U. S. ARMY CORPS OF ENGINEERS

NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE

Prepared by
U. S. Army Engineer District, New Orleans, New Orleans, La.
October 1970

3

LMVED-TD (NOD 28 Oct 70) 1st Ind
SUBJECT: New Orleans to Venice, Louisiana, Design Memorandum No. 2, Detail Design, Reach B1 - Tropical Bend to Fort Jackson, Empire Floodgate

DA, Lower Mississippi Valley Division, Corps of Engineers, Vicksburg, Miss. 39180 18 Dec 70

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

Subject design memorandum is forwarded for review and approval pursuant to ER 1110-2-1150. Approval is recommended subject to the inclosed comments.

FOR THE DIVISION ENGINEER:

A. J. Davis

A. J. DAVIS
Chief, Engineering Division

2 Incl (10 cy)
wd 6 cy incl 1
Added 1 incl (10 cy)
2. Comments

CF:
NOD-LMNED-PP w incl

18 December 1970

DEPARTMENT OF THE ARMY
LOWER MISSISSIPPI VALLEY DIVISION, CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI 39180

COMMENTS ON DESIGN MEMORANDUM NO. 2, DETAIL DESIGN, REACH B1 - TROPICAL BEND TO FORT JACKSON, EMPIRE FLOODGATE, INCLOSED WITH LETTER, LMNED-PP, 28 OCTOBER 1970, SUBJECT: NEW ORLEANS TO VENICE, LOUISIANA, DESIGN MEMORANDUM NO. 2, DETAIL DESIGN, REACH B1 - TROPICAL BEND TO FORT JACKSON, EMPIRE FLOODGATE

SECTION I

1. Para 4, Page I-2. This paragraph should state the requirements of local cooperation or make reference to GDM No. 1 containing such requirements.

2. Plate I-1. The Vicinity Map should show the project location with respect to a region of the State of Louisiana.

3. Plate I-4. a. Section B-B. The reason for providing the 6-ft wide strip of shell fill and riprap between the U-frame structure and the sill structure is not understood. Consideration should be given to replacement of the riprap with an extension of the sill structure. Also, a concrete protection slab or derrick stone should be provided on the flood side of the sill in order to prevent riprap from being washed into the gate recess.

b. Sections C-C and D-D. Driving of the concrete bearing piles and steel sheet piles through the shell fill shown in these sections would prove difficult if not impossible. Driving these piles subsequent to excavation but prior to shell backfill operations is considered satisfactory provided the shell fill is properly compacted.

c. Section D-D. Piping, erosion or blowout of the clay fill on the protected side of the I-wall, may occur due to the entrance of water through the riprap on the flood side during high stages. It is suggested that an impervious membrane or other similar impervious protection be provided beneath the riprap to aid in prevention of the above.

d. Section E-E. The 12-in riprap shown on the breakwater side slopes will not be sufficient to withstand hurricane wave action, thus requiring replacement maintenance subsequent to hurricanes. If not previously accomplished, it is suggested that the cost of provision of larger stone that will withstand hurricane wave action be considered and compared to projected maintenance costs for replacement of the proposed 12-in riprap subsequent to hurricanes. The most economical plan should then be selected.

e. The reason for providing erosion protection in the channel in excess of that proposed on page 27, para 37e(6) of GDM No. 1, should be stated. Gradation of riprap should conform to criteria given in EM 1110-2-1601, dated 1 July 1970.

SECTION III

4. Para 8, Pages III-2 and III-3, and Plates III-3 and III-4. a. For clarity this paragraph should include a discussion of the reason for considering the flood water surface at elevation 2.0 for the first and second stage excavations and elevation 5.0 for the third stage excavation.

b. We agree, in general, with the methods of analysis and basic principles used for pressure relief and dewatering computations on these plates and consider the results sufficiently accurate to provide the basis for the cost estimate for dewatering and piezometers presented in Section VIII. However, we consider that the specification for this work should be an end-result type, wherein the Contractor is responsible for the design and performance of the pressure relief and/or dewatering system. The contract should also provide for construction piezometers, as shown on the drawings. The specifications should clearly state any requirements for pressure relief and/or dewatering, during any of the three stages of excavation.

5. Paras 9 and 10, Pages III-3 through III-5, and Plate III-24. a. The design shear strength with depth should be represented as indicated in red on Plate III-24. This design shear strength line provides better agreement with all the available shear test data and was determined by use of a 10-pound per foot of depth shear strength increase. Further, this design shear strength can also be verified by use of a c/p ratio of 0.27 based on $\phi = 15°$, c = 0.0 tsf from the R triaxial shear test result.

b. The Q ($\phi = 0°$) design analyses for slopes (construction and final), cantilevered I-walls, T-walls, and bearing pile foundations discussed in these paragraphs and shown on the plates should be checked using the design shear strength discussed in subparagraph a above. Changes, if any, required by use of this design shear strength should be incorporated into the plans and specifications.

6. Para 10f, Page III-6. Reference markers should be placed on both sides of the monolith joints of the T-wall sections, in order to monitor movement of these walls.

7. Para 11, Page III-6. a. This paragraph should contain a statement that any unuseable or excess spoil will be disposed of in a manner to prevent degradation of the environment.

b. Consideration should be given to additional dressing, seeding, and landscaping.

8. Para 12, Pages III-6 and III-7. a. The construction sequence outlined in subparagraphs a, b, and c,does not agree with the sequence of construction steps 1 through 6 shown on Plate III-2. This discrepancy should be reconciled.

b. Subparagraphs c and d indicate that the first stage excavation is to be unwatered to el -6.0, while the base line section and step 6 of the sequence of construction shown on Plate III-2 indicate unwatering the first stage excavation to el -8.0. This discrepancy should be resolved.

c. It is not apparent how a 16-ft lowering of the water in the sand drains discussed in the last sentence of subparagraph f was determined. This 16-ft figure should be explained or corrected as applicable.

d. The last sentence of subparagraph g does not agree with data presented on Plate III-4, which indicates the piezometric head to be four feet below the bottom of the third stage excavation. This discrepancy should be reconciled. Maintaining the piezometric head four feet below the bottom of the third stage excavation is considered more satisfactory.

9. Plate III-1. The adequacy of the two lines of well points for dewatering should be investigated.

10. Plate III-10. The length of the temporary steel sheet pile wall should be checked and the proposed pile section indicated. The cost estimate should include an item for the sheet pile and the five-foot clay plug.

11. Plates III-5 and III-6. The stability analyses shown on these plates should be checked prior to preparation of the plans and specifications considering the SM stratum between els -13.0 and -15.0. This stratum was considered in all other stability analyses presented in the design memorandum.

12. Plates III-5 through III-9 and III-11 through III-16. Rough calculations indicate that crossover points were properly considered in the stability analyses: However, these have not been shown on the plates. The crossover points used should have been indicated on the plates.

13. Plates III-16 and III-17. The design penetration elevation for the cantilevered sheet pile floodwall is shown on Plate III-17 as -15.0. However, a required penetration to el -25.0 is indicated. Telecon with NOD F&M Br personnel indicated that the pile tip was extended to el -25.0 to force potential failure surfaces below this elevation. Apparently failure surfaces between el -15 and -25.0 will result in factors of safety lower than the required minimum of 1.30. The steel sheet piling should not be used to contribute to levee stability. The levee stability analyses should be checked neglecting the steel sheet piling. However, the design shear strengths discussed in para 5 above may be increased somewhat to account for gain in shear strength due to consolidation of the levee embankment for the analyses for the flood condition, i.e. water table at el -2.0 or +2.0 on the protected side and el 12.1 on the floodside. This is considered appropriate as the levee will, in all likelihood, be in place a period of time prior to the hurricane loading condition analyzed. The levee section should be designed for a factor of safety of 1.3 against sliding.

14. Plate III-21. Downward drag forces due to the weight of shell fill should be considered in the determination of the pile loads.

SECTION IV

15. Para 5e and f, Page IV-10. Include the criteria used to determine the allowable design stress for the timber piles. Combined stresses due to axial and lateral loads should be investigated for both concrete and timber piles.

16. Para 8c, Page IV-13. Furnish the base slab reinforcement design computations prior to submittal of the plans and specs.

17. Para 8c(1) and Plates IV-9 through IV-13. The slab thickness is not deep enough to assume that the applied loads from the wall would be equally distributed on the monolith base. Investigate the slab deflection pattern to see if the assumption of uniform distribution is appropriate. Also, ~~tensional~~ torsional stresses in the base slab should be investigated at a plane near the inside face of the side wall. telcon firm [illegible] 11/69

18. Para 11, Page IV-15. a. The vertical needles need to be secured to the girder for stability. This possibility should be investigated.

b. The proposed method of handling the needle beam girders and the concrete needle beams and provision for the storage of the needle beams should be discussed.

19. Para 13c, Page IV-16. The quantity of Z-32 sheet pile to be extended to elevation 15 is not stated. Plate IV-1 shows the top sheet pile to be elevation 9.0. It is not clear if the sheet pile is to be cut off and capped or capped without cut off. In either case, the additional pile should be included in the estimate.

20. Figures IV-32 thru IV-37, and Plate IV-5. a. No provisions have been made for adjustments to insure that equal portions of the gate weight and water forces will be taken by each hinge. Consideration should be given to the possibility of unequal loading.

b. The option of furnishing a weldment instead of a casting for the hinge assembly should be considered.

21. Plate IV-8. The two piles adjacent to each end of the gate outer sill monolith will interfere with the piles under the U-frame monolith. Also, the interior piles beneath the gate outer sill monolith should be located in line with the vertical piles in the U-frame structure to provide driving clearance.

