEZ ST. WHARF
STA. 90+42
STA. 91+70
STA. 93+94
STA. 89+06
STA. 88+74
STA. 88+39
STA. 88+25
STA. 88+193
"T"-TYPE WALL
"T" WALL EL. 14.0
TRACK TO BE REMOVED
RELOCATED TRACK
R/W
Floodlight pole Conc. found 5.5'x5.5'
Wooden bulkhead
Floor wharf
Bldg. line
Edge of roof overhang
Edge of platform
Power Line
12" water main
OVERHEAD GATE IN STORED POSITION
REMOVABLE BEAM OVER OPENING
NEW SWITCH TRACK
SWING GATE IN STORED POSITION
TRACKS TO BE REMOVED
RELOCATED TRACKS
GATE 12 20' CLEAR
GATE 11 17' CLEAR
MATCH LINE
THIS AREA OVERLAPS PLATE IV-17
PLAN
SCALE IN FEET
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
FLOODWALL ALIGNMENT
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1957
FILE NO. H-2-23909
PLATE IV-16

THIS AREA OVERLAPS PLATE IV-16



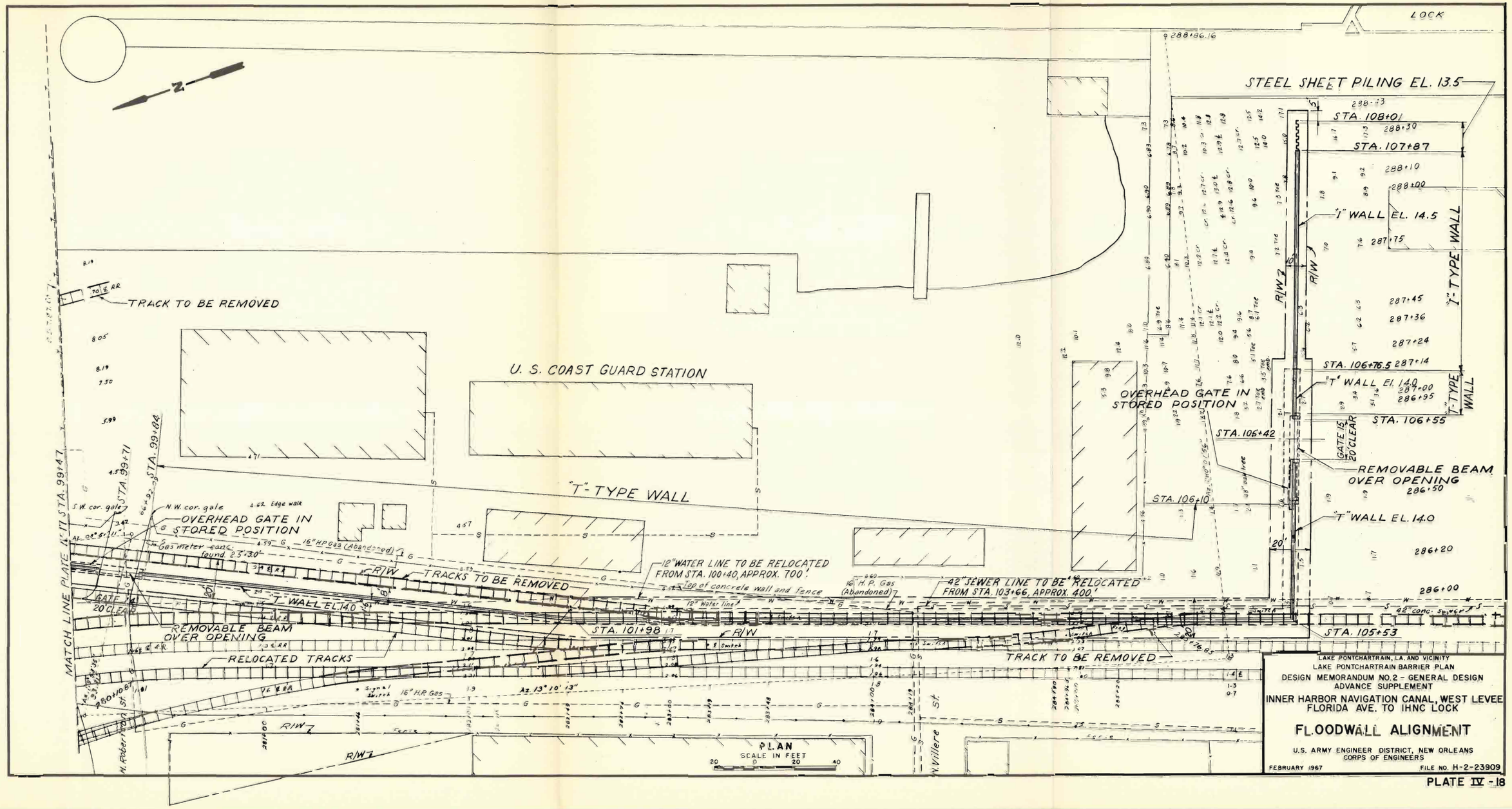
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE TO IHNC LOCK

FLOODWALL ALIGNMENT

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

FEBRUARY 1967 FILE NO. H-2-23909



LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT

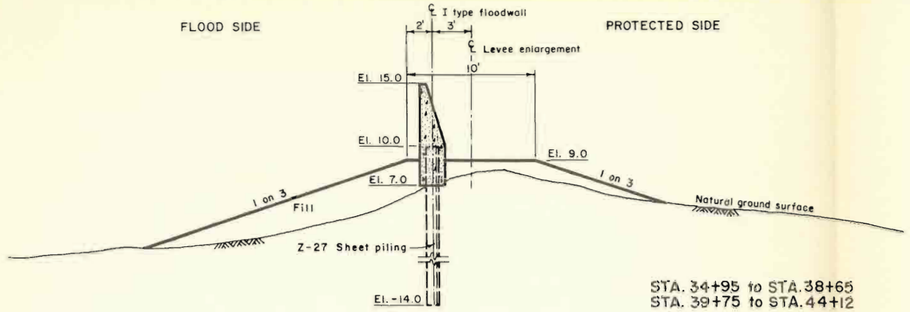
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

FLOODWALL ALIGNMENT

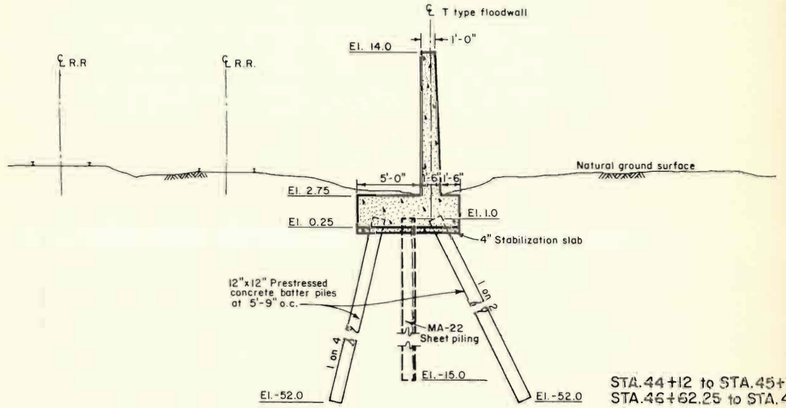
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-23909

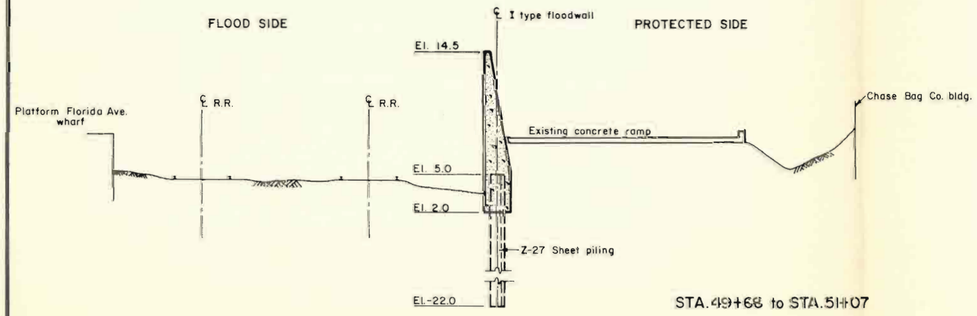
PLATE IV - 18



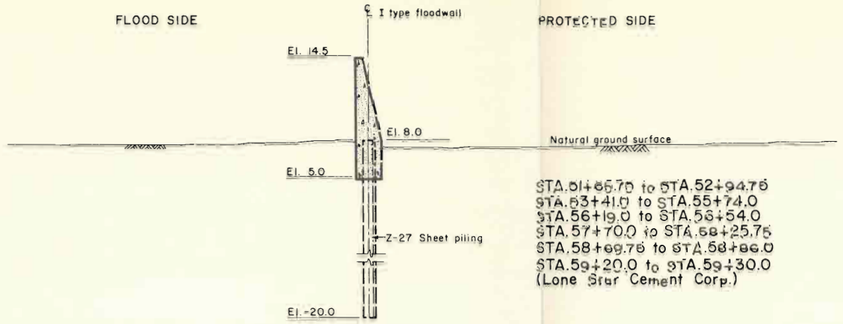
STA. 34+95 to STA. 38+65
STA. 39+75 to STA. 44+12



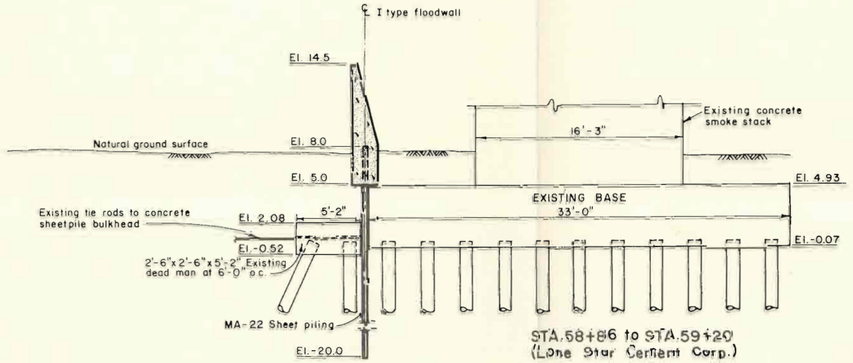
STA. 44+12 to STA. 45+74
STA. 46+62.25 to STA. 49+68



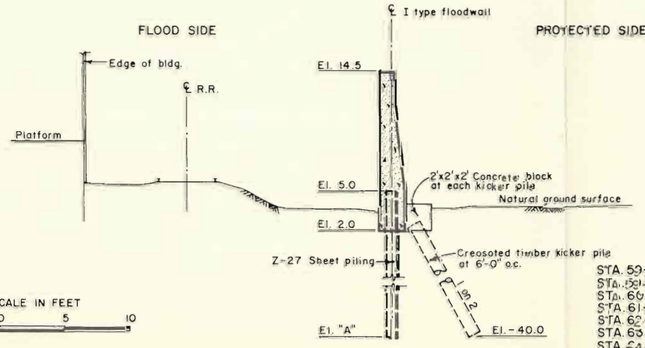
STA. 49+68 to STA. 51+07



STA.51+86.70 to STA.52+94.76
STA.53+41.0 to STA.55+74.0
STA.56+19.0 to STA.56+54.0
STA.57+70.0 to STA.58+25.75
STA.58+69.75 to STA.58+86.0
STA.59+20.0 to STA.59+30.0
(Lone Star Cement Corp.)



STA.58+86 to STA.59+20
(Lone Star Cement Corp.)



SCALE IN FEET
5 0 5 10

	SHEET PILE TIP EL. "A"
STA.59+30 to STA.59+49.5	-36.0
STA.59+93.75 to STA.60+18.60	-36.0
STA.60+73.50 to STA.60+93.50	-36.0
STA.61+58.50 to STA.61+73.50	-36.0
STA.62+38.50 to STA.62+63.50	-36.0
STA.63+18.00 to STA.63+27.0	-36.0
STA.64+05 to STA.65+39.75	-29.0

(Jones & Laughlin Steel Corp.)

NOTE:
Elevations are in feet referred to M.S.L.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
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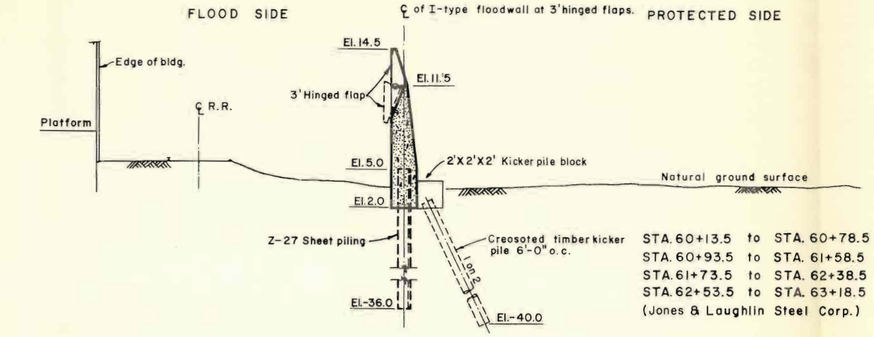
FLORIDA AVE. TO IHNC LOCK

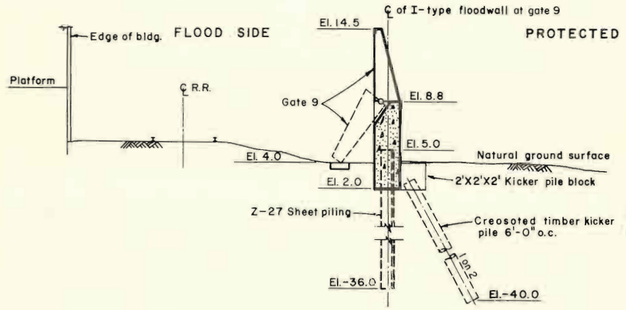
DESIGN SECTIONS

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

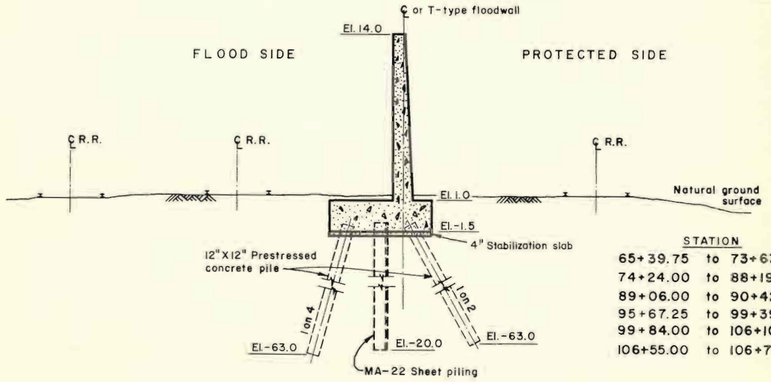
FEBRUARY 1967 FILE NO. H-2-23909



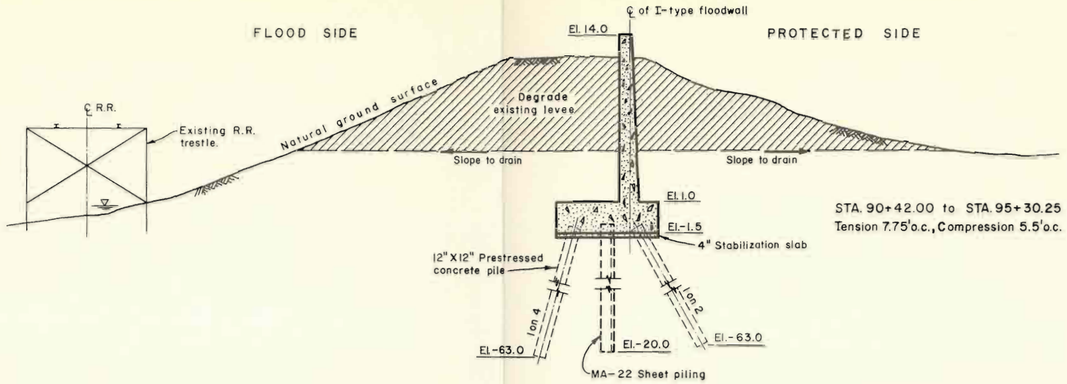
STA. 60+13.5 to STA. 60+78.5
STA. 60+93.5 to STA. 61+58.5
STA. 61+73.5 to STA. 62+38.5
STA. 62+53.5 to STA. 63+18.5
(Jones & Laughlin Steel Corp.)