22. Plate IV-9. The ability of the tension pile connectors to develop an ultimate 15 ton tension load is not substantiated. Supporting information should be given on the design of the anchorage.

SECTION V

23. Para 2h, Page V-1. Specify the type of material to be used for the hoist chains. Since the gate cathodic protection system will not protect the submerged

chain, consideration should be given to the use of corrosion resistant chain material or to cathodic protection for the normally submerged links.

24. Figure V-20. The effect of the eccentric loading should be considered in the computation of the concrete bearing stress.

SECTION VI

25. Para 10, Page VI-3. Type SIS wire (G.E. Vulkene, or equal) should also be permitted.

26. Plate VI-2. a. Since the torque-limiting coupling will be set to slip at a torque equivalent to approximately 5.5 horsepower motor output, a 5-hp motor may be adequate.

b. A single set of overcurrent relays (I-1 and I-3) may be adequate. It is assumed that the purpose of these relays is to stop the motors if the gate jams and the torque-limiting couplings fail to slip, or if the couplings slip excessively. The relays should have inverse, long-time tripping characteristics adjustable over a range of motor currents suitable for coordination with the calibration of the torque-limiting couplings.

c. Because of the long operating time, "seal-in" contacts should be provided in parallel with the "open" and "close" contacts. Stopping should be accomplished by providing a spring-return, push-to-stop contact on the control switch.

SECTION VIII

27. Page VIII-1. a. The estimated quantity of MA-22, sheet pile should be 8,200 sf in lieu of 7,700 sf if computations are based on dimensions shown on Plate IV-1.

b. Since local interests are required to furnish right-of-way, the cost thereof should be shown in the cost estimate as a non-Federal cost.

28. Refer to comments marked in red on pages i and v of the Table of Contents and pages III-3, IV-6, IV-7, IV-8, VI-1, VII-1, VII-2, Fig. IV-34, and Plates I-3, I-4, III-2 through III-5, III-10, III-22, III-24, IV-1, IV-2, and VI-2.

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

LMNED-PP

28 October 1970

SUBJECT: New Orleans to Venice, Louisiana, Design Memorandum No. 2, Detail Design, Reach B1 - Tropical Bend to Fort Jackson, Empire Floodgate

Division Engineer, Lower Mississippi Valley
ATTN: LMVED-TD

1. The subject design memorandum is submitted herewith for review in accordance with the provisions of ER 1110-2-1150 dated 19 June 1970.

2. Approval of the subject detail design memorandum is recommended.

1 Incl (16 cys)
DDM No. 2 fwd sep

Herbert R. Haar, Jr.
HERBERT R. HAAR, JR.
Colonel, CE
District Engineer

NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE

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APPENDIXES

PERTINENT DATA

Floodgate structure

"U" frame, reinforced concrete bay supported on timber piles

Gate

Bottom-hinged type

Guide walls

Timber

Dimensions	Feet
Width of floodgate (inside)	84
Length of guide walls	300
Length of fenders	100

Elevations	Feet, m.s.l.[1]
Top of floodgate structure	15.0
Gate sill	-17.5/-14.0
Operating floor of control house	24.0
Top of guide walls	9.5
Top of fenders	9.5

Hydraulic design criteria	Gulfside	Landside
Direct head from hurricane	12.1	2.0
Reverse head from hurricane	-2.0	6.3
Direct head for maintenance	5.0	-1.0
Reverse head for maintenance	-2.0	5.0

Floodgate cost $3,040,000

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

NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE

STATUS OF DESIGN MEMORANDUMS

Design Memo No.	Title	Status
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Reach B1 - Tropical Bend to Fort Jackson	Approved Aug 67
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design, Reach B1 - Tropical Bend to Fort Jackson, Supplement No. 1, Alteration of Method of Constructing Stream Closures	Approved Dec 68
2	New Orleans to Venice, La., Design Memorandum No. 2, Detail Design, Reach B1 - Tropical Bend to Fort Jackson, Empire Floodgate	Submitted Oct 70
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Supplement No. 2, Reach C - Phoenix to Bohemia	Scheduled Dec 70
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Reach B1 - Tropical Bend to Fort Jackson - Supplement No. 3	Scheduled Feb 71
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Supplement No. 4 - East Bank Barrier Levee Plan	Scheduled Jun 71
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Supplement No. 5, Reach B2 - Fort Jackson to Venice	Scheduled Aug 71
1	New Orleans to Venice, La., Design Memorandum No. 1 - General Design Supplement No. 6, Reach A - City Price to Tropical Bend	Scheduled Sep 71

NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE

SECTION I - GENERAL

1. Project authorization.

a. Public Law 874-87th Congress, approved 23 October 1962, authorized the project "New Orleans to Venice, La.," to provide hurricane protection in accordance with the recommendations of the Chief of Engineers in his report entitled "Mississippi River Delta at and below New Orleans, La.," and contained in House Document No. 550, 87th Congress, 2d Session. The report of the Chief of Engineers submitted for transmittal to Congress the report of the Board of Engineers for Rivers and Harbors, accompanied by the reports of the District and Division Engineers. The Chief of Engineers in his report concurred in the recommendations of the Board of Engineers for Rivers and Harbors which are as follows:

> "...Accordingly, the Board recommends improvements along the Mississippi River below New Orleans, Louisiana, for prevention of hurricane tidal damages by increasing the heights of the existing back levees and modifying the existing drainage facilities where necessary in four separate reaches consisting of:
>
> "Reach A on the west bank for about 15 miles between City Price and Empire;
>
> "Reach B on the west bank for about 21 miles between Empire and Venice and with such modifications of the main levee as may be required;
>
> "Reach C on the east bank for about 16 miles between Phoenix and Bohemia; and
>
> "Reach E on the east bank for about 8 miles between Violet and Verret;
>
> "generally in accordance with the plans of the District Engineer and with such modifications thereof as in the discretion of the Chief of Engineers may be advisable,..."

b. Subsequent to project authorization, Reach B was divided into two independent reaches--Reach B1 - Tropical Bend to Fort Jackson and Reach B2 - Fort Jackson to Venice, and Reach E was incorporated into the enlarged Chalmette Area Plan, a feature of the Lake Pontchartrain, Louisiana and Vicinity project.

2. Purpose. This detail design memorandum presents the essential data, assumptions, and criteria used in the design of the principal features of the floodgate. It was prepared to facilitate the preparation of plans and specifications for construction of the floodgate and also to assist in the review of the construction plans and specifications.

3. Previous reports. For general information on the project and basic data, reference is made to the report entitled "New Orleans to Venice, Louisiana, Design Memorandum No. 1, General Design, Reach B1 - Tropical Bend to Fort Jackson."

4. Local cooperation. The Act of Assurances and supporting resolution from the Plaquemines Parish Commission Council were accepted for and on behalf of the United States on 14 April 1965.

5. Location. The floodgate will be located near Empire in the Empire to Gulf Waterway, as shown on plate I-1.

6. Datum plane. All elevations indicated are in feet and refer to mean sea level datum unless otherwise noted.

7. Description. The floodgate will consist of the following features:

a. Gate bay. Reinforced concrete U-frame gate bay with a clear opening of 84 feet and sill elevation of -14.0 with steel gate hinged at the bottom, all supported on untreated timber piling. The total structure width will be 106 feet and top of the walls will be at elevation 15.0. A control house will be provided above one wall for operation of the gate, and needle dams will be provided for unwatering the gate while the gate is in the closed position.

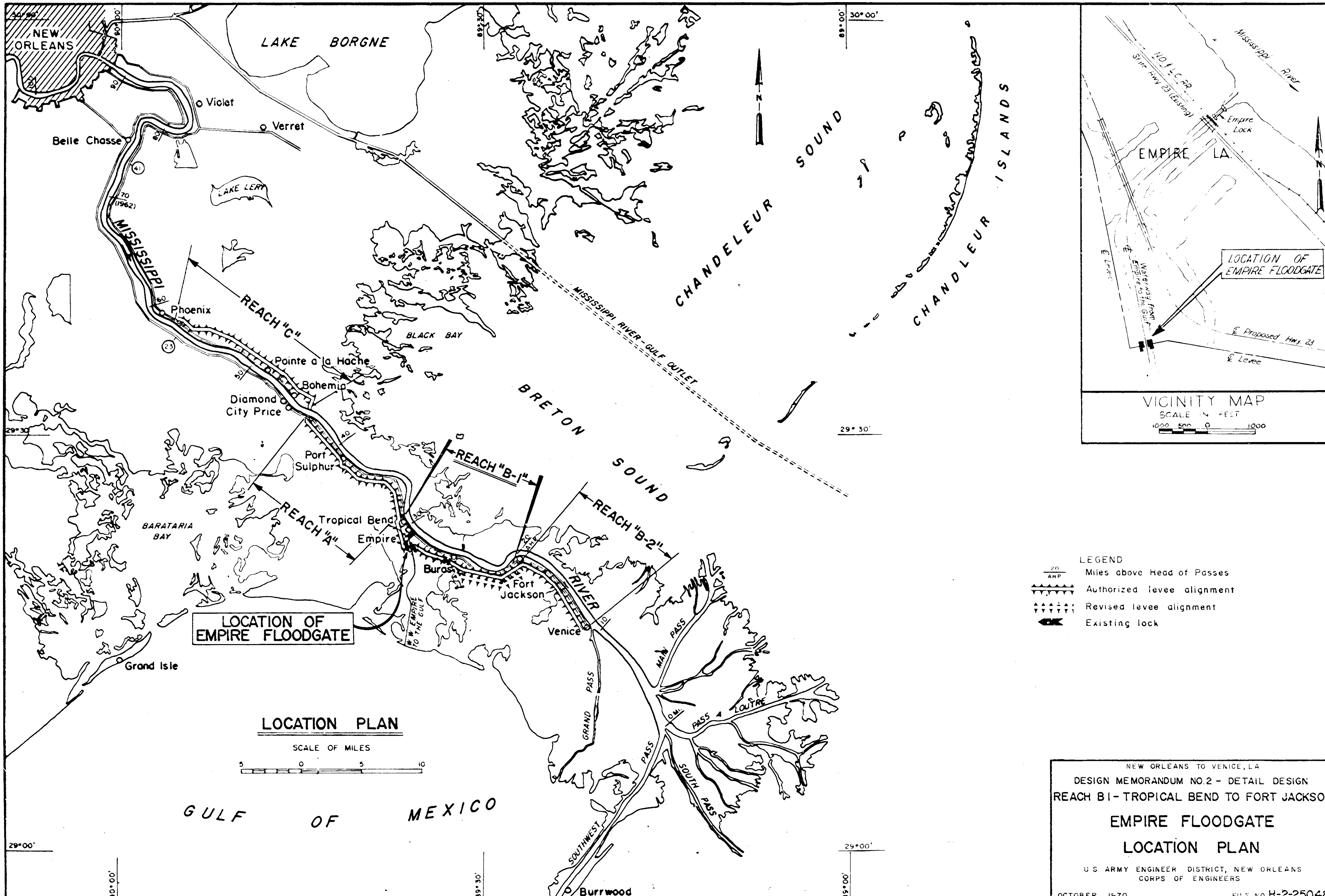
b. Timber guide walls and fenders. A 300-foot timber guide wall and a 100-foot long timber fender will be located on each side of the gate structure. The guide wall is on the west side of the channel and the fender is on the east side of the channel.