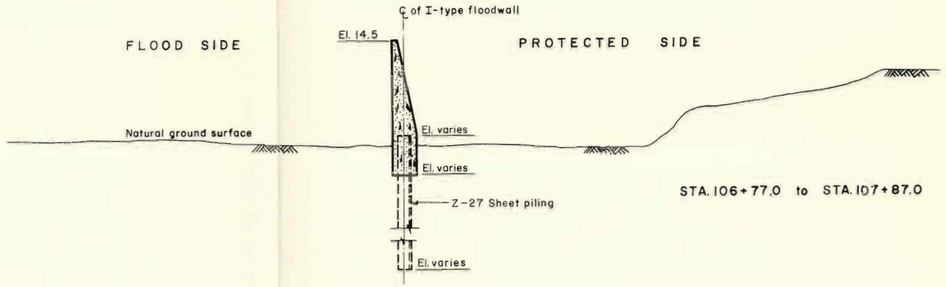
GATE 9
STA. 63+27.0 to STA. 64+05.0
(Jones & Laughlin Steel Corp.)



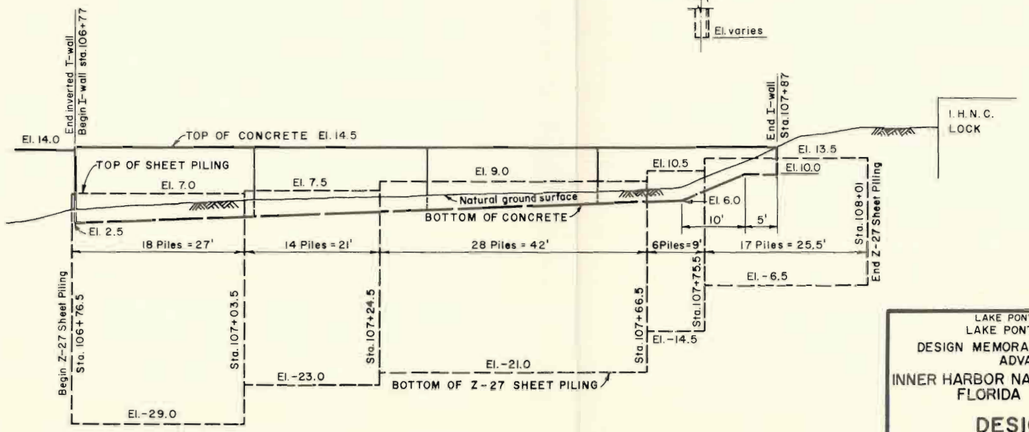
STATION			BATTER PILE SPACING TENSION	BATTER PILE SPACING COMPRESSION
65+39.75	to	73+63.50	7.00'o.c.	5.5'o.c.
74+24.00	to	88+19.25	7.00'o.c.	5.5'o.c.
89+06.00	to	90+42.00	7.75'o.c.	5.0'o.c.
95+67.25	to	99+39.00	7.00'o.c.	5.5'o.c.
99+84.00	to	106+10.00	7.25'o.c.	5.5'o.c.
106+55.00	to	106+77.00	7.25'o.c.	5.5'o.c.



STA. 90+42.00 to STA. 95+30.25
Tension 7.75'o.c., Compression 5.5'o.c.



STA. 106+77.0 to STA. 107+87.0



ELEVATION
STA. 106+76.5 to STA. 108+01.0

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
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FLORIDA AVE. TO IHNC LOCK

DESIGN SECTIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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FEBRUARY 1967
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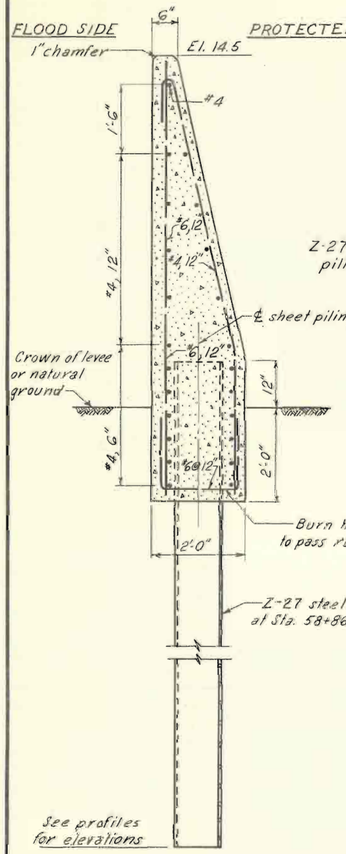
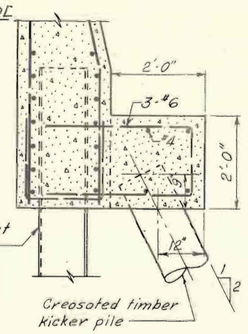


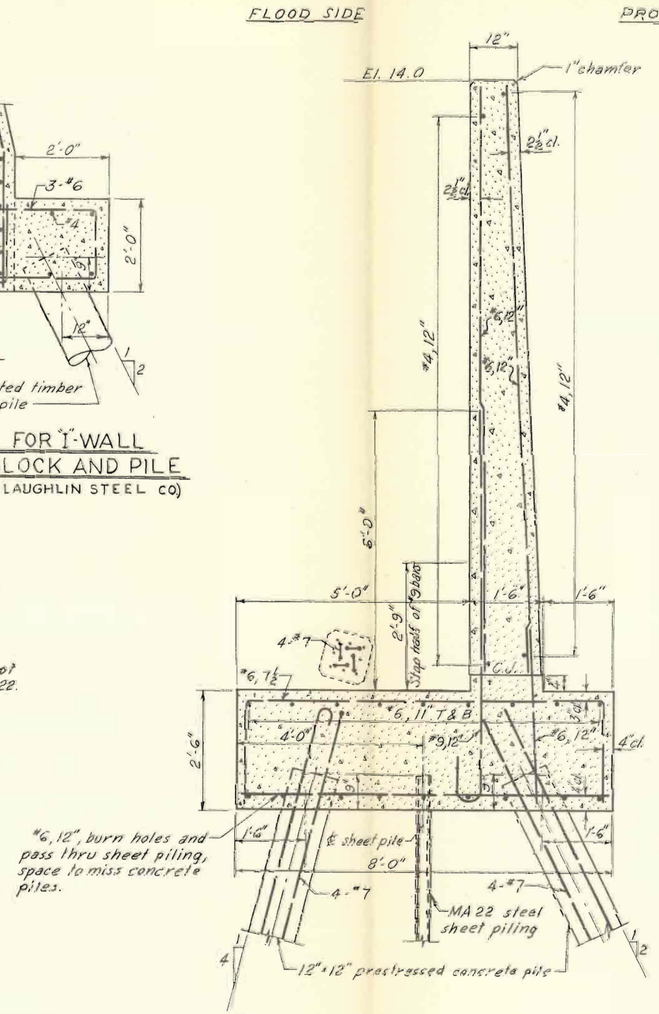
TYPICAL DETAIL "I"-TYPE WALL

Scale 3/4"=1'-0"



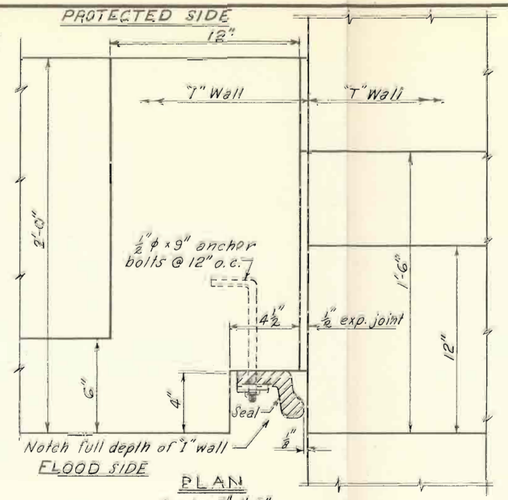
DETAIL FOR "I"-WALL KICKER BLOCK AND PILE

(IN JONES AND LAUGHLIN STEEL CO)



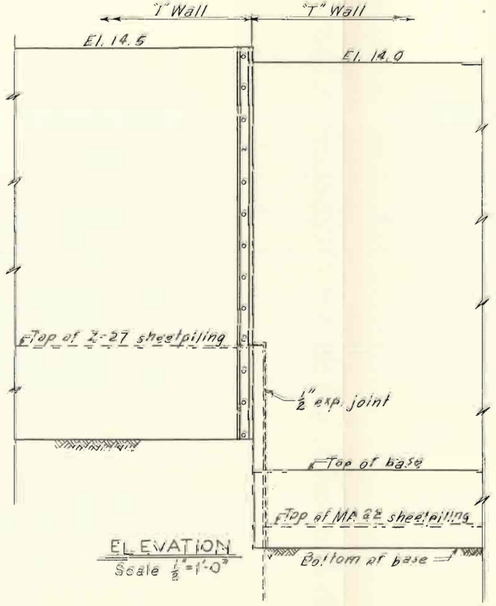
TYPICAL DETAIL "T"-TYPE WALL

Scale 3/4"=1'-0"



PLAN

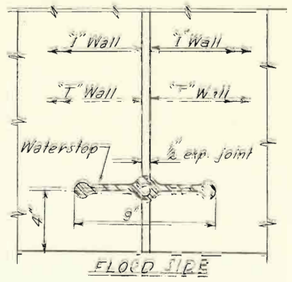
Scale 3"=1'-0"



ELEVATION

Scale 1/2"=1'-0"

TYPICAL JOINT BETWEEN "I"-WALL AND "T"-WALL



TYPICAL FLOODWALL JOINT

Scale 3"=1'-0"

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
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TYPICAL WALL SECTIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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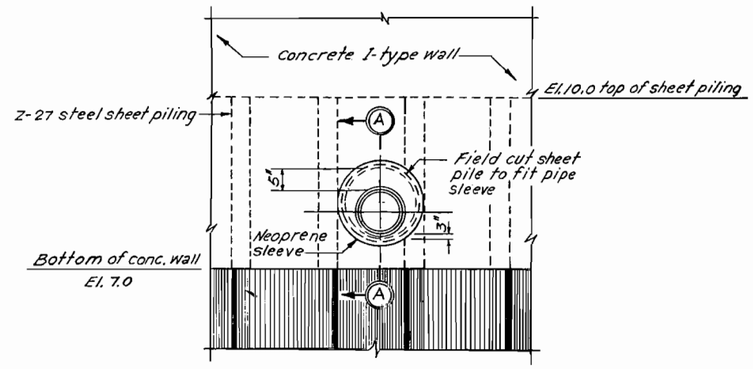
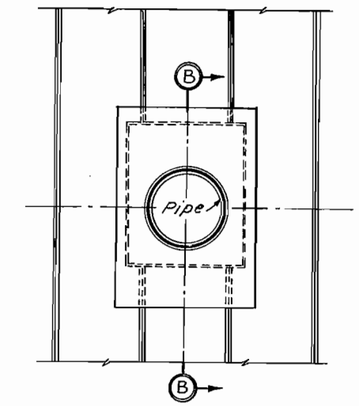
FEBRUARY 1967 FILE NO. H-5-239109

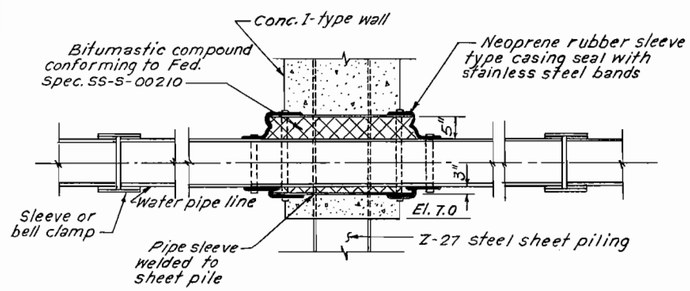
PLATE IV-21



ELEVATION



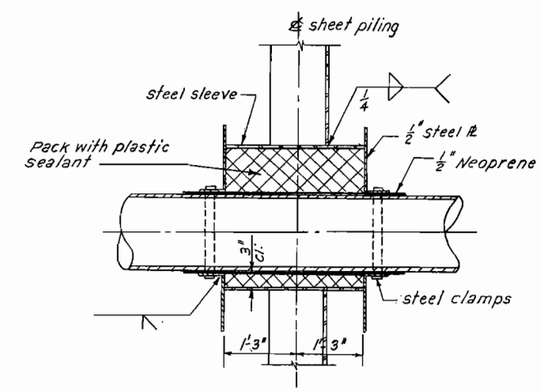
ELEVATION



SECTION A-A

DETAILS OF WATER PIPE LINES, SEWER LINES AND CABLE CROSSINGS

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



SECTION B-B

TYPICAL DETAIL OF 16"ϕ H.P.G.
48"ϕ WATER MAIN
54"ϕ SEWER FORCE MAIN

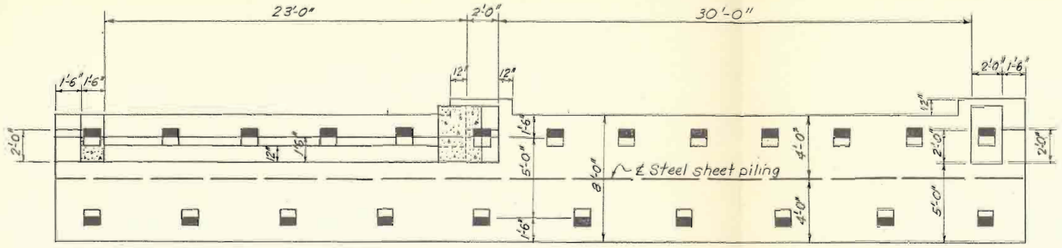
Scale: $\frac{3}{4}$" = 1'-0"

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

UTILITY CROSSINGS

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

FEBRUARY 1967 FILE NO. H-2-23909



PLAN AT EL. 23.56

SCALE: 1/4" = 1'-0"

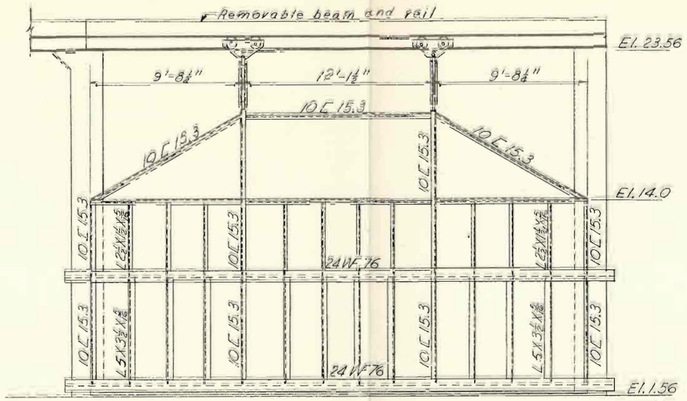
GATE NOT SHOWN

LEGEND

1 on 2 Batter (Compression Pile)

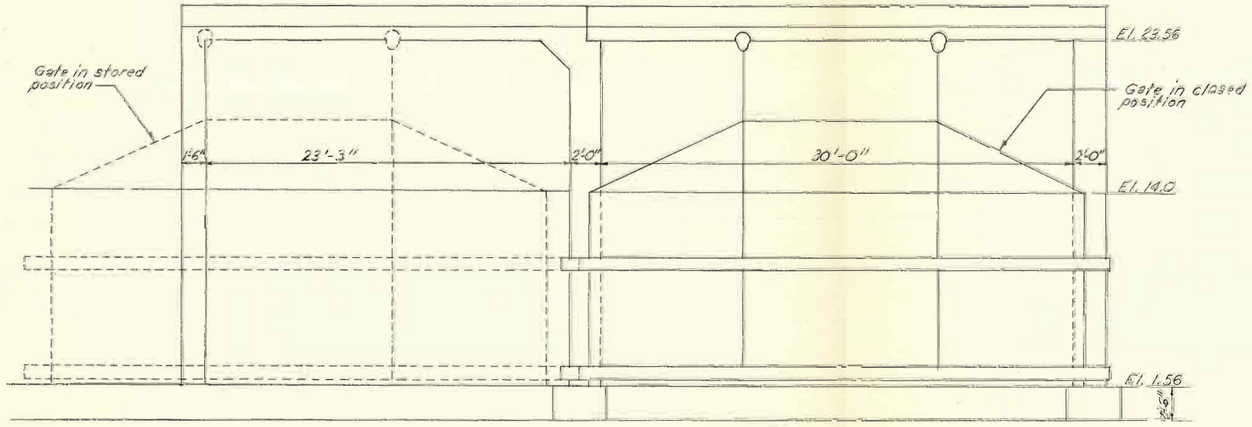
1 on 4 Batter (Tension Pile)

Note: All piles 12"X12" prestressed concrete piles.