c. Floodwalls. An inverted T-type reinforced concrete floodwall will abut the structure wall and extend for a distance of 150 feet on each side of the structure, at which point I-type reinforced concrete floodwalls will extend an additional 105 feet on each side of the structure. The top of the floodwalls will be at elevation 15.0. A reinforced concrete walkway will be provided on the walls for access to the structure and control house.

d. Access road. An access road will be provided to the structure and will have a shell surface.

e. Breakwater. A breakwater with top elevation of 2.0 will be provided to the southwest of the structure.

8. Temporary bypass channel. During construction of the floodgate, navigation in and out of Empire will be provided by the existing bypass channel. The channel is located generally as shown on plate 3 of the report referenced in paragraph 3.

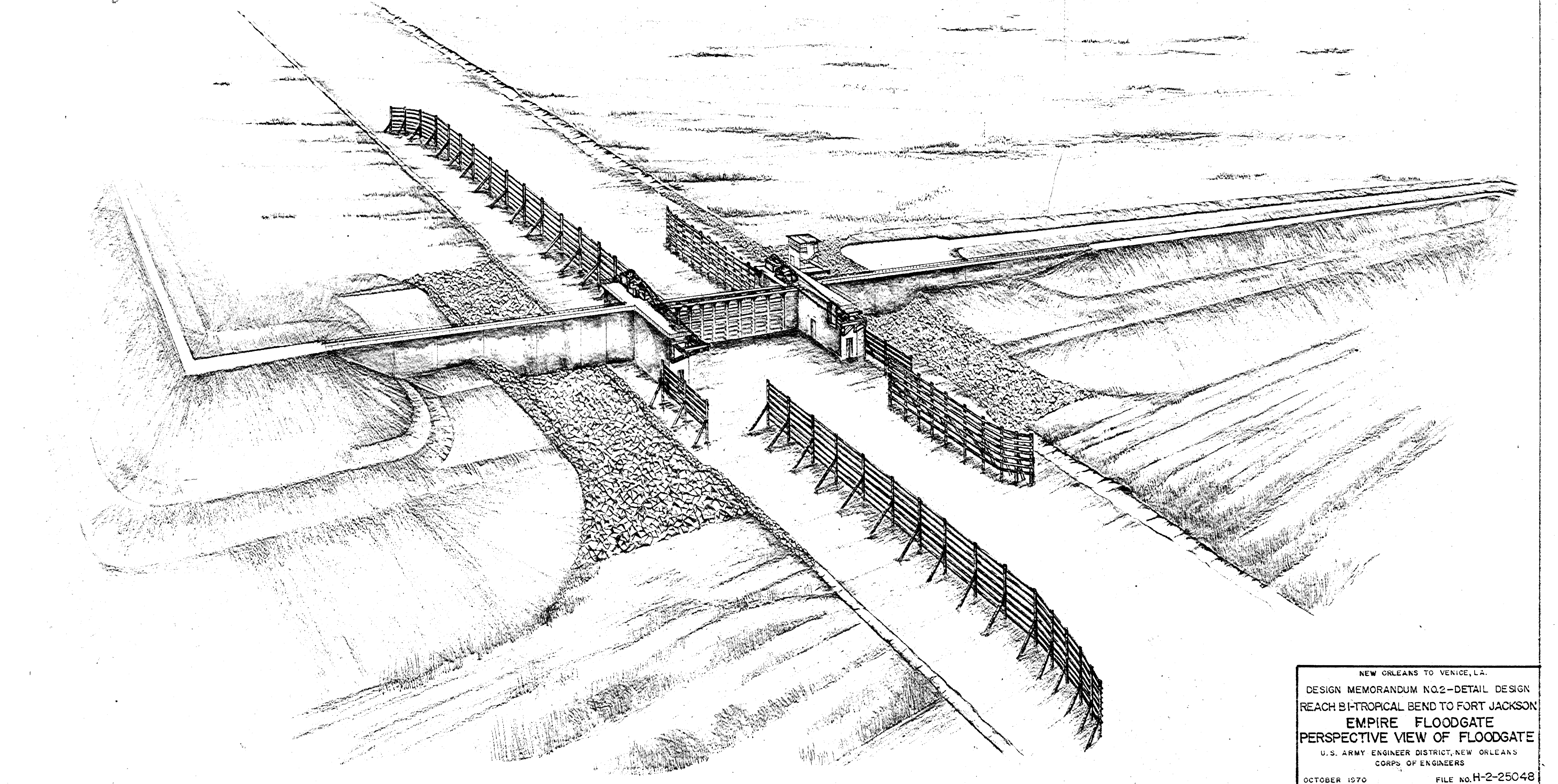


NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
LOCATION PLAN

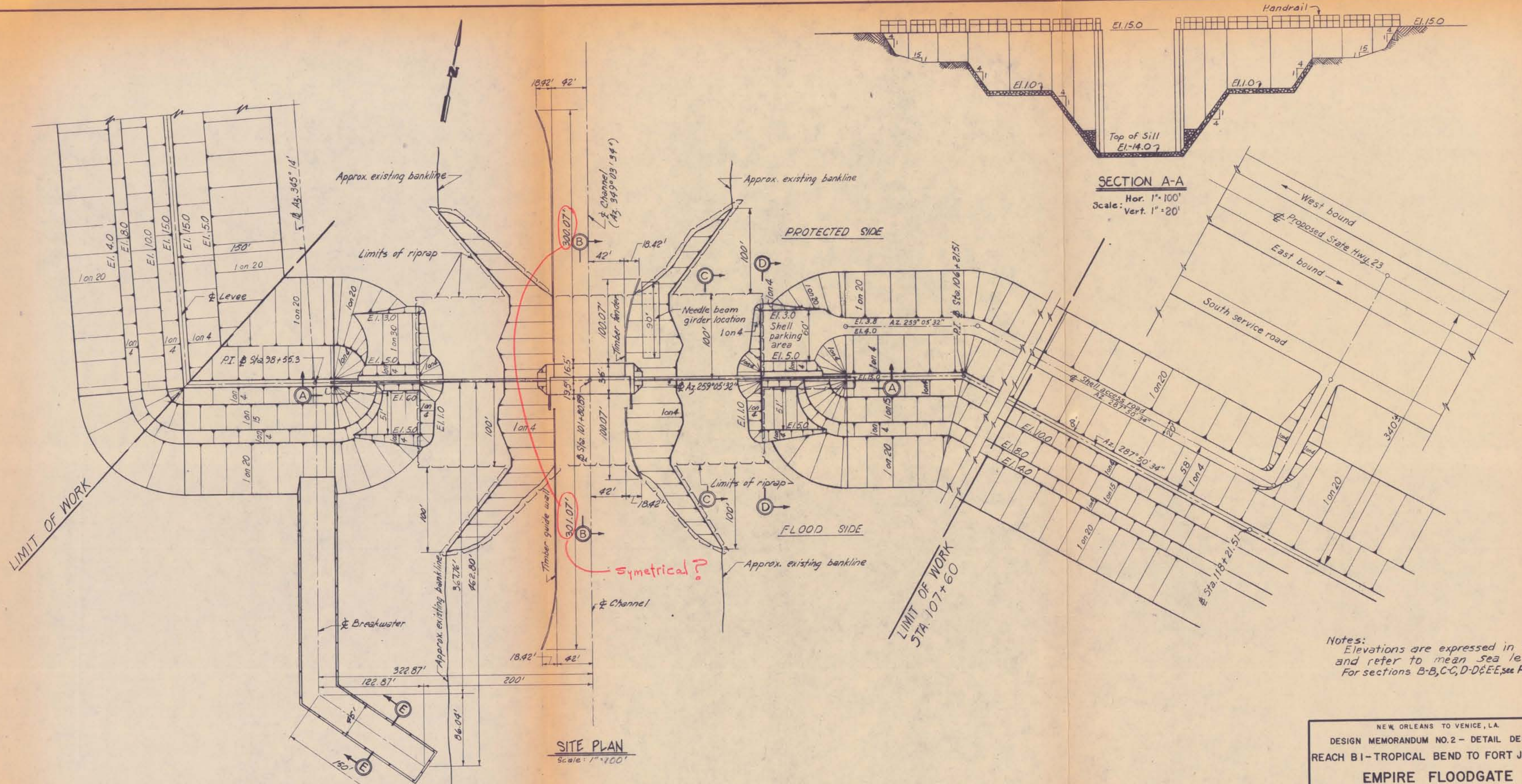
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO H-2-25048



NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO.2-DETAIL DESIGN
REACH B1-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
PERSPECTIVE VIEW OF FLOODGATE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048

PLATE I-2



SECTION A-A
Scale: Hor. 1" = 100'
Vert. 1" = 20'

SITE PLAN
Scale: 1" = 100'

Notes:
Elevations are expressed in feet and refer to mean sea level.
For sections B-B, C-C, D-D & E-E, see Plate I-4

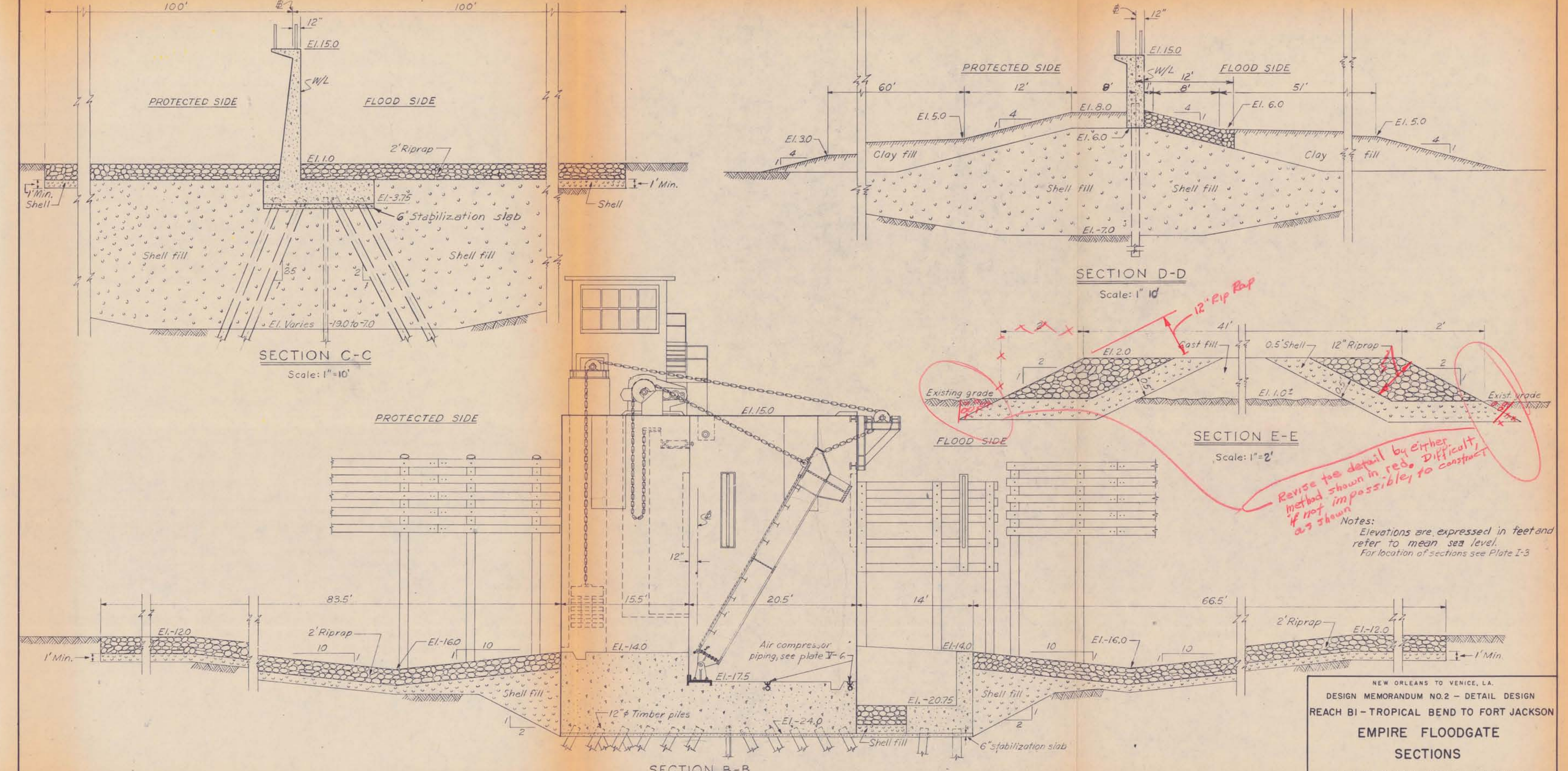
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

SITE PLAN

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

OCTOBER 1970 FILE NO. H-2-25048



Notes:
Elevations are expressed in feet and refer to mean sea level.
For location of sections see Plate I-3

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
SECTIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970
FILE NO. H-2-25048

PLATE I-4

SECTION II - HYDRAULIC ANALYSIS OF FLOODGATE

1. General.

a. The construction of the Empire Floodgate will intercept drainage from an area of about 365 acres. This area will be inclosed by the hurricane protection levee, the levee along the Mississippi River, and the levees essentially parallel to the Mississippi River levee which have been constructed to approximate elevation 8 by local interests. To meet the requirements of navigation, the floodgate will have an 84-foot width at the sill elevation of -14.

b. Since the floodgate, once closed, cannot be opened until hurricane tides have receded and the stage on the landside is equal to or higher than the stage on the gulfside, closure will be delayed until overall weather conditions are so severe that the arrival of water craft fleeing the scene of a hurricane is unlikely. However, closure of the floodgate will probably not be delayed after the ingress of hurricane tides have produced an elevation of 5.0 on the landside of the structure.

2. Hydraulic computations.

a. Discharges for the floodgate were computed using the equation, $Q = C_s Lh \sqrt{2g\Delta H}$, and data presented in the U. S. Army Engineer Waterways Experiment Station publication, Technical Report No. 2-655, "Spillways for Typical Low - Head Navigation Dam Arkansas River, Arkansas." The net length of spillway crest was determined by the use of Sheet 111-3/1, Hydraulic Design Criteria. Symbols used in the equation are defined as follows:

Q = Total discharge in c.f.s.
L = Net length of spillway crest in feet
h = Tailwater elevation referred to sill crest in feet
g = Acceleration due to gravity in ft./sec.^{2}
ΔH = Differential between total energy of the approach channel and depth of tailwater in reference to the crest in feet
C_s = Discharge coefficient for submerged-uncontrolled flow

For the case of a rising hurricane tide coincident with a landside elevation of 5.0, the maximum inflow into the area is $Q = 2.17 x 80.18 x 19.0 \sqrt{2 x 32.2 x .08}$ or 7,500 c.f.s. The curve presented on plate 41 of Technical Report 2-655 was used to obtain the C_s value of 2.17 with a h/H ratio of 0.996. Flood routing computations were not made in detail and, consequently, are not presented herein. Sufficient computations were made to estimate the magnitude of maximum stage differentials and average velocities and to insure that these velocities would not produce serious, adverse effects on navigation or on the riprap protection which will be provided at each end of the floodgate.

3. Synthetic inflow hydrograph. Runoff data for the area are not available. In order to estimate the magnitude of differential heads resulting from high intensity rainfalls over the drainage area, the inflow hydrograph for the storm with a duration of 24 hours and a frequency of 100 years was synthesized with the use of values contained in U. S. Weather Bureau Technical Paper No. 40, "Rainfall Frequency Atlas of the United States," published in 1961. This storm has a maximum hourly rainfall of 4.80 inches and a total of 14.5 inches. Based upon an infiltration rate of 0.10 inch per hour, the rainfall excess from this storm is 12.10 inches. Data pertinent to this storm are tabulated below:

At end of hour	Rainfall during hour-inches	Runoff during hour-inches	Inflow at end of hour-c.f.s.
0	0	0	0
1	0.84	0.74	272
2	1.80	1.70	626
3	4.80	4.70	1,730
4	1.00	0.90	331
5	0.68	0.58	213
6	0.58	0.48	177
7	0.48	0.38	140
8	0.47	0.37	136
9	0.45	0.35	129
10	0.40	0.30	110
11	0.37	0.27	99
12	0.33	0.23	85
13	0.30	0.20	74
14	0.25	0.15	55
15	0.24	0.14	51
16	0.22	0.12	44
17	0.21	0.11	40
18	0.17	0.07	26
19	0.17	0.07	26
20	0.17	0.07	26
21	0.15	0.05	18
22	0.14	0.04	15
23	0.14	0.04	15
24	0.14	0.04	15
25	0	0	0

4. Overall hydraulic capacity.

a. The area of 1,176 square feet below mean sea level, which is required to meet the requirements of navigation, is so large with respect to the drainage area that negligible head differentials will be experienced when the fully open floodgate is admitting inflows from hurricane tides to the area or is releasing runoff from high intensity storms occurring in conjunction with normal tides on the gulfside.

b. As the hurricane tide rises, the maximum flow through the floodgate will be 7,500 c.f.s. The maximum average velocity will be 4.7 f.p.s. under a differential head of less than 0.1 foot (see paragraph 2 above). The maximum inflow of 1,730 c.f.s. resulting from the 100-year 24-hour storm occurring in conjunction with the average elevation of 0.5 on the gulfside of the floodgate can be conveyed through the structure under a head of about 0.02 foot. For this flow, the maximum average velocity through the structure will be 1.4 f.p.s.

5. Maximum probable sump pool elevation and reverse heads.

a. After the floodgate is closed to prevent further ingress of water from rising hurricane tides, additional ponding above elevation 5 in the area between the hurricane protection levee and the existing levees will result from rainfall during the period of gate closure. It is unlikely that the floodgate will be closed for more than 72 hours. With some wave overtopping and with 100 percent runoff from the 25-year 3-day rainfall of 13 inches indicated by the data contained in U. S. Weather Bureau Technical Paper No. 49, "Two to ten-day Precipitation for Return Periods 2 to 100 Years in the Contiguous United States," ponding would occur to elevation 6.3 feet. With a sudden reversal of winds during the periods of gate closure, an elevation of -2 could be produced on the gulfside.

b. Although the reverse head of 8.3 feet must be considered in the structural design of the floodgate, it is not critical for the conditions in the channel on the gulfside of the floodgate which will normally prevail when the floodgate is reopened after a hurricane has subsided. Since gulfside stages generally recede more slowly after a hurricane has passed than they rise during the passage of a hurricane, maximum velocities for a complete and uninterrupted opening of the gate will normally be less than those experienced before closure during the approach of the hurricane.

c. If necessary, the floodgate can be opened gradually to slowly reduce the elevation on the landside. This landside stage can also be gradually lowered by the operation of the Empire Lock to convey impounded water to the Mississippi River. This lock, owned by the State of Louisiana, has a usable length of 200 feet and a sill at elevation -10 with a width of 40 feet.

SECTION III - FOUNDATION INVESTIGATION

1. General. An existing bypass channel will route traffic around the site where the floodgate is to be constructed. The floodgate will be located on the levee alignment and constructed within an unwatered area in the existing Empire Waterway. Floodwalls extending from each side of the floodgate and tieing into the levee will provide protection against the design hurricane (100 years).

2. Previous investigations. The design memorandum "New Orleans to Venice, La., Design Memorandum No. 1, General Design, Reach B1, Tropical Bend to Fort Jackson," dated March 1967, contains foundation investigation data for the recommended levee. These data include borings and the results of soil mechanic laboratory tests. The above data, where pertinent, were used in the design of the foundation features of the floodgate and walls.

3. Field exploration. One 5-inch diameter undisturbed boring No. 1-SEU and four general-type disturbed core borings Nos. 2-SE through 5-SE were made for this report and are located as shown on plate IV-1. The undisturbed and general-type borings extended in depth to approximate elevations -90 and -80, respectively. The general-type and undisturbed boring logs are shown on plates III-23 and III-24, respectively. Refer to plate A for the legend of symbols and information on the boring logs.

4. Laboratory tests. Visual classifications were made on all samples and water content determinations were made on all cohesive samples obtained from the borings. Unconfined compression (UC), unconsolidated undrained (Q), consolidated undrained (R), consolidated drained (S), and shear and consolidation (C) tests were performed on samples of representative soils from the undisturbed boring. The laboratory test data for the undisturbed samples are shown on plate III-25.

5. Foundation conditions. The subsurface at the project site is generally similar to that shown on the profile in the GDM. The foundation soils, as indicated by borings 1-SEU and 2-SE through 5-SE, consist predominantly of Recent backswamp clays having soft to medium consistencies, and extending to depths of approximately 90 feet below the natural ground surface. The Recent clays contain 3- to 5-foot thick layers of silts and sands at approximate elevations -20, -30, and -50. The 5- to 10-foot thick clay layer, extending from the ground surface, contains organic matter with some peat.

6. Design problems. The principal problems to be resolved in the foundation design were as follows:

a. The unwatering and hydrostatic pressure relief required to construct the floodgate and tie-in walls in the dry.

b. The structure excavation slopes and berm distances, temporary protection dike, stream closure sections, and berm distances required for stability during construction with a minimum of excavation, fill, and backfill quantities.

c. The final slopes required for stability to provide a minimum height and length of tie-in wall and a minimum of channel excavation quantities.

d. Bearing pile lengths and subgrade moduli data for the floodgate and T-wall.

e. The penetrations required for the steel sheet piling beneath the concrete I-walls, and the penetrations and unbalanced lateral waterloads acting on the steel sheet pile cutoffs beneath the floodgate and T-wall.

f. Spoil disposal, erosion protection, and provision for permanent engineering measurement devices.

g. The sequence of construction.

7. Temporary protection dike. The temporary dike will have a 10-foot crown at elevation 6.0 with 1 on 4 side slopes and will provide protection against high tides during construction of the floodgate. The dike will encircle the working area and tie into the existing 1st lift levee, as shown on plate III-1. The land portion of the dike will be constructed with material cast from the structure excavation area. The stream closure portions of the dike will be constructed with barged-in clamshell and clay cast from the structure excavation area and from side borrow, as shown on plate III-1. After the floodgate and tie-in walls are complete, the dike material in the vicinity of the I-wall will be used in the levee berm, and the shell in the stream closures will be used in the access road and parking areas.

8. Unwatering and hydrostatic pressure relief during construction. In order to build the floodgate and floodwalls in the dry and to insure stability of the structure excavation during construction, hydrostatic pressure relief will be provided in the silt and sand layers within the soil foundation. The pressure relief will be accomplished by vertical sand drains and well points as shown and described on plates III-2, III-3, and III-4. To allow time for pore pressure relief, the rate of unwatering of the working area will be maintained at a maximum of 2 feet per day for the first 10 feet, and 1 foot per day thereafter until completely unwatered. Temporary construction piezometers will be installed in the pervious

layers to monitor the pore pressure during the unwatering and pressure relief period. The design procedures and calculations for the sand drain and well point systems are shown on plates III-3 and III-4, respectively. After the structure is complete and operating, the sand drains will discharge into the shell backfill and provide a degree of permanent pressure relief. Conventional sumps and pumps will maintain the area free of surface water during construction.

9. Shear stability.

a. Construction slopes. The stability of the excavation, dike and closures, existing first lift levee, and berm distances were determined by the method of planes based on a minimum factor of safety of 1.3 with respect to shear strength and the (Q) design shear strengths as shown on plates III-5 through III-9. Stability was investigated at various depths in the foundation, and factors of safety with respect to shear strength were determined for various assumed failure planes. The water conditions, assigned foundation stratification, design shear strengths, critical failure surfaces, and their corresponding analyses are shown on plates III-2 through III-9. The relief facilities will provide the required pressure reduction in the pervious layers for stability. 5

b. Final slopes. The (Q) stability governed for design. The stability analyses are shown on plates III-11 through III-16. The final slopes will be constructed by clamshell backfilling. In the vicinity of the structure, the inclinations of the rebuilt slopes were determined by the requirement that the length of the floodwalls be as short as possible without sacrificing stability of the tie-in levee into the inlet and outlet channels. The remaining rebuilt slopes were designed to be stable with a minimum of backfilling.

c. Cantilevered I-wall. The results of tidal hydraulic analyses indicate that the I-wall will be subjected to the pressure and forces imparted by breaking waves. In the stability analyses, the dynamic wave effect was applied as a line force acting through the centroid of the dynamic wave pressure distribution diagram. The static water pressure diagram resulting from wave action was considered effective only to the top of the impervious clay, inasmuch as the period of time the wave will exist is too short to allow water pressure to become effective in the impervious clays. The stability and required penetration of the steel sheet piling below the fill surface were determined by the method of planes. The long-term (S) shear strengths (C=0) governed for design. A factor of safety of 1.25 was applied to the friction angle as follows: ϕ_d = developed friction angle = $\tan^{-1}\left(\frac{\tan \phi_A}{\text{Factor of Safety}}\right)$. This developed angle was used to determine K_A and K_P lateral earth pressure coefficient values as follows: $K_A = \tan^2\left(45° - \frac{\phi_d}{2}\right)$ and $K_P = \frac{1}{KA}$.

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 pressures, the summation of horizontal forces was equated to zero for various tip penetrations. At these penetrations, summations of overturning moments about the bottom of the sheet pile were determined. The depths of penetration required for stability were determined as those where the summation of moments was equal to zero. The analysis is shown on plate III-17.

10. Control structure and T-wall.

a. Steel sheet pile cutoff. A steel sheet pile cutoff will be used beneath the floodgate and T-walls to provide protection against hazardous seepage. The recommended tip elevations of the cutoffs beneath the floodgate and T-walls are shown on plates III-18 through III-20. The net pressure diagram along the sheet pile cutoff was determined as follows:

(1) Conventional stability analysis by the method of planes, utilizing a factor of safety of 1.3 incorporated in the soil strength parameters, was performed to determine the stability against rotational failure. The use of a factor of safety of 1.3 is also recommended by Mr. Gregory P. Tschebotarioff in Chapter 5 of "Foundation Engineering," edited by G. A. Leonards, and dated 1962. The analysis was performed at 1-foot intervals with the active wedge located at the flood side edge of the structure and the passive wedge located at the protected side edge of the structure.

(2) The assumption was made that the value of (R_B) at the bottom of the base of the structure was zero.

(3) For each analysis the net driving force, i.e., $(D_A - D_P) - (R_A + R_B + R_P)$ was determined. The value of D_A included the weight of water between the tailwater elevation and the SWL elevation located above the active wedge.

(4) The assumption was made that the net driving force above the bottom of the base of the structure was carried by the structure.

(5) Considering driving (D_A) positive and all resistance negative (D_P, R_P, R_B, and R_A) in the expression $D = D_A - D_P - R_P - R_B - R_A$, using the method of planes stability analyses, ΣD was determined by assuming failure at the bottom of the base of the structure and at each foot in depth thereafter. The value of the algebraic difference in ΣD, between 1-foot intervals, was used to develop the pressure diagram. If the incremental difference is negative, the pressure diagram indicates an available horizontal resistance in excess of that required, and if the incremental difference is positive, the pressure diagram indicates an unbalanced horizontal pressure in excess of the available soil resistance. It is considered that such an excess must be carried by the sheet pile cutoff.