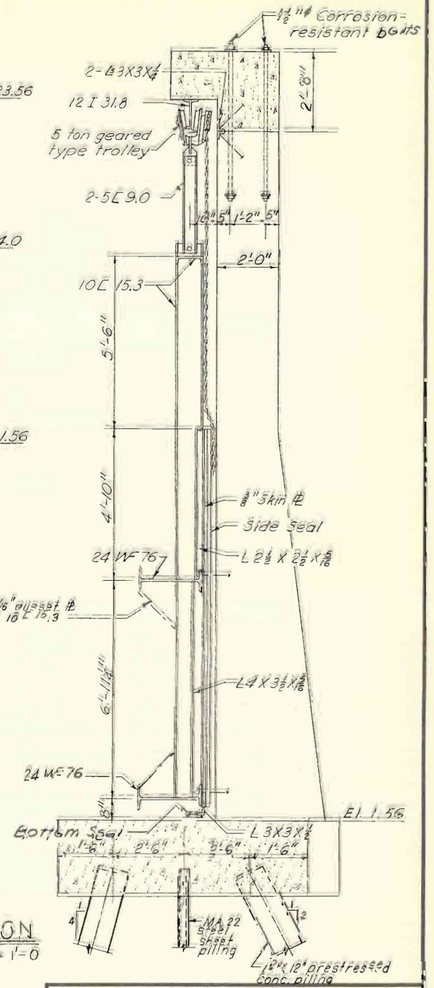
FLOOD SIDE ELEVATION OF GATE

SCALE: 1/4" = 1'-0"



ELEVATION

SCALE: 1/8" = 1'-0"



SECTION

SCALE: 1/2" = 1'-0

GATE 10

GALVEZ ST. GAP CLOSURE

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
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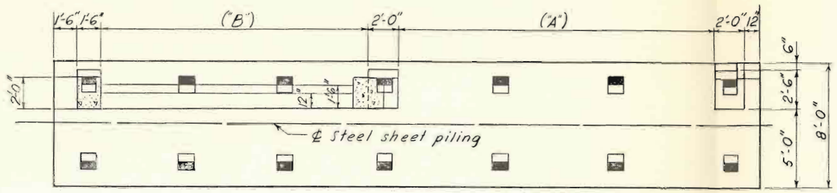
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
GATE 10 - GALVEZ ST.
OVERHEAD ROLLER GATE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1967
FILE NO. H-2-23909



PLAN AT ELEVATION 23.75

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

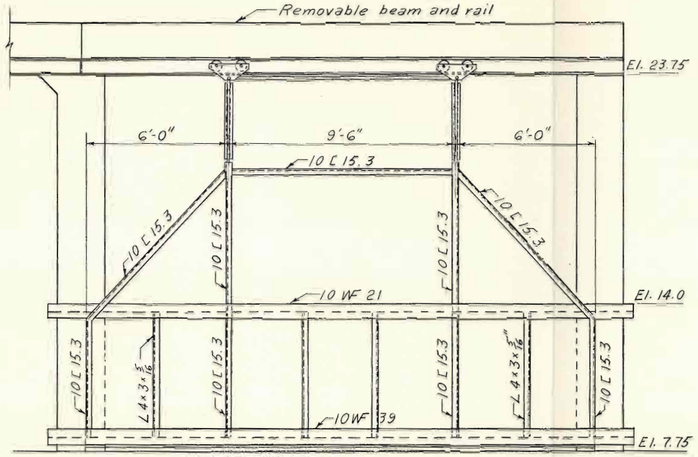
GATE NOT SHOWN

LEGEND

1 on 2 Batter (Compression Pile)

1 on 4 Batter (Tension Pile)

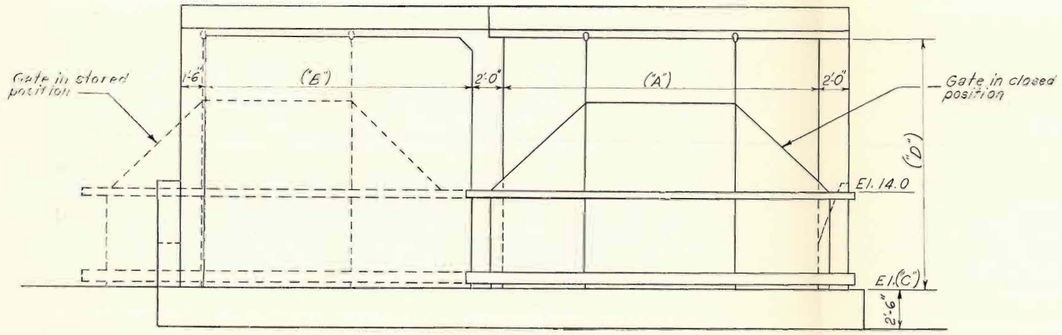
Note: All piles 12" X 12" prestressed concrete piles.



FLOOD SIDE ELEVATION OF GATE

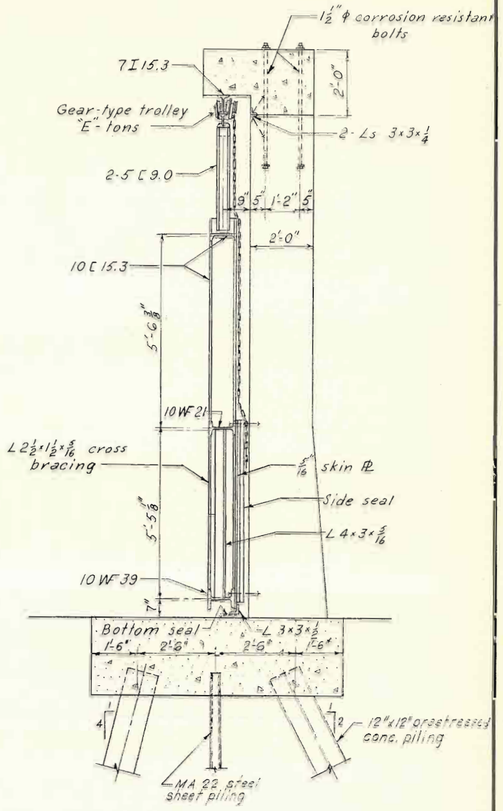
Scale $\frac{3}{8}$" = 1'-0"

Note:
Gate No. 5 shown. Gates listed in "Schedule" below are similar except for size and spacing of bars.



ELEVATION

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



SECTION

Scale $\frac{1}{2}$" = 1'-0"

GATE SCHEDULE

GATE No.	GAP CLOSURE	HORIZ. CL. "A"	STORAGE BAY "B"	SILL EL. "C"	VERT. CL. "D"	TROLLEY-"E"-TONS
1	Street	20'	17'-0"	2.75	16'	2
2	Street	20'	17'-0"	7.2	16'	2
4	Street	20'	17'-0"	7.0	16'	1½
5	Street	20'	17'-0"	7.75	16'	1½
6	Railroad	20'	17'-0"	8.0	22'	1½
7	Street	15'	12'-0"	7.25	22'	1½
8	Railroad	18'	15'-0"	4.3	22'	2
12	Street	20'	17'-0"	5.1	16'	2
14	Street	20'	17'-0"	3.25	16'	3
15	Street	20'	17'-0"	2.0	16'	3

Note: Top beam and rail of gates 6 and 8 will not be removable.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
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FLORIDA AVE. TO IHNC LOCK

TYPICAL FLOODGATE
OVERHEAD ROLLER GATE

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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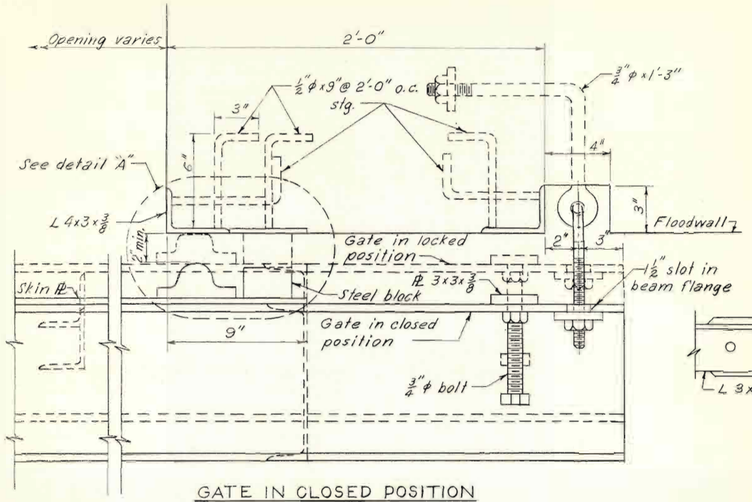
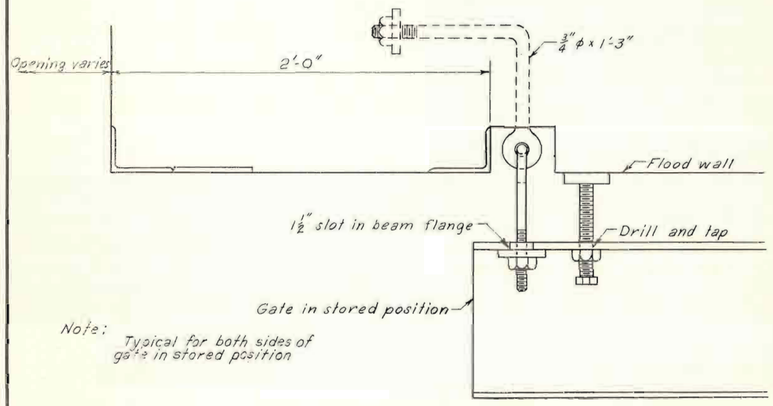


GATE IN CLOSED POSITION

PLAN

Scale 3"=1'-0'

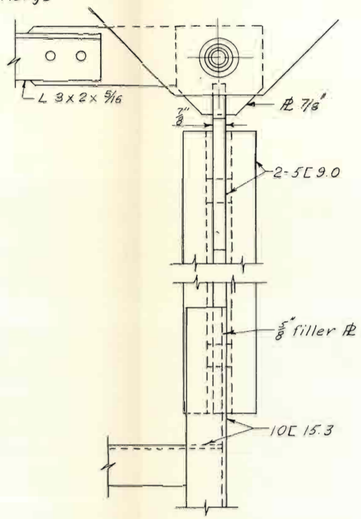


Note: Typical for both sides of gate in stored position

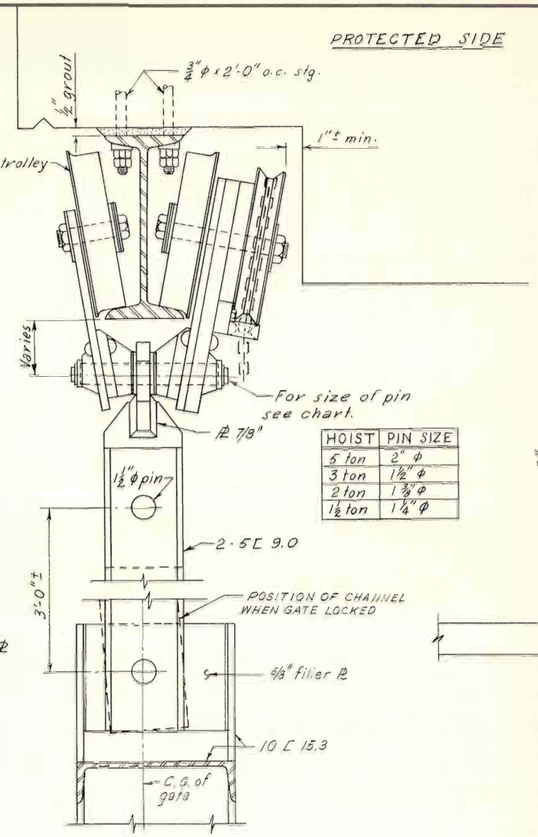
GATE IN STORED POSITION

PLAN

Scale 3"=1'-0"



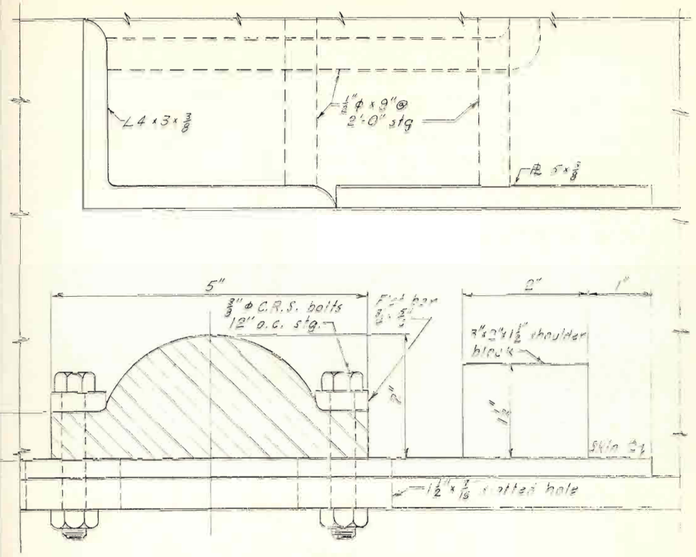
VIEW AT FLOOD SIDE



HOIST	PIN SIZE
5 ton	2" φ
3 ton	1 1/2" φ
2 ton	1 3/8" φ
1 1/2 ton	1 1/4" φ

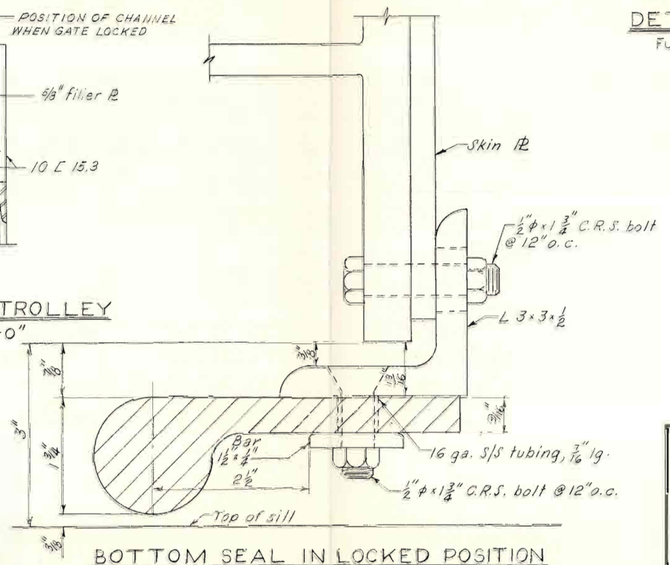
SECTION THRU TROLLEY

Scale 3"=1'-0"