(6) The net pressure diagrams presented on plates III-18 through III-20, indicate that the total available horizontal resistance is in excess of the total horizontal waterload. Therefore, the bearing piles are not required to carry any additional lateral load acting on the sheet pile cutoff.

b. Bearing pile foundations.

(1) The floodgate and T-walls will be supported by piling, battered as required, to provide stability against the unbalanced lateral waterloads. The inverted T-type floodwalls will be used in lieu of I-type floodwalls where the height of the I-wall above ground and the magnitude of the dynamic wave force render the I-type floodwall impracticable. In compression, a factor of safety of 1.75 was applied to the shear strengths, and a lateral earth pressure coefficient (K_o) = 1.0 was used for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths and a coefficient (K_o) = 0.7 was used. Pile design loads vs. tip elevations and subgrade moduli vs. tip elevations are shown on plates III-21 and III-22. Settlement of the piles due to consolidation will not be a problem since the major loads are caused by hurricane water heads of insufficient duration for consolidation of the foundation clays to ensue. See paragraph 13c, Section IV, page IV-16 for providing for recompression settlement of I-wall.

(2) During construction, three 12" diameter class B untreated timber piles of different lengths will be driven at the locations indicated on plate IV-8. The intermediate pile will be tested in compression. If test results show that the pile can safely carry twice the design load, the pile will be tested in tension. If the intermediate pile fails before the required capacity is attained in compression, the long pile will be tested in compression and in tension. If the intermediate pile safely carries compression loads significantly in excess of that required, the short pile will be tested in compression and in tension.

(3) The test site will be in the vicinity of boring 1-SEU. The elevations of the tips of the test piles will be -70, -77, and -85. The test piles will be left in place unless they fail during the test. Pile test loads will be 15 tons in tension and 40 tons in compression.

c. Shell backfill. Clamshell will be used as backfill around the structure to reduce lateral pressures, and to keep the settlement of the riprap protection and the heights and lengths of the floodwalls to a minimum.

d. Impervious levee and berm fill. After the floodgate and floodwalls are completed and protection against flooding is no longer necessary, the material in the protection dike will be used in the levees and berms at the end of the tie-in walls.

e. Erosion protection. To guard against loss of channel and backfill material due to erosion and subsequent undermining of the floodgate and floodwalls, 2 feet of riprap on a minimum 1-foot blanket of clamshell will be provided as shown in plan on plate I-3.

f. Settlement observations. Settlement observations will be made along the structure and floodwalls promptly after construction and yearly thereafter.

11. Spoil disposal. The major portion of the first stage excavation material will be used to construct the land dikes and a significant portion of the second stage excavation material will be used to construct the inside berms for the stream closures. The material remaining to be excavated will be deposited in the tie-in levee areas as shown on plate III-10. A portion of the material will also be stockpiled in certain areas outside of the protection dike for use in selective backfilling of the excavation in the vicinity of the structure.

12. Construction methods and sequence.

a. Perform the first stage excavation for the structure in the wet and build the land portion of the temporary protection dike with the cast material.

b. Construct the stream closure cores with shell from barges to elevation 3.0. Cast in stream closure clay blankets and berms, in the wet, and complete closure sections by casting from the structure excavation and outside borrow in the stream bottom.

c. After the protection dike and stream closures are completed, unwater the area within the dikes to the bottom of the first stage excavation at elevation -6.0, and install construction piezometers for use in monitoring the pore pressure during dewatering.

d. After the excavation is unwatered to elevation -6.0, install 12-inch diameter vertical drains in the foundation, and fill with concrete sand to provide pressure relief during the second stage excavation.

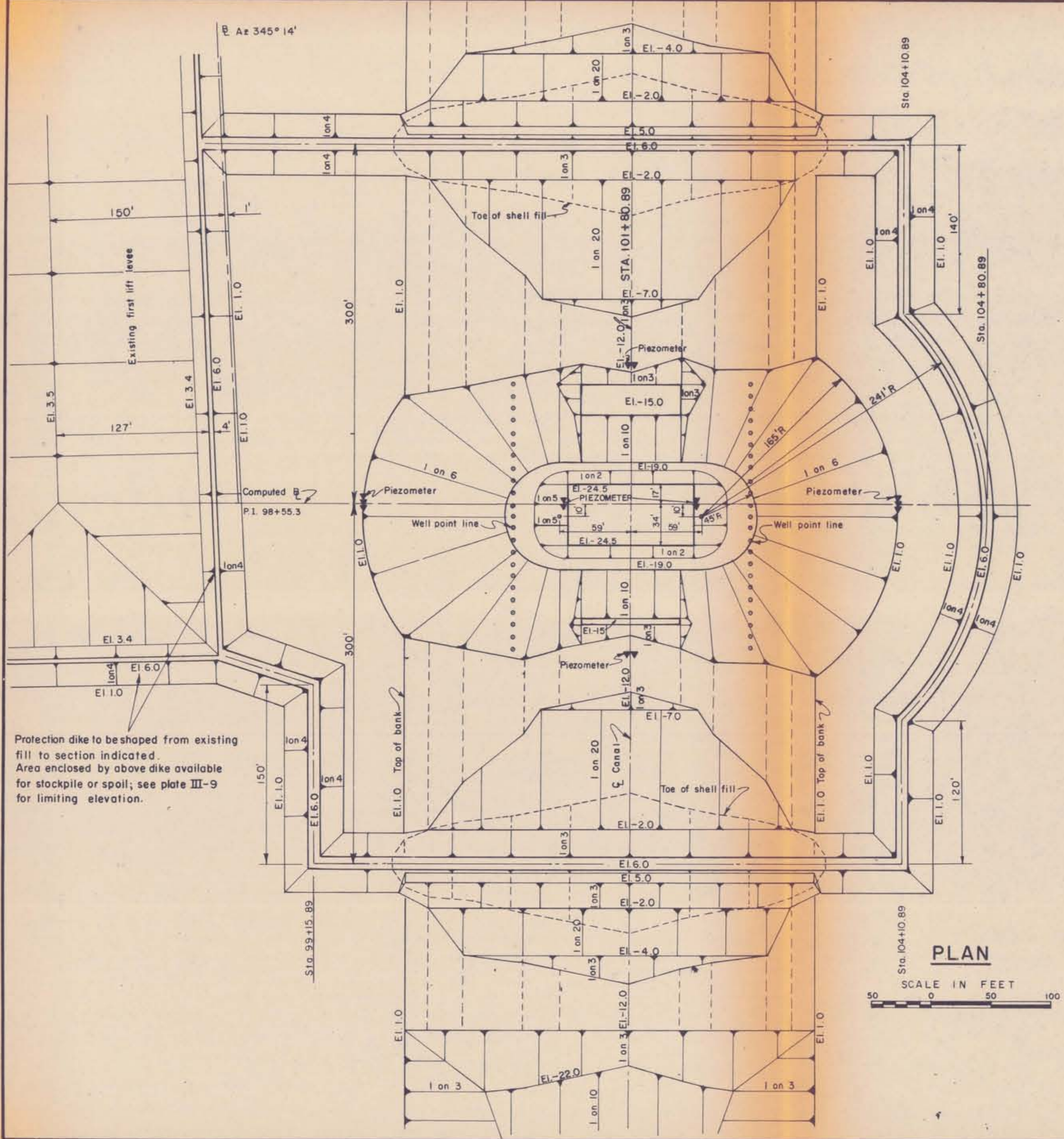
e. After the construction piezometers and vertical sand drains are installed and functioning, complete the second step excavation to elevation -19.0 in the wet by casting methods.

f. Upon completion of the second stage, unwater the excavation to elevation -19.0, and install additional construction piezometers. Install well points in the vertical sand drain and operate to lower the water in the sand drains about 16 feet.

g. Complete the third stage excavation, in the dry, to elevation -24.5. Operate well points such that the piezometric heads in the sands and silts do not exceed the elevation of the bottom of the structure excavation.

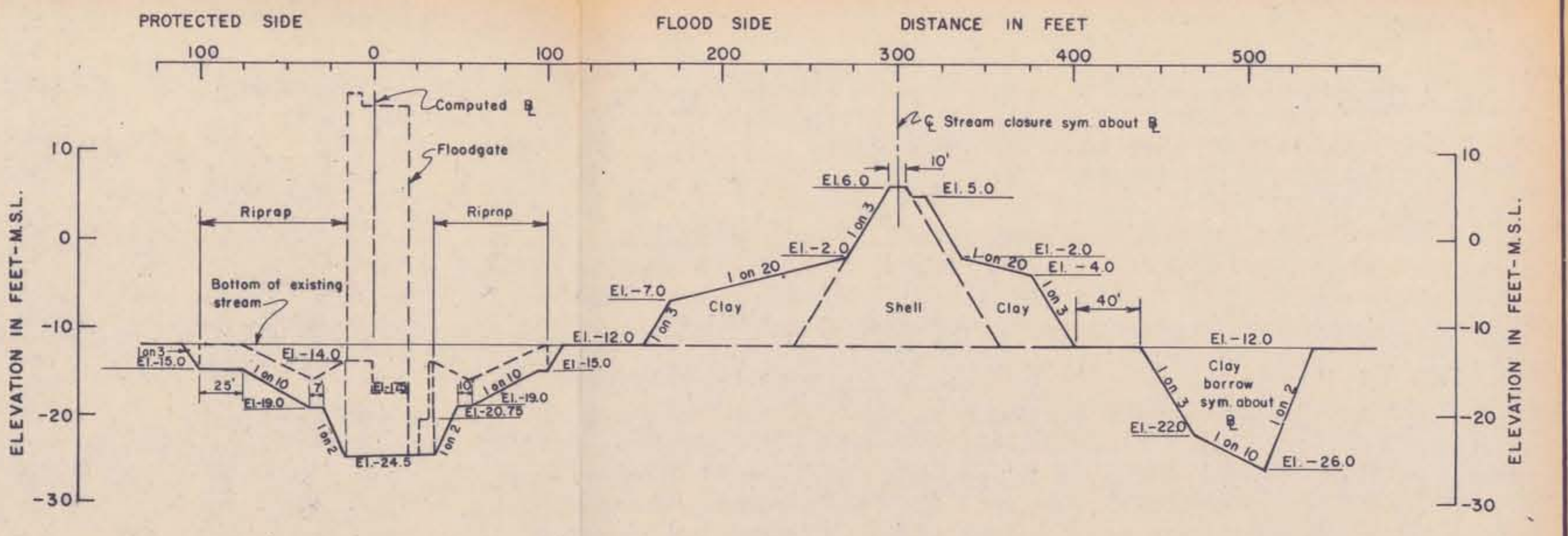
h. Construct the floodgate in the dry. Construct the T-wall and I-wall in the dry, relocating the temporary protection dike and performing the necessary excavation for the shell core under the I-wall (see plate III-10). Place backfill and riprap to an interim elevation of -4.0. Allow water to enter the area within the floodgate and channel to elevation -5.5. Place backfill and riprap to interim elevation 1.0. Remove closure and allow water to flood the area. (Natural water level is at approximately elevation -2.) Complete earthwork around I-wall and construct tie-in levees at the ends of the floodwalls.

i. Remove shell material from closures and use in access road and parking areas.

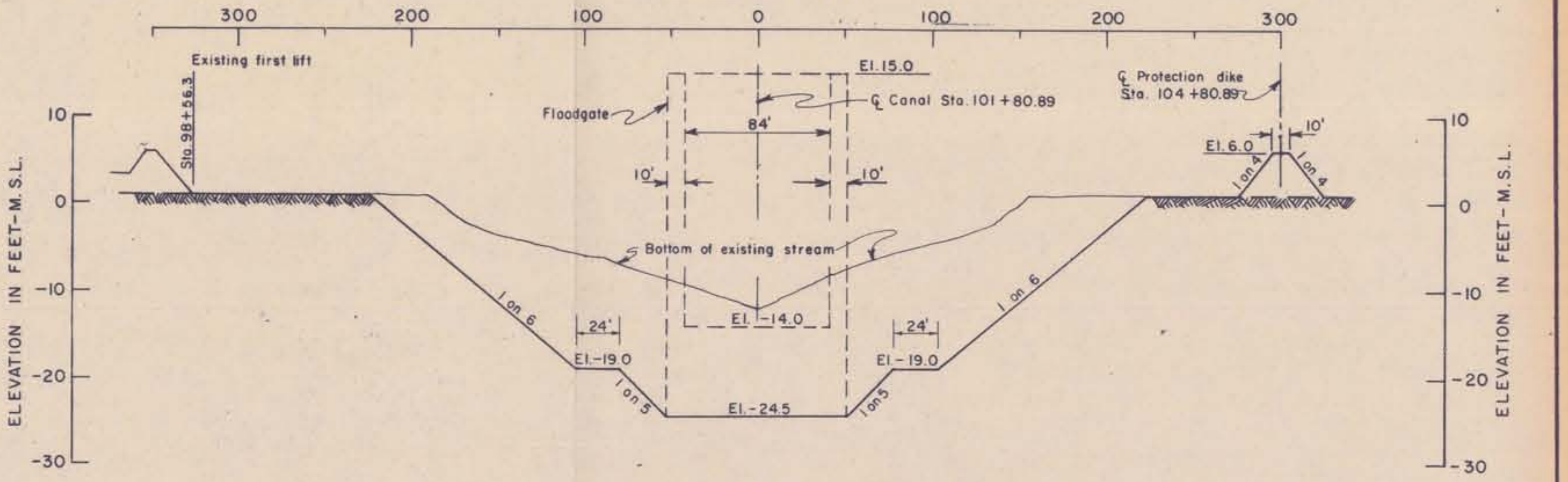


PLAN

SCALE IN FEET



SECTION ALONG ℄ OF CANAL



SECTION ALONG COMPUTED ℄

NOTE:

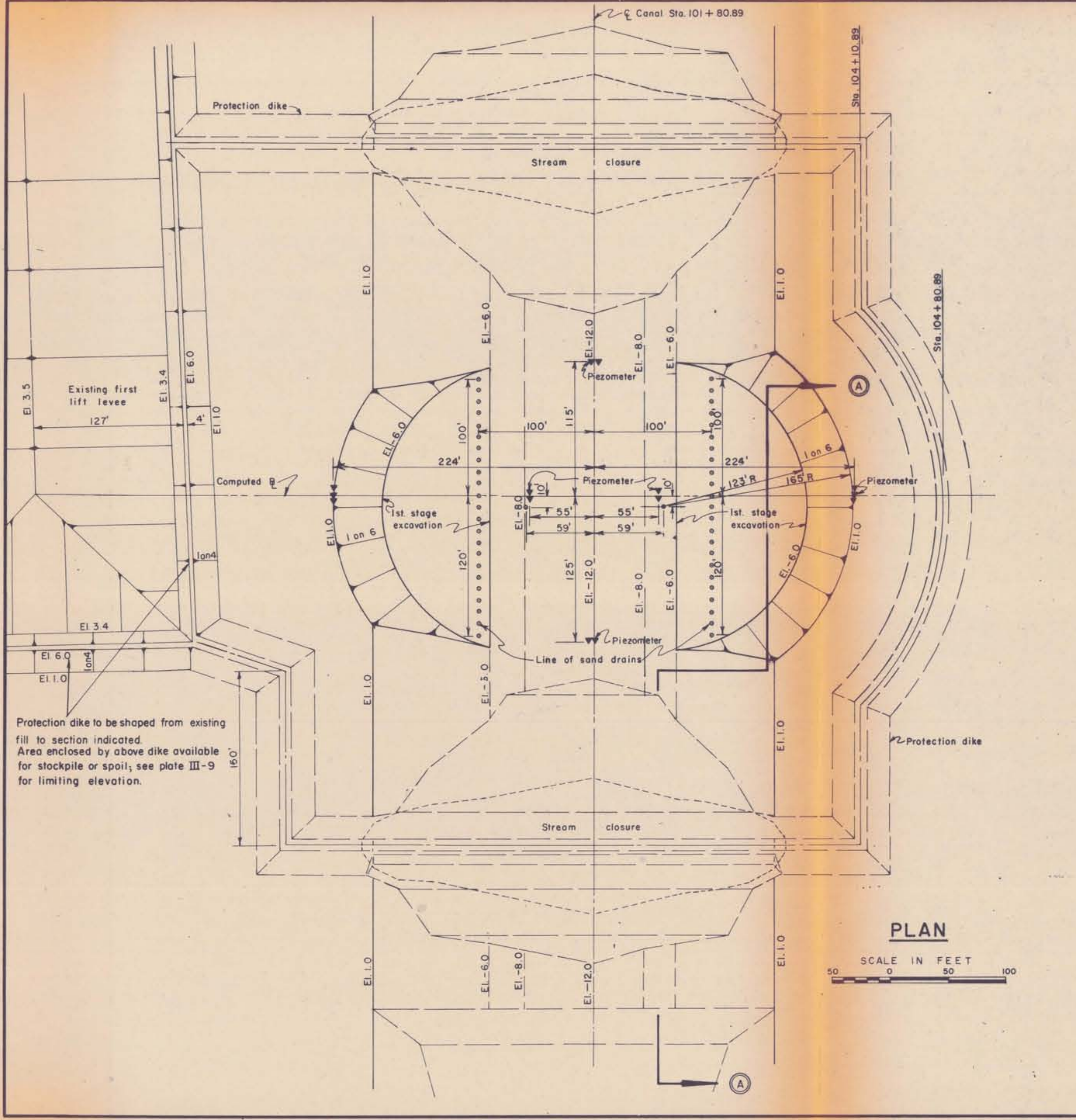
For sequence of excavation and pressure relief installation, see plate III-2.
For sand drain pressure relief, see plate III-3.
For well point pressure relief, see plate III-4.
For stream closure (Q) stability, see plates III-5 and III-6.
For protection dike (Q) stability, see plate III-7.
For structure excavation stability, see plates III-8 and III-9.
For pile capacities and subgrade moduli, see plates III-21 and III-22.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B I - TROPICAL BEND TO FORT JACKSON