DETAIL "A"

Full scale



BOTTOM SEAL IN LOCKED POSITION

Full scale

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
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INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

OVERHEAD ROLLER GATE
DETAILS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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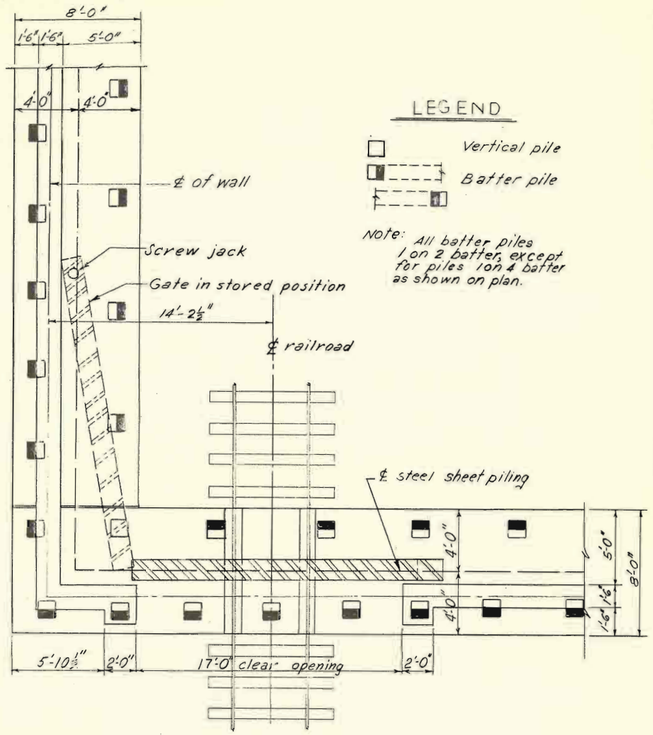


PLAN
Scale ¼"=1'-0"

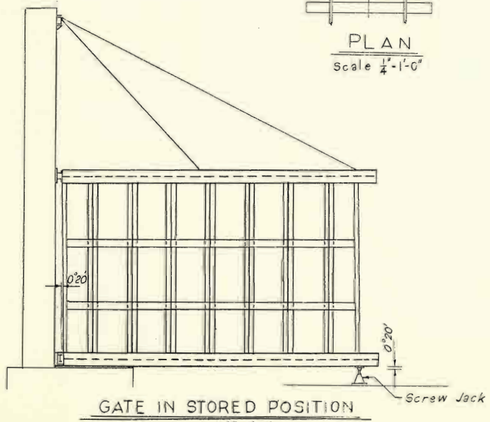
LEGEND

Vertical pile
Batter pile

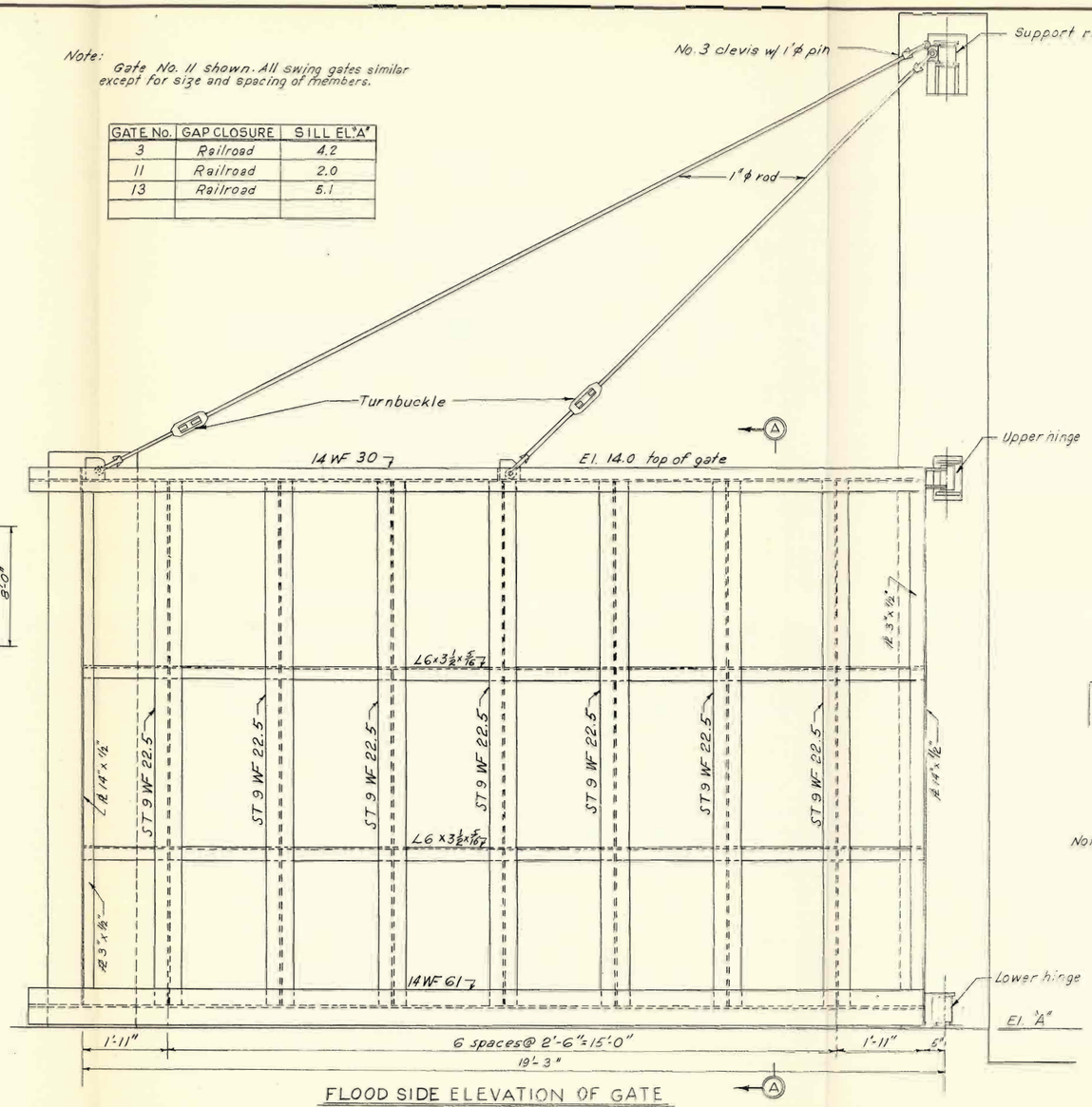
Note: All batter piles 1 on 2 batter, except for piles 1 on 4 batter as shown on plan.

Note: Gate No. 11 shown. All swing gates similar except for size and spacing of members.

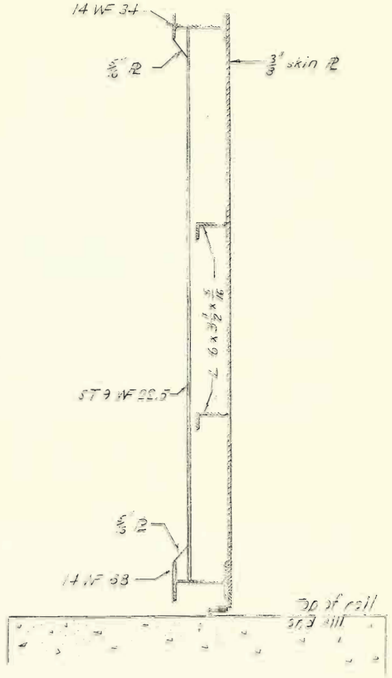
GATE No.	GAP CLOSURE	SILL EL."A"
3	Railroad	4.2
11	Railroad	2.0
13	Railroad	5.1



GATE IN STORED POSITION
Scale: ¼"=1'-0"



FLOOD SIDE ELEVATION OF GATE
Scale: ¾"=1'-0"



SECTION A-A
Scale ¼"=1'-0"

Note: For details see plate IV-27 and IV-28

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
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FLORIDA AVE. TO IHNC LOCK

TYPICAL FLOODGATE
SWING GATE

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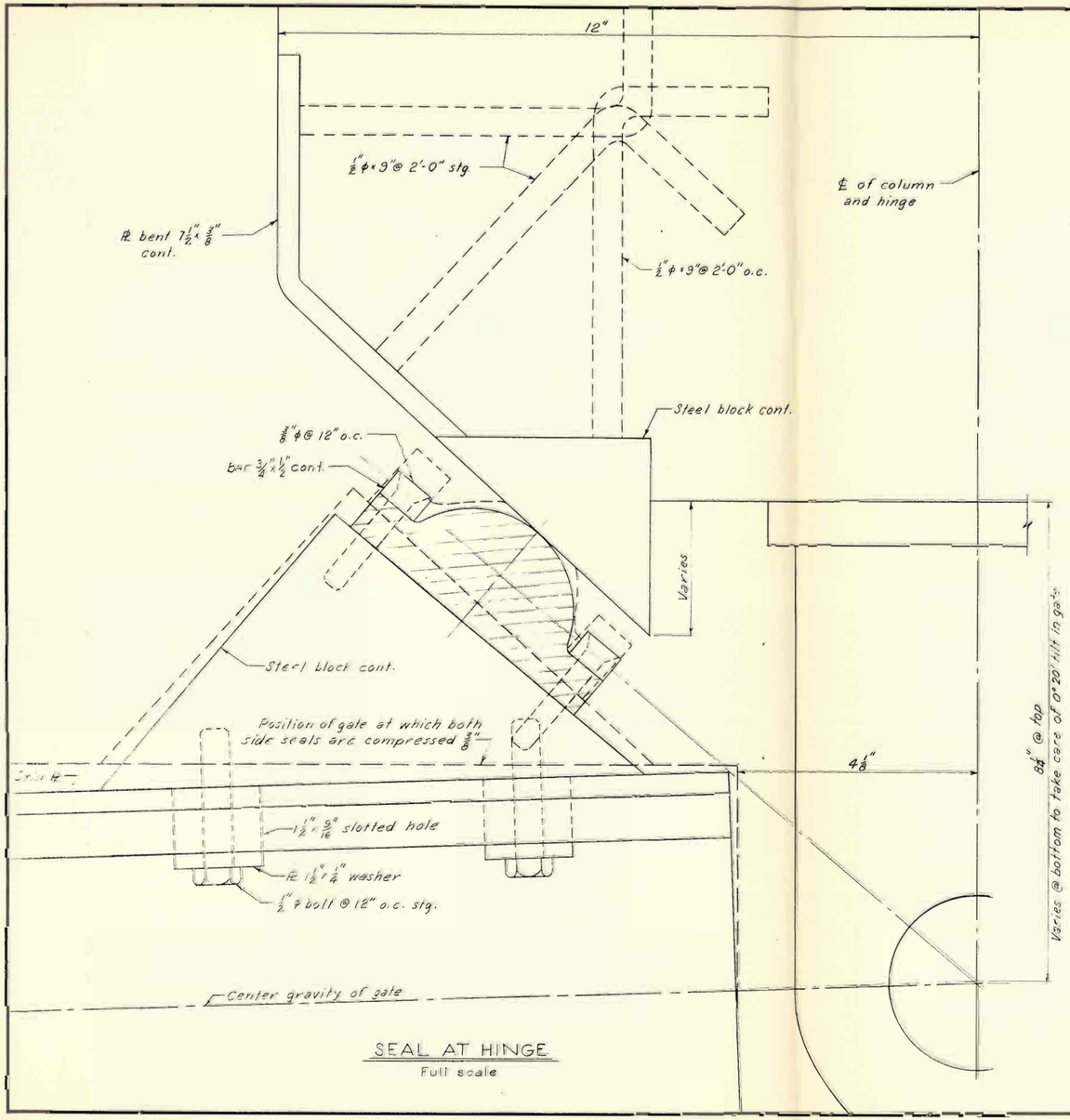
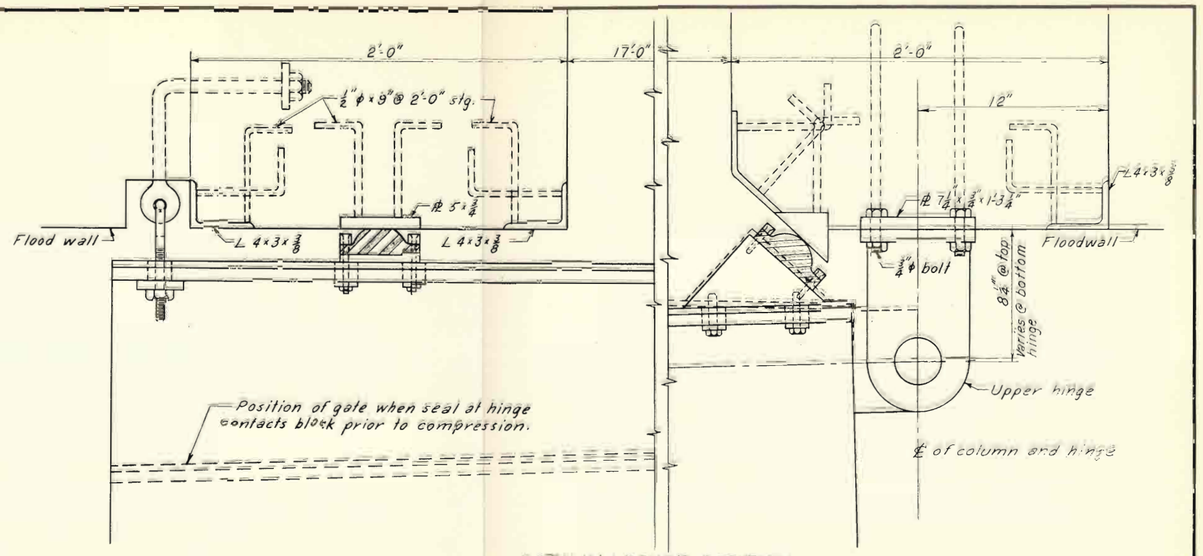
CORPS OF ENGINEERS
FEBRUARY 1967 FILE NO. H-2-23909



SEAL AT HINGE

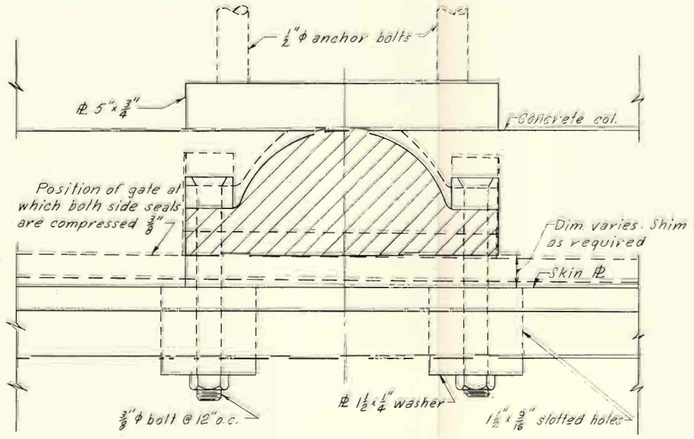
Full scale



GATE IN LOCKED POSITION

PLAN VIEW OF SWING GATE AT UPPER HINGE

Scale 3"=1'-0"



SEAL AT COLUMN

Full scale

LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
ADVANCE SUPPLEMENT

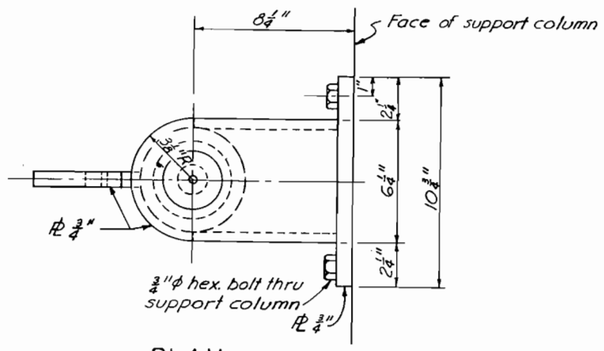
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

SWING GATE
DETAILS

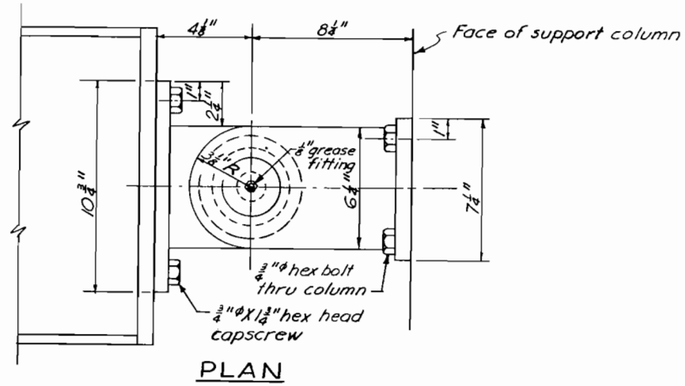
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

FEBRUARY 1967 FILE NO. H-2-53909

PLATE IV-27



PLAN



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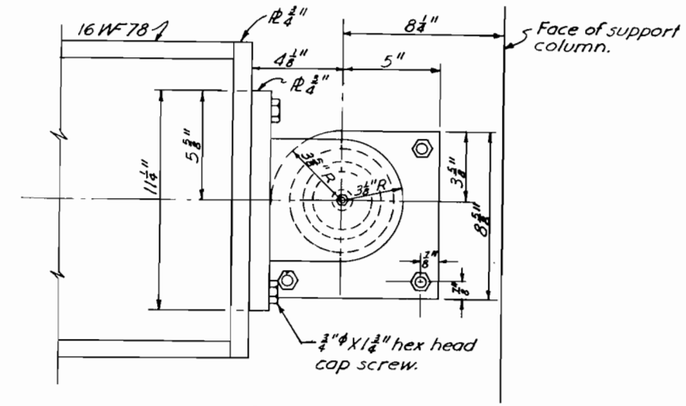
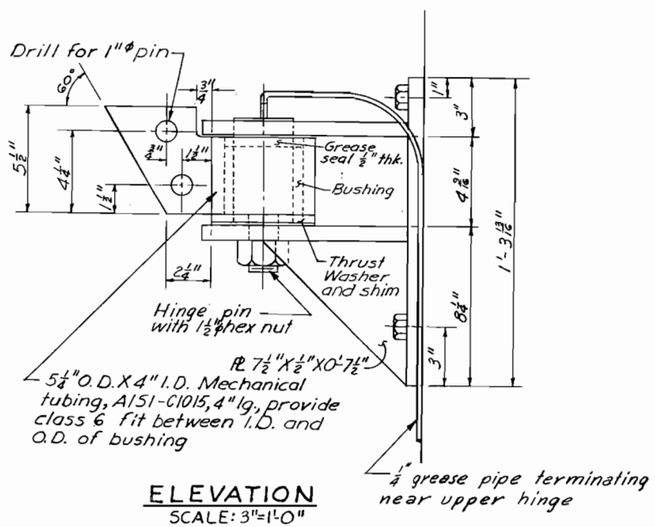
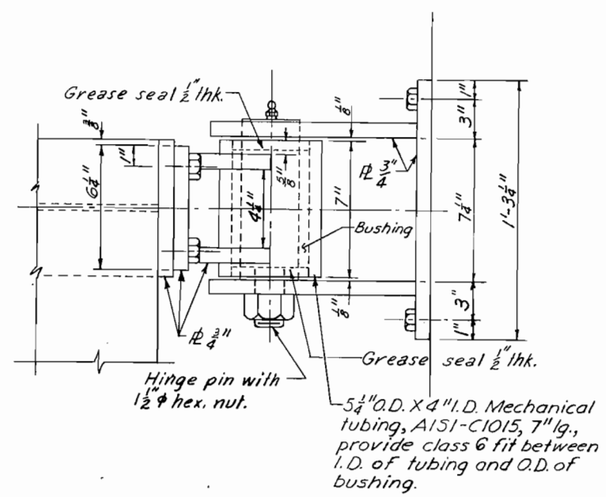


PLAN



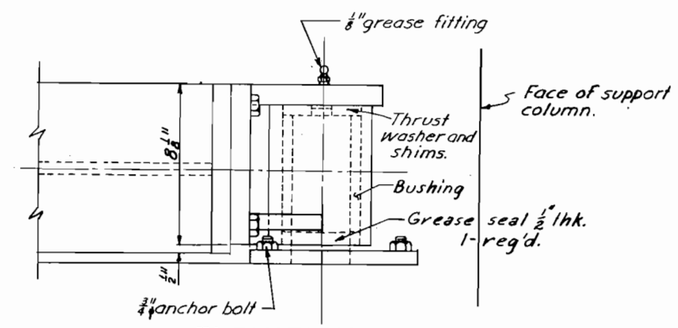
ELEVATION
SCALE: 3" = 1'-0"

SUPPORT ROD HINGE



ELEVATION
SCALE: 3" = 1'-0"

UPPER HINGE



ELEVATION
SCALE: 3" = 1'-0"

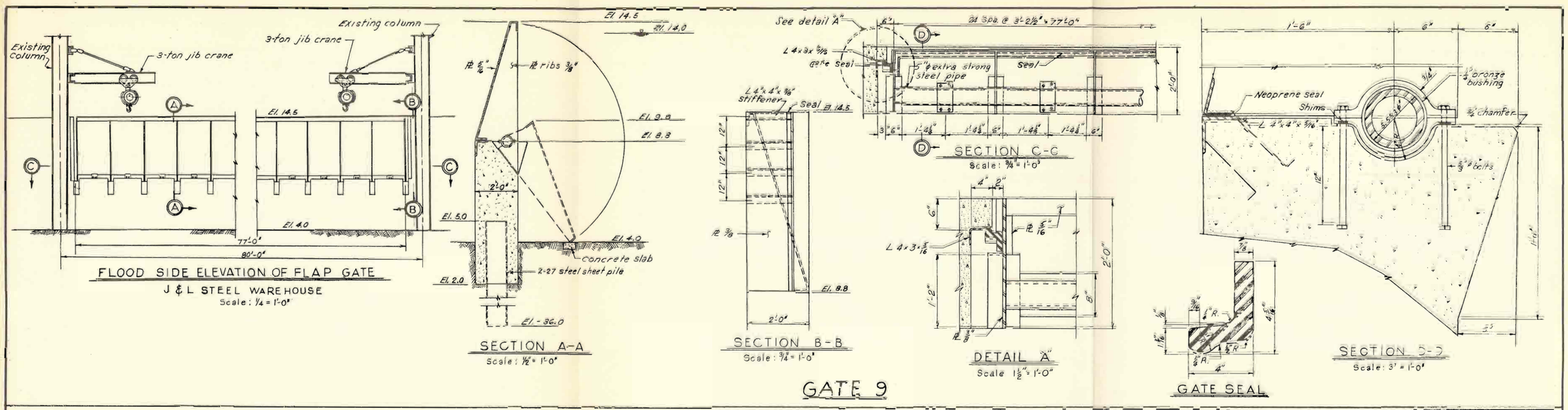
LOWER HINGE

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK

SWING GATE
HINGE DETAILS

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

FEBRUARY 1967 FILE NO. H-2-23909



LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL, WEST LEVEE
FLORIDA AVE. TO IHNC LOCK
GATE 9 AND HINGED FLAPS
JONES AND LAUGHLIN STEEL CORP.
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEBRUARY 1957
FILE NO. H-2-34904

PLATE IV-29

SECTION V - MECHANICAL

1. Overhead roller gates. Overhead roller gates will be operated with geared-type manually operated trolleys as shown on plate IV-25. These trolleys will be equal to the Yale and Towne Mfg. Co., Models T-Geared Type.

2. Flap gates. Flap gates will be operated with wall bracket jib cranes equipped with manually operated trolley hoists, as shown on plate IV-29. The jib cranes will be equal to Detroit Hoist and Machine Co. Model D-1900, and the trolley hoists will be equal to Harnischleger Mfg. Co. catalog No. 135-Plain.

SECTION VI - RELOCATIONS AND MODIFICATIONS

1. General. Under the authorizing law, local interests are responsible for the accomplishment of all "...necessary alterations and relocations to roads, railroads, pipelines, cables, wharves, drainage structures and other facilities required by the construction of the project;..." Because of the complex coordination of construction and relocation work engendered by the highly congested conditions in the area, it is considered essential that the construction and relocation (except for drainage rectification) work be accomplished as a single coordinated job. Ordinarily, this would require a local interest contribution prior to starting construction. In the instant case, however, all work is to be accomplished under the auspices of the Orleans Levee District, local sponsor for the project, and all construction funds for the work will be made available by that agency, subject to credit toward the overall contribution required of local interests. The Orleans Levee District is undertaking the construction in the absence of Federal funds as a means of expediting the realization of protection in a critical area.

2. Included in the required relocations are utilities, concrete ramps, railways, electrical raceways, modification to two industrial plants, and drainage rectification work.

3. Utility modifications. The following utility crossings will be modified to pass through the floodwall as shown on plate IV-22. The locations of the crossings are shown in plan on plates IV-9 through IV-18 and in profile on plates IV-2 through IV-6.

Station	Utility
36+06	54" sewer force main
36+17	48" water main
45+80	8" water main
45+98	Telephone cable
51+53	8" water main
53+90	3" diesel pipeline
54+11	3" pipeline
54+80	6" water pipeline
55+41	3" air pipeline
55+41	2" water pipeline
55+95	3" water pipeline
58+46	6" water pipeline
58+46	3" drain pipeline
73+82	12" water pipeline crossing
88+35	8" water main
98+80	6" pipe containing telephone cable
99+75	16" gas pipeline

In addition to the above, approximately 700 linear feet of 12" diameter water main and 400 linear feet of 42" diameter sewer force main in the vicinity of the U. S. Coast Guard Station will have to be relocated. The treatment shown on plate IV-22 will permit the maximum anticipated differential movement between the wall and the crossing utility to occur without damage to the utility or loss of the seal. The congested nature of the area, the excessive cost involved, and vehement objection by the utility owners all combine to make routing of the utilities over the floodwall impracticable.

4. Ramps. Three concrete ramps as described below will be removed and reconstructed.

Location	Description
Florida Avenue Wharf	Concrete slab, 8" thick, covering 1,470 sq.ft.
Chase Bag Company	Concrete slab, 8" thick, covering 6,790 sq.ft.
North Galvez Street crossing	Concrete slab, 8" thick, covering 400 sq.ft.

5. Railways. Approximately 1,420 linear feet of railroad track will be relocated and 3,100 linear feet removed, as shown on plates IV-10 through IV-18. Track relocations will require the acquisition of 0.06 acre of additional rights-of-way.

6. Lone Star Cement Company Plant. Construction of the floodwall inside the plant will require removal and partial (one-half) reconstruction of a concrete block building with a floor area of 625 square feet, and dismantling and reassembly of belt conveyor facilities, including housing. In addition, 31 tie rods for an existing bulkhead will have to be individually cut to permit driving the sheet pile in a reach approximately 350 feet in length. A section of each rod will be removed after first being isolated by attaching a "C" type clamp. The section will then be removed, the piling driven, and the severed rod rejoined by passing a new section of rod through a hole burned in the sheet pile and welding this new section to the existing rod.

7. Jones and Laughlin steel warehouse. In order to drive the sheet piling inside the warehouse, it will be necessary to remove approximately 3,650 square feet of Robinson "Q-Deck" roofing covered with composition material. This section will be replaced, after the piling is driven, with corrugated sheet metal. In

addition, a 500-square foot section of the warehouse north wall, where the floodwall is to pass through, will have to be removed and subsequently replaced. The plan for protection in this area was closely coordinated with the Jones and Laughlin Company and bears their approval.

8. Drainage rectification. The floodwall adjacent to the Galvez Street Wharf will intercept the drainage of an area east of the wall of about 1.7 acres. The intercepted drainage will be accommodated by installing a subsurface collector system and pumping station discharging into the IHNC at the north end of the wharf.

SECTION VII - BEAUTIFICATION

1. General. The construction of the protective works covered herein will alter the existing terrain only to the extent of superimposing a floodwall and levee upon it. No borrow is to be taken from the area and all fill removed from existing levees will be hauled away and disposed of elsewhere. Levee work will be sodded in accordance with standard levee practice. Further opportunity for enhancing the appearance of the area is limited to measures involving the wall itself. In connection with the above, consideration was given to planting of shrubbery or vine-like plants or ivy for wall cover, or alternatively, providing a textured wall surface by use of exposed aggregate or other means. None of the above was, however, deemed worthy of adoption for the reasons outlined below.

2. The area in question is occupied by an industrial complex, and it is considered that the optimum esthetic effect will result from harmonizing the project works involved with their surroundings to the maximum extent practicable. It is considered that the unadorned concrete wall, which is suggestive of massive strength and functionality, will actually create a motif with which the existing structural elements can cohere to produce an overall effect which is both appropriate and pleasing. In time, painting the wall, as is the practice of local interests on other floodwalls in the area, may become desirable.

SECTION VIII - REAL ESTATE REQUIREMENTS

General. All rights-of-way will be acquired by the local agencies involved and furnished without cost to the United States. There will be no acquisition by the United States.

SECTION IX - ESTIMATE OF COST

1. Estimate of cost. The total estimated first cost of the authorized project, based on December 1966 price levels, is $3,240,000, of which $2,270,000 is Federal cost and $970,000 is non-Federal cost. A summary of first cost is shown in table IX-1. The apportionment of cost between Federal and non-Federal interests is shown in table IX-2. A detail estimate of first cost is shown in table IX-3.

TABLE IX-1

Summary of First Cost
(Dec 1966 price level)

Item	Total cost (Conting.,E&D,S&A)
Levee and floodwalls	$ 2,667,000
Lands	181,500
Relocations	391,500
Total first cost	$ 3,240,000

TABLE IX-2

Estimate of Apportionment of Costs Between Federal and non-Federal Interests

Project first cost

Levees and floodwalls	$ 2,667,000
Lands, damages, and relocations	573,000
Amount to be apportioned	3,240,000

Apportionment of costs	Federal 70%	Non-Federal 30%
First cost	$2,270,000	$ 970,000
Less lands, damages, and relocations		573,000
Cash contribution		$ 397,000

TABLE IX-3

Detail Estimate of First Cost

Levees and floodwalls

Item	Description	Estimated quantity	Unit	Unit price	Estimated amount
1	Structure excavation	23,100	c.y.	$ 1.50	$ 34,650
2	12"Ø prest.conc.piling	113,830	l.f.	5.50	626,065
3	MA-22 st'l sheet piling	98,667	s.f.	3.00	296,001
4	Z-27 st'l sheet piling	63,420	s.f.	3.25	206,115
5	Concrete	8,400	c.y.	50.00	420,000
6	Portland cement	11,500	bbl.	5.00	57,500
7	Reinforcing steel	973,700	lb.	0.14	136,318
8	Neoprene rubber gate seals	977	l.f.	5.00	4,885
9	Water stops	2,600	l.f.	4.00	10,400
10	Structural steel	114,000	lb.	0.40	45,600
11	Trolley, plain (2-ton)	15	ea.	150.00	2,250
12	Trolley, plain (3-ton)	2	ea.	300.00	600
13	Trolley, plain (5-ton)	1	ea.	400.00	400
14	Trolley, geared (2-ton)	10	ea.	300.00	3,000
15	Trolley, geared (5-ton)	1	ea.	600.00	600
16	Chain hoist (2-ton)	5	ea.	1,200.00	6,000
17	Chain hoist (3-ton)	2	ea.	1,500.00	3,000
18	Jib crane (2-ton)	5	ea.	1,000.00	5,000
19	Jib crane (3-ton)	2	ea.	1,000.00	2,000
20	Miscl. metal, imbedded metal, hardware, etc.	20,000	lb.	0.65	13,000
21	Cathodic protection	Lump sum	L.S.	50,000.00	50,000
22	Backfill	16,300	c.y.	1.50	24,450
23	Coal tar epoxy coating on Z-27 sheet piling	34,500	s.f.	0.40	13,800
24	Treated timber piles	5,000	l.f.	2.75	13,750
25	Test piling	Lump sum	job	50,000.00	50,000
	Subtotal				$2,025,384
	15% contingencies				299,816
	Total construction cost				$2,325,200
	7.8% E&D				181,400
	Subtotal				$2,506,600
	6.4% S&A				160,400
	Total cost				$2,667,000

TABLE IX-3 (cont'd)

Lands and damages

Item	Quantity	Unit	Unit price	Total price
Floodwall R/W	4.11	acres	$40,000	$164,400
Total				$164,400
Improvements				None
Severance				None
Total land value				$164,400
Contingencies, 10%				16,500
Real estate hired labor cost (3 tracts)				75
Acquisition cost by others (3 tracts)				525
Total cost				$181,500

Relocations

Item	Description	Estimated quantity	Unit	Unit price	Estimated amount
1	Florida Ave. wharf				
a.	Remove concrete ramps (8" thick)	1,470	s.f.	$ 1.00	$ 1,470
b.	Replace ramp				
	Concrete	18	c.y.	30.00	540
	Cement	25	bbl.	5.00	125
	Reinforcing steel	1,800	lb.	0.15	270
	Subtotal				$ 2,405
2	Chase Bag Company				
a.	Break out 8" concrete pavement and roadway	6,790	s.f.	1.00	6,790
b.	Replace pavement and roadway				
	Concrete	168	c.y.	30.00	5,040
	Cement	232	bbl.	5.00	1,160
	Reinforcing steel	16,800	lb.	0.15	2,520
	Subtotal				$15,510

TABLE IX-3 (cont'd)

Relocations (cont'd)

Item	Description	Estimated quantity	Unit	Unit price	Estimated amount
3	Lone Star Cement Company				
a.	Remove conc.block bldg.	625	s.f.	$ 3.00	$ 1,875
b.	Rebuild conc.block bldg.	625	s.f.	15.00	9,375
c.	Remove buried concrete foundation	75	c.y.	50.00	3,750
d.	Remove and replace 24" belt conveyor and housing	15	l.f.	270.00	4,050
e.	Cut and splice bulkhead tie rods	Lump sum	L.S.	27,050.00	27,050
	Subtotal				$46,100
4	J & L Steel Company				
a.	Remove roof	13,650	s.f.	0.50	6,825
b.	Replace roof and new roofing	14,500	s.f.	1.00	14,500
c.	Remove exterior wall	500	s.f.	2.00	1,000
d.	Replace wall	500	s.f.	4.00	2,000
	Subtotal				$24,325
5	Galvez Street Wharf				
a.	Break out 8" thick concrete ramp	400	s.f.	1.00	400
b.	Replace ramp				
	Concrete	10	c.y.	30.00	300
	Cement	14	bbl.	5.00	70
	Reinforcing steel	1,000	lb.	0.15	150
	Subtotal				$ 920
6	New Orleans Public Belt Railroad				
a.	Relocate tracks	Lump sum	L.S.	70,000.00	70,000
b.	Additional property	0.06	ac.	50,000.00	3,000
c.	Building alterations	1	job	8,000.00	8,000
	Subtotal				$81,000

TABLE IX-3 (cont'd)

Relocations (cont'd)

Item	Description	Estimated quantity	Unit	Unit price	Estimated amount
7	Interior drainage rectification work between floodwall and Galvez St. wharf (drain lines, manholes, catch basins, and pumping plant)		L.S.	70,000.00	$ 70,000
	Subtotal				$ 70,000
8	54" sewer force main	1	ea.	5,000.00	5,000
9	48" water main	1	ea.	5,000.00	5,000
10	8" water line	3	ea.	1,000.00	3,000
11	Telephone cable	1	ea.	200.00	200
12	12" water main crossing	1	ea.	1,200.00	1,200
13	12" water main relocation	700	l.f.	12.00	8,400
14	6" pipe containing telephone cable	1	ea.	500.00	500
15	16" H.P. gas line	1	ea.	2,000.00	2,000
16	42" concrete sewer force main relocation	400	l.f.	70.00	28,000
17	6" water line	2	ea.	900.00	1,800
18	3" pipeline (water air, diesel)	5	ea.	300.00	1,500
19	2" water line	1	ea.	200.00	200
	Subtotal				$ 56,800
	Subtotal relocations				$297,060
	15% contingencies				44,340
	Subtotal				$341,400
	7.8% E&D				26,600
	Subtotal				$368,000
	6.4% S&A				23,500
	Total relocations				$391,500

2. Comparison of cost estimates. The estimate of $3,240,000 is an increase of $1,710,000 over the corresponding cost included in the PB-3 dated July 1966. Inasmuch as the cost for the portion of the floodwall from the lock to Florida Avenue is not shown as a separate cost in the project document or the PB-3 dated July 1966, it was necessary to compute this cost by proportioning the length of the floodwall in question to the total length of the floodwall along the west side of the IHNC and applying the resulting ratio to the total cost shown in the project document and the PB-3 dated July 1966. Comparisons of the project document, PB-3 and Supplement No. 1 estimates, are shown on table IX-4. The reasons for the increase are as follows:

a. Levees and floodwalls. The increase of $1,207,800 reflects the added cost for constructing the protective works to a higher net grade resulting from hydraulic studies utilizing more severe parameters for the Standard Project Hurricane which were furnished by the U. S. Weather Bureau subsequent to project authorization; an increase in average floodwall height above natural ground of about one foot resulting from releveling in the area by the U. S. Coast and Geodetic Survey which in 1965 disclosed that ground surfaces in the project area were about one foot lower than they were considered to be when the cost estimates appearing in the project document were made; for replacing the "sheet piling wall with concrete cap" provided in the survey report with inverted "T"-type floodwall or "I"-type floodwall, as appropriate; and for increase in price levels.

b. Engineering and design. The increase of $94,200 reflects the added E&D on the increased construction cost.

c. Supervision and administration. The increase of $83,300 reflects the added S&A on the increased construction cost.

d. Lands and damages. The decrease of $66,800 reflects a reduction in rights-of-way area for the revised alignment as compared with the alignment presented in the project document.

e. Relocations. The increase of $391,500 is the total relocations cost for the protective works covered in this supplement. The project document plan did not recognize the need for any relocations on the IHNC.

TABLE IX-4

Comparison of Estimates
(IHNC west levee, Florida Ave. to IHNC lock)

		Project document	PB-3 appvd. 2 Jun 66 effective 1 Jul 66	Supplement	Difference--Supplement and PB-3
11	Levees and floodwalls	$ 937,600	$1,117,400	$2,325,200	+$1,207,800
30	Engineering and design	37,500	87,200	181,400	+94,200
31	Supervision and administration	56,300	77,100	160,400	+83,300
	Lands and damages	198,600	248,300	181,500	-66,800
	Relocations	0	0	391,500	+391,500
	Total	$1,230,000	$1,530,000	$3,240,000	+$1,710,000

TABLE IX-5

Schedule for Design and Construction

Contracts	Design*		Construction		Estimated construction cost (includes contingencies)
	Start	Complete	Start	Complete	
Floodwall and associated relocations	1966	May 67	Jun 67**	Oct 67**	$2,528,700***
Pile tests	1966	Mar 67	Apr 67	Jun 67	57,500****

*Includes advance supplement and plans and specifications.

**Dates are based on construction by local interests. Because sufficient Federal funds are not available in FY 1967 for starting construction on the reach presented in this supplement, all construction will be accomplished by Orleans Levee District, using non-Federal funds, as a means of expediting completion of the work.

***This cost includes $261,000 for relocations that are the responsibility of local interests, accomplishment of which, however, under a separate contract, is not practicable.

****Pile testing will be accomplished with Federal funds available from funds appropriated for FY 1967.

SECTION X - OPERATION AND MAINTENANCE

General. As specified in the authorizing act, local interests are to maintain and operate all completed works in accordance with regulations prescribed by the Secretary of the Army (except the navigation lock at the Rigolets, for which operation and maintenance they will provide a cash contribution equal to the capitalized value of the estimated annual cost of operation and maintenance of the lock). Maintenance of the levees and floodwalls covered in this supplement is estimated to cost $1,100 annually. Operation and maintenance of the 15 gap closures and the four hinged flaps in the Jones and Laughlin Company building is estimated to cost a total of $3,820 annually; further, it is estimated that replacement of these gates and flaps will be necessary at 30-year intervals. The annual charge for these replacements is $3,830. The total estimated annual cost to local interests for operation and maintenance and replacement of the features covered in this supplement is, accordingly, $8,750.

SECTION XI - RECOMMENDATIONS

Recommendations. The plan of protection presented herein for the protective works on the west bank of the IHNC between Florida Avenue and the IHNC lock, except for the reach in the vicinity of the Lone Star Cement Company rock storage bin, which is to be covered in a subsequent supplement, consists basically of I-type and inverted T-type floodwall. Eleven overhead roller gates and three swing gates are provided at vehicular and railroad crossings to preserve access during nonhurricane periods and permit rapid closure when hurricanes impend. In the Jones and Laughlin Steel Company warehouse, the concrete floodwall will be topped by a hinged flap gate, which, in its lowered position, will permit normal loading operations. The plan is considered to be the optimum one for accomplishing the project purposes, and is, accordingly, recommended for approval.

LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN

DESIGN MEMORANDUM NO. 2, GENERAL
ADVANCE SUPPLEMENT
INNER HARBOR NAVIGATION CANAL WEST LEVEE
FLORIDA AVENUE TO IHNC LOCK

APPENDIX I

CORRESPONDENCE

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FOOT OF PRYTANIA STREET
NEW ORLEANS, LOUISIANA

ADDRESS REPLY TO:
DISTRICT ENGINEER
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
P. O. BOX 60267
ORLEANS, LA. 70160

REFER TO FILE

LMNED-PP 7 October 1965

SUBJECT: Outline of Proposed Planning Procedures for Proposed "Lake Pontchartrain, La. and Vicinity," Project

TO: Division Engineer
U. S. Army Engineer Division
Lower Mississippi Valley
ATTN: LMVED-TD

1. Reference is made to the following:

a. LMNED letter "Review of Possible Engineering and Design Overload" dated 17 August 1965, and 1st Indorsement thereto.

b. Record of telecon between Messrs. Dement, LMVD, and Chatry, NOD, dated 15 September 1965, relative to subject project.

c. DIVR 1110-2-9.

2. The occurrence of hurricane "Betsy" has exerted a distinct influence on the course that should be followed in initiating planning of the subject project. First, it has introduced a requirement for increased tidal hydraulics coverage in the design process; second, it has generated substantial pressure for so arranging the planning that construction may be initiated at the earliest practicable date; and finally, it has preempted, for other purposes, the services of District engineering personnel required for participation in the overall planning effort.

3. We propose to respond to the peculiar requirements imposed by the above-described conditions by utilizing the following planning procedure. In the descriptions, please refer to inclosed map (incl 1):

a. A design memorandum (No. 1) on tidal hydraulics will be prepared in-house with maximum use of overtime when effective. Based on the project being funded on or before 15 October 1965, this memorandum would be forwarded for approval in January 1966. This submission date presupposes that studies now being made by the U. S. Weather Bureau will not result in a change in any of the parameters of the design hurricane. The scheduling of other design memoranda also is influenced by this presupposition.

b. Preparation of a general design memorandum (No. 2) on the barrier complex; i.e., the system of levees and structures required to exclude storm tides from Lake Pontchartrain, will be initiated concurrently with the memorandum on tidal hydraulics. Preparation of this memorandum would be by an A-E contractor with a local office. This memorandum will involve extensive coordination with various local entities, and for this reason is not considered suitable for accomplishment by another Corps office. Work would continue into fiscal year 1967 with submission date estimated to be March 1967, assuming initial funds are available by 15 October 1965.

c. Preparation of a general design memorandum (No. 3) on the Chalmette area also will be initiated concurrently with the memorandum on tidal hydraulics, utilizing an A-E contractor with a local office. For the reason cited in subparagraph b. above, this memorandum is not considered suitable for accomplishment by another Corps office. Work would continue into fiscal year 1967 and the memorandum would be submitted in November 1966, assuming initial funds are available by 15 October 1965.

d. In order to permit the earliest practicable start of construction, a single memorandum (No. 2A) supplementary to both the above GDM's, covering all levees along the Inner Harbor Navigation Canal, will be prepared in-house and submitted in advance of the GDM's. The existing Inner Harbor Navigation Canal levees proved to be very vulnerable during "Betsy." Further, the existing levee system, which will, in effect, provide the base for the project improvements, is under the exclusive control of the Orleans Levee Board, which agency is most anxious to cooperate. In addition, the entire project levee along the west bank of the canal, and that part of the project levee on the east bank of the canal which is north of the Gulf Intracoastal Waterway, will be integral parts of the barrier system, which system will produce more widespread benefits than any other project feature. Assuming that funds are available by 15 October 1965 and maximum use of overtime when effective, the advance supplement would be submitted in May 1966. With normal review time and allowing eight months, after submission of the advance supplement, for preparation of plans and specifications, review, advertisement, etc., construction could be initiated by January 1967.

e. A combination general and detailed design memorandum (No. 3 of the Mississippi River-Gulf Outlet series) for the Seabrook Lock will be prepared by another Corps office or by A-E contractor, using Mississippi River-Gulf Outlet funds. Assuming that funds are available by 15 October 1965, this memorandum would be submitted in July 1966.

LMNED-PP 7 October 1965

SUBJECT: Outline of Proposed Planning Procedures for Proposed "Lake Pontchartrain, La. and Vicinity," Project

4. CPM schedules and estimated planning and construction costs (including E&D and S&A) for the features described above are shown on inclosure 3. The funds required for fiscal year 1966, assuming A-E accomplishment of the barrier and Chalmette general design memoranda, exclusive of the $180,000 of Mississippi River-Gulf Outlet funds required for Seabrook Lock (preparation by A-E), are indicated to be in excess of $450,000 which is the amount expected to be made available. A request for additional funds will, however, be deferred until negotiations with A-E contractors are complete and a more positive requirement for additional funds exists.

5. Reference a. (1st Indorsement) indicated that our request for engineering assistance should be deferred until receipt of definite information that initial funds will be made available. We consider that receipt of initial funds in the amount of $450,000 for the subject project is, for all practical purposes, now assured. Accordingly, it is requested that we be authorized to proceed with arrangements to have the design memoranda for the barrier and the Chalmette area prepared by an A-E contractor. It is further requested that you arrange for preparation of the design memorandum on Seabrook Lock by another Corps office, or that we be authorized to arrange for its preparation by an A-E contractor.

6. Twelve copies of plates 3 and 9 from the survey report on the project are furnished herewith for use in briefing other Corps offices on the Seabrook Lock. Additional copies will be made available on request.

7. Approval of the procedure outlined in paragraphs 3-5 is requested. Further information on planning subsequent to that described will be the subject of future correspondence.

Thomas J. Bowen

THOMAS J. BOWEN
Colonel, CE
District Engineer

4 Incl
1. Map H-2-22077, plate 3 (12 cys)
2. Map H-2-22077, plate 9 (12 cys)
3. CPM - 1 sheet (12 cys)
4. CPM - 8 sheets (trip)

LMVED-TD (7 Oct 65) 1st Ind
SUBJECT: Outline of Proposed Planning Procedures for Proposed "Lake Pontchartrain, La. and Vicinity," Project

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39181 9 Dec 65

TO: District Engineer, New Orleans District, ATTN: LMNED-PP

1. Returned for terminal filing.

2. Action on basic letter was correlated with that taken on your letter, LMVED-PP, 5 November 1965, subject: Revised Outline of Planning Procedures for "Lake Pontchartrain, La. and Vicinity," Project, by LMVED-TD 1st indorsement dated 8 December 1965.

FOR THE DIVISION ENGINEER:

4 Incl
wd 1 cy ea

A. J. DAVIS
Chief, Engineering Division

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

IN REPLY REFER TO
LMNED-PP

5 November 1965

SUBJECT: Revised Outline of Planning Procedures for "Lake Pontchartrain, La. and Vicinity," Project

TO: Division Engineer
Lower Mississippi Valley Division
ATTN: LMVED

1. Reference is made to letter LMNED-PP dated 7 October 1965 subject "Outline of Planning Procedures for Proposed 'Lake Pontchartrain, La. and Vicinity,' Project."

2. Continuing consideration of the subject planning procedures reveals that certain revisions in the procedures outlined in the referenced letter are desirable. A discussion of proposed procedural changes follows in subsequent paragraphs.

3. It is understood that your office is opposed to the combined general and detail memorandum on Seabrook Lock. Accordingly, both an abbreviated general design memorandum establishing the general features of the lock and its precise location and a detail design memorandum will be prepared. Preliminary discussions have already been held with the Buffalo District and WES, and it has been determined that both memoranda will be prepared by Buffalo with assistance from WES on soils, foundations, and geology. Buffalo and WES have agreed to furnish estimates of time and cost for preparation of the two memoranda in the near future. We shall schedule the memoranda after receipt of the above data.

4. In order to reduce the time required to begin construction of elements covered in the general design memorandum for the barrier (see par. 3.b. of referenced letter), we now propose to prepare a general design memorandum for the entire Lake Pontchartrain barrier plan, with full design memorandum scope coverage limited to the two barrier structure complexes and a section of the Citrus back levee extending from the Inner Harbor Navigation Canal to near Michoud. The remainder of the plan would be given only brief coverage using survey report data with cost estimates and benefits updated. Segments of the plan given brief coverage in the general design memorandum will be developed further in a series of supplements.

LMNED-PP 5 November 1965

SUBJECT: Revised Outline of Planning Procedures for "Lake Pontchartrain, La. and Vicinity," Project

5. Preparation of the above-mentioned general design memorandum and plans and specifications for the section of levee detailed therein would be by A-E contractor. A schedule for the work and government estimate of cost (incl 1 & 2) are inclosed.

6. We plan to leave unchanged our prior proposals on design memoranda coverage for the tidal hydraulics, Inner Harbor Navigation Canal levee, and the Chalmette area. The schedules previously furnished for these memoranda are obsolete as to date and will be resubmitted.

7. A government cost estimate for the general design memorandum for the Chalmette area, which is also to be prepared by the A-E contractor, will be forwarded at an early date.

8. A list of proposed design memoranda covering the entire project is inclosed (incl 3).

9. Approval of the revised procedure discussed in paragraphs 3-7 is requested.

10. Approval of the government estimate of cost for the A-E contract for the general design memorandum on the Lake Pontchartrain barrier plan and authority to proceed with contract negotiations are requested.

Thomas J. Bowen

THOMAS J. BOWEN
Colonel, CE
District Engineer

3 Incl (dupe)
1. Schedule
2. Gov't est.
3. List of DM's

LMVED-TD (NOD 5 Nov 65) 1st Ind
SUBJECT: Revised Outline of Planning Procedures for "Lake Pontchartrain, La. and Vicinity," Project

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39181 8 Dec 65

TO: District Engineer, New Orleans District, ATTN: LMNED-PP

1. In addition to letter, LMNED-PP, NOD, 7 Oct 65, cited in par 1 of basic communication, the following correspondence is pertinent to the contents of this indorsement:

a. Letter, LMNED-PP, NOD, 19 Oct 65, subject: Lake Pontchartrain, Louisiana and Vicinity, Dual-Purpose Control Structure at Seabrook (Seabrook Lock), and 1st Ind, LMVED-PH/LMVED-TD, LMVD, 17 Nov 65, thereon.

b. Letter, ENGCW-EY, OCE, 12 Nov 65, subject: Engineering Assistance for the New Orleans District, and 1st Ind, LMVED-PH, LMVD, 23 Nov 65, to NOD.

c. Letter, LMVED-TV, LMVD, 24 Nov 65, to OCE, subject: Request for Engineering Assistance for the New Orleans District, 1st Ind, ENGCW-EZ, OCE, 29 Nov 65, thereon, and 2d Ind, LMVED-T, LMVD, 3 Dec 65.

2. This indorsement is in response to both basic communication and your letter of 7 Oct 65 referred to in par 1 hereof. Your proposed planning on this project provides for use of Architect-Engineer services for preparation of general design memoranda on the Barrier Complex and on the Chalmette Area, for the use of the Buffalo District for preparation of both the GDM and FDM on Seabrook Lock and for the in-house preparation of design memoranda on tidal hydraulics and the Inner Harbor Navigation Canal levees. Reference 1b above authorized the use of A-E services subject to the conditions stated in par 2 thereof. Transfer of the planning work on Seabrook Lock to the Buffalo District was authorized by 1st Ind, ENGCW-EZ, OCE, 29 Nov 65 (see reference 1c above, copy inclosed).

3. Revised planning procedure discussed in par 3 through 7 of basic letter is approved subject to the comments in par 5 below. Government estimate of cost for A-E contract (Incl 2) is approved. You are authorized to proceed with contract negotiations which should be conducted in accordance with par 2 of reference 1b above.

4. In reference 1a above, you were authorized to design Seabrook Lock on a controlling elevation of 7.2 ft msl. Recent review of the authorization contained in the Flood Control Act of 1965 has raised the question as to whether this modification in the controlling elevation is within the discretionary powers of the Chief of Engineers. In view of this uncertainty, it is desired that you proceed with definite project studies to a stage where a firm controlling elevation can be established

for the lock. Upon either confirmation of elevation 7.2 ft msl or the establishment of a new controlling elevation, it is further desired that a letter report be prepared covering this modification in the project. The letter report will be forwarded to the Chief of Engineers for approval pursuant to par 10, EM 1110-2-1150.

5. The following comments are furnished for consideration in firming up your planning schedules:

a. Your understanding in par 3 of basic letter that we are opposed to a combined GDM and FDM on Seabrook Lock is correct. Confirming par 3 of 1st Ind, LMVED-PH/LMVED-TD, LMVD, 17 Nov 65 (reference 1a above), separate general design and feature design memoranda are desired.

b. The schedule for preparation of GDM No. 2 provides very little slack time. Following the notice to proceed, the schedule requires a number of various field and design operations to proceed concurrently. This apparently will require a fair size staff of experienced engineers available to proceed with the work shortly after the notice to proceed is given. It may be difficult to find an A-E with this capability on short notice. The time scheduled may not make sufficient allowance for the various contingencies.

c. Normally, it is desirable to firm up the general requirements and types of structures to be built in the GDM. If the time scheduled does not permit a study of alternate types of structures, then the structures in the GDM could be based on previous structures of similar nature developed only in sufficient detail to serve as a basis for cost estimating. In this case the study of types of structures would either be covered in the FDM or in a letter report submitted for review prior to starting the FDM.

d. The furnishing of satisfactory assurances by local interests is prerequisite to construction. In view of the large non-Federal costs involved (a contribution of $19,021,000 for the Lake Pontchartrain Area

and $3,644,000 for the Chalmette Area, plus the costs of rights-of-way and relocations), it may be a distinct advantage in working out your planning schedule to ascertain from responsible local interests their attitude toward the project and their ability to provide the necessary cooperation. This action, if successful, may dictate changes in your schedule which would permit planning to progress in parallel with the activities of non-Federal interests. Under the present authorization, which includes the recommendation of the Secretary of the Army that the cost of Seabrook Lock be shared equally between hurricane protection and navigation, assurances from local interests must be obtained for the Lake Pontchartrain Area before construction of Seabrook Lock can be initiated.

FOR THE DIVISION ENGINEER:

George B. Davis
for A. J. DAVIS
Chief, Engineering Division

4 Incl
wd 1 cy ea incl 1-3
Added 1 incl
4. Ltr, LMVD, 24 Nov 65, w/1st & 2d Ind

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

IN REPLY REFER TO
LMNED-PP

8 November 1966

SUBJECT: Lake Pontchartrain, La. and Vicinity - Revised Approach to Advance Supplement on Inner Harbor Navigation Canal Levees

TO: Acting Division Engineer, Lower Mississippi Valley
ATTN: LMVED-TD

1. The advance supplement on the Inner Harbor Navigation Canal (IHNC) levees is presently scheduled for submission on 31 December 1966. As you know, this supplement is being prepared as a means of accelerating construction in an area proven to be critical by the passage of hurricane "Betsy" in September 1965. Preparation of the supplement has proceeded on the basis of having it cover all of the IHNC levees except those on the east bank between the IHNC lock and Florida Avenue, which segment is included in the GDM for the Chalmette area plan submitted to you under date of 1 November 1966 (see incl). As planning has progressed, it has become apparent that certain alignment and design problems would not permit coverage of some areas in sufficient detail for preparation of plans and specifications to follow directly from the advance supplement, and that additional design reports of a detailed nature would be required. Coverage in the advance supplement for such areas (which include the siphon crossings at Florida Avenue, all work on the west bank between the IHNC lock and Florida Avenue, and work on both banks in the vicinity of Interstate Highway 10 and U. S. Highway 90) was, accordingly, to have been limited to survey report scope, with detail design memoranda to follow as required.

2. On 24 October 1966, Mr. Armand Willoz, Chief Engineer of the Orleans Levee District, local cooperators for the project, expressed grave concern over the fact that current schedules would not result in Federal construction on the west bank of the Canal between the IHNC lock and Florida Avenue prior to the next hurricane season. Mr. Willoz explained that construction by the Orleans Levee District since "Betsy" has resulted in a significant increase in the degree of protection in all other areas which proved to be critical in "Betsy," and pointed out the technical factors which render impracticable an interim approach to providing protection such as has been applied in other areas. He also described the difficulties experienced and the disruption involved in providing emergency protection in the area by makeshift means during the hurricane season just past.

3. We pointed out to Mr. Willoz that, if our planning were revised to emphasize work in the subject area and all preconstruction planning completed in time to permit start of construction prior to next hurricane season, the fiscal outlook was such that construction funds in the amount required might not be available. Mr. Willoz indicated that the Levee District would be happy to undertake the construction with their own funds (subject to credit as in the case of the interim work already done by them), provided we gave them authority to do so. He further offered to have the necessary engineering done by architect-engineers if such action would expedite the production of approved plans.

4. In view of the above, we propose to modify our present planning schedules as follows:

a. The advance supplement will be modified so as to cover only that portion of the Canal between IHNC lock and Florida Avenue on the west bank. Coverage will be in sufficient detail to permit preparation of plans and specifications directly from the advance supplement.

b. Available in-house capability in the structural design area will be concentrated on preparation of the modified advance supplement. We expect that, under these conditions, the advance supplement can be submitted on or before 15 February 1967. Assuming normal review time, an approved set of plans and specifications for the work could be available by 15 June 1967.

c. Two additional supplements to the GDM for the barrier plan will be prepared for other segments of the IHNC levees. One would cover the Florida Avenue siphon crossings, and the other the remainder of the work on the Canal.

d. The emphasis placed on completing the advance supplement will result in slippage of the present schedule for the GDM for the barrier plan. However, this memorandum will cover in detail only the levee on the north bank of the Mississippi River-Gulf Outlet between the IHNC and Michoud. This levee has been raised to elevation 13 feet m.s.l. by the Levee District and currently affords a very high degree of protection. We plan to submit, at an early date, and prior to completion of the barrier plan GDM, a letter report on evaluation of alternate barrier locations, approval of which will permit site selection studies for the barrier structures to proceed without delay. Thus, progress on planning for these structures, which are crucial to the project, will not be delayed by slippage of the barrier plan GDM. We expect that this GDM, presently scheduled for submission on 31 January 1967, can be submitted on or before 1 September 1967. In view of the above, it is considered that this delay can be tolerated.

LMNED-PP 8 November 1966

SUBJECT: Lake Pontchartrain, La. and Vicinity - Revised Approach to Advance Supplement on Inner Harbor Navigation Canal Levees

5. Based on the foregoing, it is recommended that the advance supplement for the Inner Harbor Navigation Canal levees presently scheduled for submission on 31 December 1966 cover only the work between the IHNC lock and Florida Avenue on the west bank of the Canal, and that revised submission dates of 15 February 1967 and 1 September 1967, respectively, for this supplement and the GDM for the barrier plan, be approved.

Thomas J. Bowen

THOMAS J. BOWEN
Colonel, CE
District Engineer

1 Incl
Mosaic fwd sep

LMVED-TD (NOD 8 Nov 66) 1st Ind
SUBJECT: Lake Pontchartrain, La. and Vicinity - Revised Approach to Advance Supplement on Inner Harbor Navigation Canal Levees

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39180 18 Nov 66

TO: District Engineer, New Orleans District, ATTN: LMNED

1. Recommendation in para 5 of basic communication is approved.

2. Confirming telephone conversation between Messrs. A. J. Davis and George Hudson 17 Nov 66, tentative agreement has been reached with Chief, Engineering Division, OCE, for concurrent review of the design memorandum followed by a field conference about 10 days after receipt of DM in OCE. When preparation of the DM has progressed to a stage where a firm submission date can be established advise us promptly in order that arrangements may be made with OCE for the field conference.

3. In order to advance work on this project in an expeditious manner, it is desired that the design memorandum, when approved by OCE, be furnished the Orleans Levee District for preparation of contract drawings and specifications and for construction. Contract drawings and specifications prepared by the Orleans Levee District will be reviewed by NOD and submitted to LMVD for review and approval prior to advertising for bids.

4. In view of the reduction in work allowance for FY 67 from $1,600,000 to $850,000, the above procedure will encourage the use of local funds for the work between the IHNC lock and Florida Avenue and permit use of available Federal funds at other locations, thereby further advancing the project.

5. We suggest that all references to "supplement" be changed to "part" in identifying portions of GDM No. 2 for the Barrier Plan as was done with Design Memorandum No. 1, Tidal Hydraulics. All future design memoranda or parts thereof should contain a flyleaf map similar to those prepared as project maps for the purpose of showing the entire project and the relation of the work covered by a specific DM to the overall project. Once the map is prepared, it can be used in all DM's by delineating thereon the work covered by the DM being submitted.

FOR THE ACTING DIVISION ENGINEER:

A. J. Davis

wd all incl

A. J. DAVIS
Chief, Engineering Division

C O P Y

LMNED-DD 30 December 1966

SUBJECT: Lake Pontchartrain, La. and Vicinity - Inner Harbor Navigation Canal (IHNC) - Floodwall Conference

TO: Division Engineer, Lower Mississippi Valley
ATTN: LMVED-G & LMVED-T

1. Forwarded herewith for transmittal to OCE are Minutes of Floodwall Conference which was held in the New Orleans District on 13-14 December 1966. The floodwall on the west side of the IHNC from Florida Avenue to the IHNC Lock and the interim floodwall from Lake Pontchartrain to U. S. Highway 90 were discussed at this conference.

2. Inclosures 1 thru 3 to the Minutes of the Conference were forwarded to LMVD by letter LMNED-DD dated 23 December 1966, subject, "Lake Pontchartrain, La. and Vicinity, IHNC-Florida Avenue to IHNC Lock." Inclosure 4 to the Minutes of the Conference was forwarded to OCE by letter LMNED-PP dated 15 December 1966, subject, "Lake Pontchartrain, La. and Vicinity - Interim Floodwall Construction on the IHNC."

1 Incl (trip)
as

GEORGE H. HUDSON
Acting District Engineer

C O P Y

LMVED-TD (NOD 30 Dec 66) 1st Ind
SUBJECT: Lake Pontchartrain, La. and Vicinity - Inner Harbor Navigation Canal (IHNC) - Floodwall Conference

DA, Lower Miss. Valley Div, CE, Vicksburg, Miss. 39180 18 January 1967

TO: Chief of Engineers, ATTN: ENGCW-ES/ENGCW-ED

1. Minutes of conference in New Orleans District, 13-14 December, and forwarded for your review and approval. Approval is recommended, subject to the following comment.

2. Reference is made to para 4b. In regard to the second sentence of the second subparagraph, it was suggested that pile tests be performed at representative sites to determine the required pile lengths for T-type floodwalls.

FOR THE DIVISION ENGINEER:

1 Incl (dupe)
wd 1 cy

A. J. DAVIS
Chief, Engineering Division

LAKE PONTCHARTRAIN, LA. & VICINITY

RÉSUMÉ OF MEETING

13-14 DECEMBER 1966

NEW ORLEANS DISTRICT, CORPS OF ENGINEERS

SUBJECT: Hurricane Flood Protection Along the Inner Harbor Navigation Canal (IHNC)

1. Present. The following conferees attended:

Mr. R. A. Barron	OCE	Soils Branch
Mr. H. W. Goodhue	OCE	Structural Branch
Mr. W. E. Best	LMVD	Chief, Design Branch
Mr. R. I. Kaufman	LMVD	Chief, Geology, Soils & Materials Branch
Mr. H. A. Huesmann	NOD	Chief, Foundation & Materials Branch
Mr. K. J. Cannon	NOD	Foundation & Materials Branch
Mr. F. M. Chatry	NOD	Projects Planning Branch
Mr. Rixby Hardy	NOD	Projects Planning Branch
Mr. R. J. Franklin	NOD	Chief, Design Branch
Mr. H. Ussing	NOD	Design Branch
Mr. J. S. Henderson	NOD	Design Branch

2. Purpose of Conference. The purpose of the conference was to review procedures for preparation of a part of GDM No. 2, Barrier Plan covering the floodwall on the west side of the IHNC from Florida Avenue to the IHNC Lock. Also discussed were LMVD comments of the design analysis and plans and specifications for interim floodwall construction on the west and east sides of the IHNC from Lake Pontchartrain to the Chef Menteur Highway (U. S. Hwy. 90). In 1st Indorsement to NOD letter 21 Oct 66, subject "Lake Pontchartrain, La. and Vicinity - Interim Floodwall Construction on the Inner Harbor Navigation Canal," LMVD requested that material covering the interim floodwall construction be furnished to OCE conferees and carried back to Washington with them for review and approval. After reviewing LMVD comments the material was forwarded to OCE as an inclosure to NOD letter dated 15 Dec 66.

3. Narrative. Mr. Franklin opened the meeting by indicating the need for expediting hurricane protection along the IHNC. Mr. Henderson displayed an aerial mosaic of the entire IHNC, pointing out the following:

a. The sections where the Orleans Levee District (OLD) has constructed interim hurricane protection.

(1) Steel sheet pile floodwall from Florida Avenue to the IHNC Lock on the east side of the IHNC.

(2) Steel sheet pile floodwall from the L&NRR bridge to within 1,000 feet of Florida Avenue.

(3) 1,500 feet of levee parallel to and 1,000 feet north of Florida Avenue.

(4) 1,000 feet of steel sheet pile paralleling the IHNC to Florida Avenue.

(5) Ramping France Road in 2 locations. (Construction to begin 21 Dec 66).

b. The sections where NOD proposes to construct interim protection floodwalls. (Same interim floodwall construction referred to in par. 2).

(1) Steel sheet pile floodwall from Lake Pontchartrain to U. S. Hwy. 90 on west side of IHNC.

(2) Steel sheet pile floodwall from Lake Pontchartrain to Dwyer Road on east side of IHNC.

c. The sections where the OLD has requested flood protection be constructed prior to the 1967 hurricane season sufficient to resist a hurricane of Betsy's intensity.

(1) From U. S. Hwy. 90 to the Mississippi River-Gulf Outlet (MR-GO) and from the MR-GO to the IHNC Lock on the east side of the IHNC.

(2) From U. S. Hwy. 90 to the IHNC Lock on the west side of the IHNC.

It was brought out that except for the section from Florida Avenue to the IHNC Lock on the west side, the OLD will complete all of the interim protection that will be practical to build before the 1967 hurricane season. The OLD has also indicated that it has sufficient funds to construct the remaining section. Mr. Chatry discussed the overall IHNC hurricane protection plan including commitments made to local interests regarding interim protection.

An aerial mosaic of the Florida Avenue crossing was displayed next. The alternate plans for crossing the drainage canal or siphon (which run parallel to Florida Avenue) were discussed briefly. Next an aerial mosaic of the section from Florida Avenue to the IHNC Lock on the west side of the IHNC was viewed. The floodwall alignment was pointed out including the location of road and railway gaps.

4. <u>IHNC - West Side - Florida Avenue to IHNC Lock</u>. Discussion of the alignment and type of floodwall as shown on drawings, File No. H-4-24062, ensued as follows:

a. Beginning at Florida Avenue, the protection will consist of an I-wall in a levee from Station 34+95 to Station 44+09 and will include a 20-foot road gap.

b. A bearing pile supported inverted T-type floodwall will be constructed between Station 44+09 and Station 49+66 and will include a 20-foot road gap and a 17-foot railroad gap. NOD will recommend the use of prestressed concrete piling in lieu of creosoted timber piling because the suppliers of the timber piling have indicated that they cannot furnish the piles in time to complete the floodwall before the 1967 hurricane season. Mr. Best concurred. Mr. Willoz, Chief Engineer of OLD, has stated that his district will purchase piles approved by the Corps of Engineers in advance of letting a construction contract. This appears to be the only way that a floodwall can be constructed in this section prior to the next hurricane season.

Testing of bearing piles was discussed next. Mr. Kaufman suggested that piles be tested in areas where the soil strength or strata vary. Mr. Huesmann stated that piles will be tested in compression and tension at three (3) sites. Three (3) piles of different lengths will be tested at each site. It was decided to contact OLD regarding its capability to test the piles in accordance with NOD specifications.

The factors of safety to be applied to the values obtained from the test piles were recommended by Mr. Kaufman, as follows:

Compression	F.S.	=	1.75
Tension	F.S.	=	2.0

Messrs. Goodhue and Best stated that the allowable stresses listed in the Engineering Manuals for Hydraulic Structures should be used and no increase allowed for short term loading or wind forces.

c. From Station 49+66 to Station 56+31, an I-type wall will be built including two 20-foot road gaps and a 20-foot combined road and railroad gap. Mr. Kaufman recommended that the wall be checked for earth load and surcharge on the land side from Station 49+66 to Station 51+00. The last 364 feet of this section parallels a concrete sheet pile bulkhead at the Lone Star Cement Plant. Tie rods, 10-foot on centers, anchor the bulkhead to a deadman approximately 200 feet from the bulkhead. The tie rods will have to be cut to drive the sheet pile section of the I-wall. Mr. Goodhue indicated that a temporary tie back should be installed to hold the bulkhead before each tie rod is cut.

d. From Station 564+31 to Station 57+49, NOD recommended that a grout curtain be installed around the canal edge of the rock storage bin belonging to Lone Star Cement; the bin and the grout curtain would then form a floodwall. Messrs. Barron, Kaufman, Goodhue, and Best objected to this plan stating that since the storage bin walls were supported by vertical piles, no lateral resistance could be obtained if

the rock were removed from inside of the bin.

e. From Station 57+49 to Station 59+22, an I-type wall will be constructed including a 15-foot access gap. A 36-foot section of this wall passes between a chimney footing and the concrete bulkhead deadman at the Lone Star Cement Plant. MA-22 steel sheet piling will be used because of the limited clearance between the footing and the deadman.

f. From Station 59+22 to Station 65+20, an I-type wall with four 2-foot by 65-foot long steel flap gates and one 5½-foot by 78-foot long steel flap gate will be installed in the Jones & Laughlin Steel Plant, including a 17-foot railroad gap. The gates will be hinged at the top of the wall and will be raised and lowered with revolving "jib" cranes fastened to building columns.

Since the height of the I-wall in this section is approximately 10 feet above ground surface, the wall will deflect 2 inches or more when the maximum flood water stage is reached. Mr. Goodhue suggested that kicker piles be used to reduce the deflection of the wall. Messrs. Goodhue and Best recommended that a seal strip be located at the junction of I-wall and gate post or T-wall to allow for differential movement.

NOD will recommend that the roof of the Jones & Laughlin Steel Plant be removed in preference to splicing the sheet piling. This was agreed to by all conferees.

g. From Stations 65+20 to 69+68 and 71+48 to 88+40, a T-type wall (similar to subpar. "b" above) will be constructed including a 30-foot road gap. This section will parallel the Galvez Street Wharf and will be located along a line now occupied by a railroad track.

h. From Station 69+68 to Station 71+48, NOD planned to use prestressed concrete cylinder piles in place of steel sheet pile for the lower section of an I-wall for the following reasons:

(1) The height of the wall would exceed 10 feet preventing the use of a conventional I-wall.

(2) The batter piles of a T-wall would be driven under an existing building.

Messrs. Barron, Kaufman, Goodhue, and Best expressed doubts as to the structural integrity of this type of wall, stated that difficult driving problems would be encountered, and objected to the higher cost of the wall. Mr. Goodhue stated that this type of wall might not be capable of resisting lateral loads due to the very poor soil conditions. Mr. Best recommended that another type of wall be constructed. Mr. Kaufman suggested that a T-type wall be constructed and that damage to the adjacent building which occurs during construction of the wall be repaired.

i. From Station 88+40 to Station 89+35, a 17-foot railroad gap and a 20-foot road gap will be constructed.

j. From Station 89+35 to Station 90+65, NOD planned to construct a prestressed concrete cylinder pile supported I-wall along the south edge of the Galvez Street Wharf. Because of the objection (indicated in subpar. "h" above) to the use of this type of wall, a T-type floodwall will be investigated. The wall will be located far enough from the wharf to allow the required batter piles to be driven.

k. From Station 90+65 to Station 101+50, a T-type wall will be constructed similar to "b" above, including a 17-foot railroad gap and a 20-foot road gap.

l. From Station 101+50 to Station 105+66, a prestressed concrete cylinder pile supported I-wall was recommended, similar to the wall in subpar. "h" above. Since OCE and LMVD conferees disapproved of the type of wall, NOD will redesign the floodwall along the U. S. Coast Guard Station. The design will include the cost of relocating adjacent railroad tracks and removing, or modifying, the Coast Guard machine shop so that a T-wall can be constructed. Also an alignment west of the existing railroad tracks previously investigated will be reviewed.

m. From Station 105+06 to Station 106+90, a T-type wall will be constructed including a 20-foot roadway gate.

n. From Station 106+90 to Station 108+15, an I-type wall tieing into the levee adjacent to the IHNC Lock will be built.

o. The type of gates which NOD proposes to include in the GDM were discussed next. An overhead roller-type gate similar to the gates designed by the Louisville District for the Jeffersonville, Indiana, floodwall will be used where storage of the gate adjacent to the wall is permissible. Swing gates will be used where storage space is not available. Details of the gates proposed are shown on the last 2 drawings.

Mr. Goodhue recommended that provisions be made to adjust the swing gates should objectionable deflection or settlement occur. He and Mr. Best indicated that spread footings should be constructed across the base of the gates to prevent differential settlement.

p. Mr. Ussing suggested that ladder rungs be installed in case anyone should get trapped in the unprotected area after the gates are closed.

q. Mr. Henderson indicated that provisions would be made to install cathodic protection of the sheet pile cut-off under T-walls if tests indicate corrosion will take place. Cathodic protection will not be installed when the wall is constructed nor will the sheet piling be coated. A continuous reinforcing bar will be welded to the top of the sheet piling of I-walls so that cathodic protection can be added if required.

5. IHNC-West Side - Lake Pontchartrain to U. S. Hwy. 90 - IHNC - East Side - Lake Pontchartrain to Dwyer Road.

LMVD comments on inclosures furnished with letter, LMNED-PP, NOD, 21 October 1966, subject: "Lake Pontchartrain, La. and Vicinity - Interim Floodwall Construction on the Inner Harbor Navigation Canal" were discussed. It was decided to send changes of design analysis and revised plans and specifications to LMVD after OCE reviews design and LMVD comments.

6. Field Trip. Visiting conferees and members of the District Office spent the morning of Wednesday, 13 December 1966, on a field inspection of the section along the west side of the IHNC from Florida Avenue to the IHNC Lock.

4 Incl (previously furnished)
1. IHNC - Florida Avenue to IHNC Lock - Aerial Mosaic, File No. 9-17 (sheet 4)
2. IHNC - West Side - Florida Avenue to IHNC Lock, File No. H-4-24062 (14 sheets)
3. IHNC - West Side - Florida Avenue to IHNC Lock, Tabulation listing type of wall from station to station
4. IHNC - Interim Floodwall Construction - LMVED-TD 1st ind dtd 25 Nov 66 to LMNED-PP ltr dtd 21 Oct 66, subject, "Lake Pontchartrain, La. and Vicinity - Interim Floodwall Construction on the IHNC" with inclosures