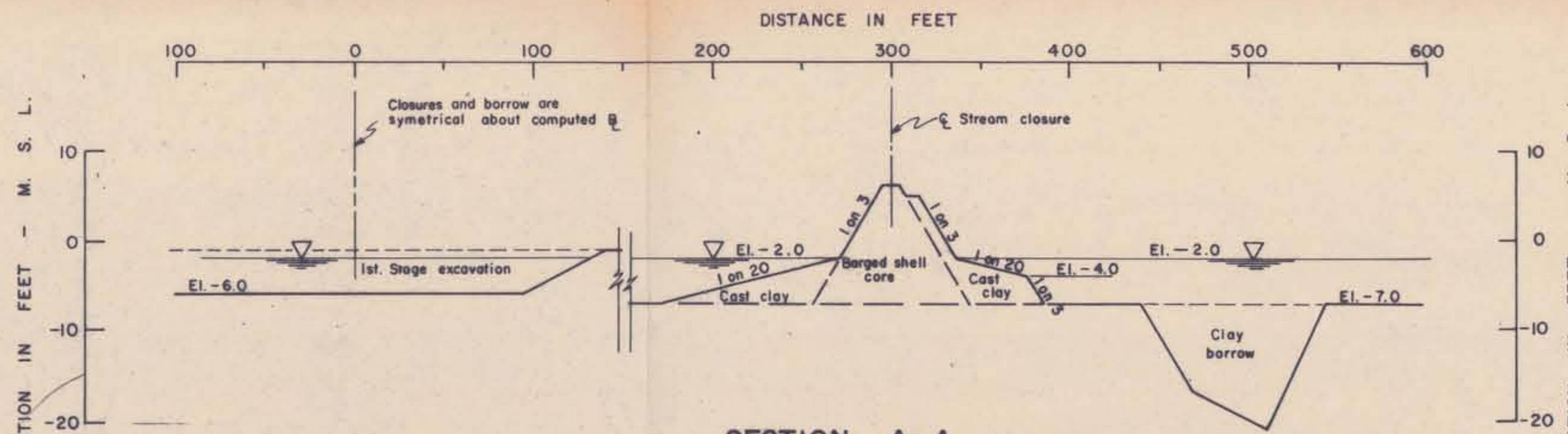
EMPIRE FLOODGATE
STRUCTURE EXCAVATION
PLAN AND SECTIONS

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

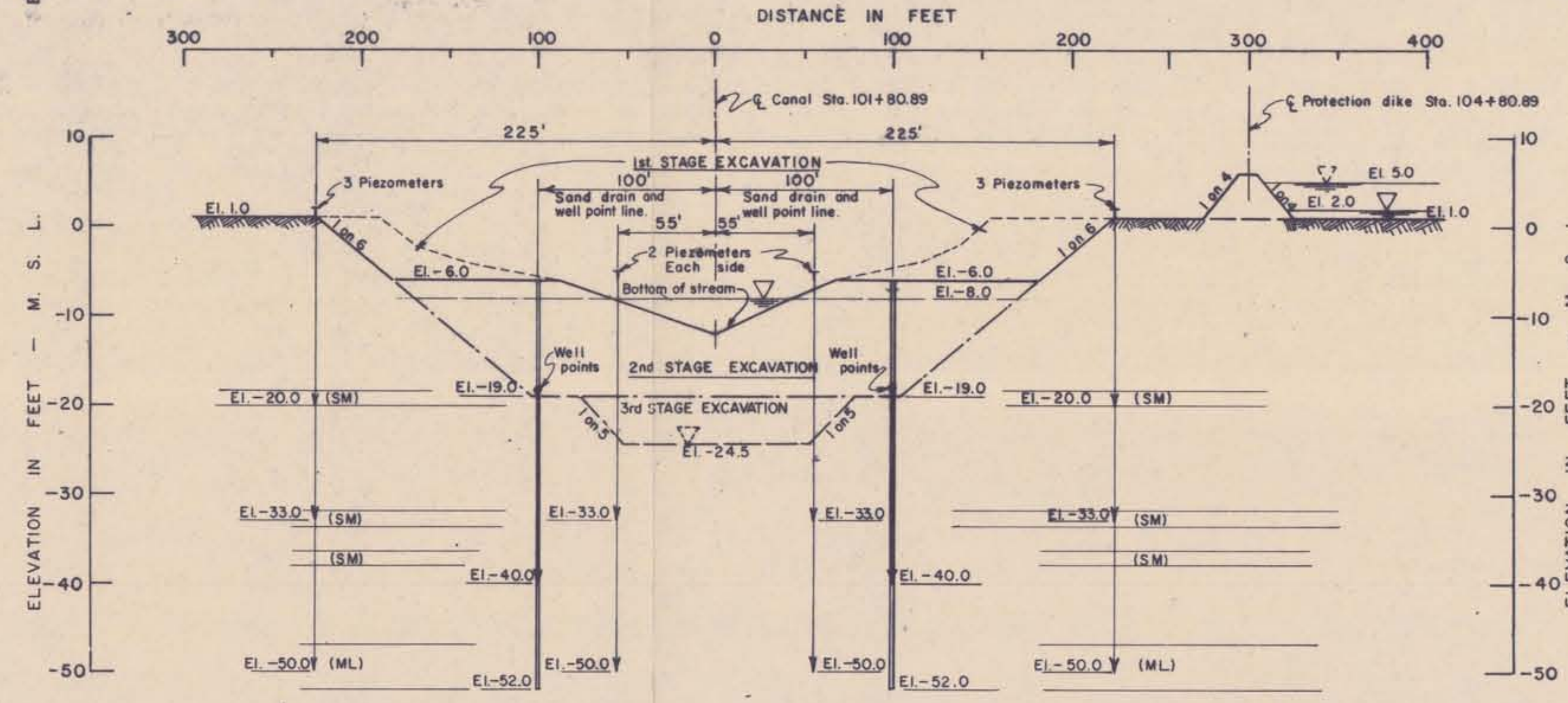
OCTOBER 1970 FILE NO. H-2-25048



PLAN



SECTION A-A



℄ SECTION

SEQUENCE OF CONSTRUCTION

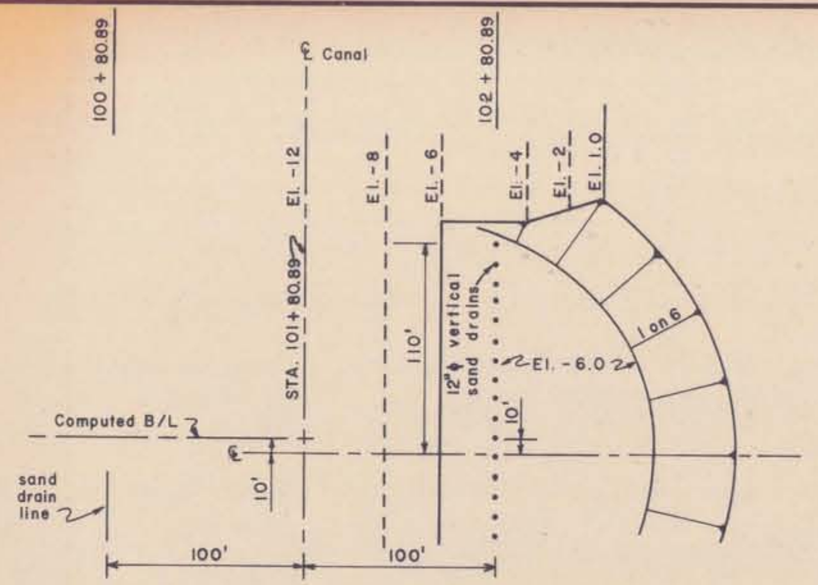
1. Construct stream closure shell core in the wet, with shell from barges to El. 3.0. (See plate III-5)
2. Construct stream closure clay blanket and berms, in the wet, by casting from 1st stage excavation and clay borrow.
3. Complete stream closure to grade and section.
4. Construct protection dike, in the wet, with the material cast from 1st. stage excavation.
5. Complete 1st. stage excavation to grade and section.
6. Unwater excavation to El. -8.0. Install construction piezometers.
7. Install 12" diameter vertical drains filled with concrete sand.
8. Complete 2nd stage excavation to grade and section. Stockpile material in tie-in levee area.
9. Unwater excavation to El. -19.0. Install construction piezometers.
10. Install well points in vertical sand drains and lower water approximately 16'. ?
11. Complete 3rd stage excavation to grade and section. Stockpile material in tie-in levee area.

NOTE:
For sand drain and well point pressure relief, see plates III-3 and III-4.
For stability analyses, see plates III-5 to III-9.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 — DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
EXCAVATION STAGES, CLOSURE AND PROTECTION DIKE, SAND DRAINS, WELL POINTS, AND PIEZOMETERS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048

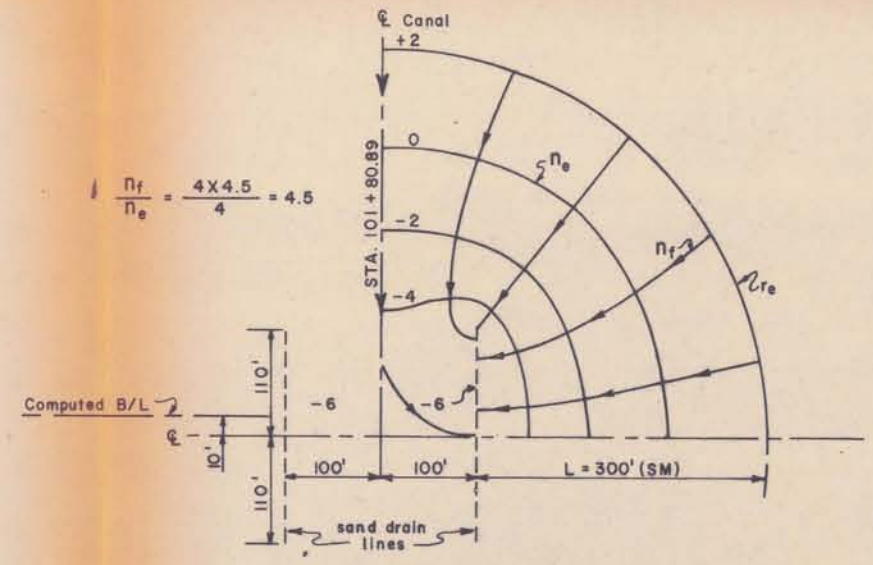
PLATE III-2



PLAN

SCALE IN FEET

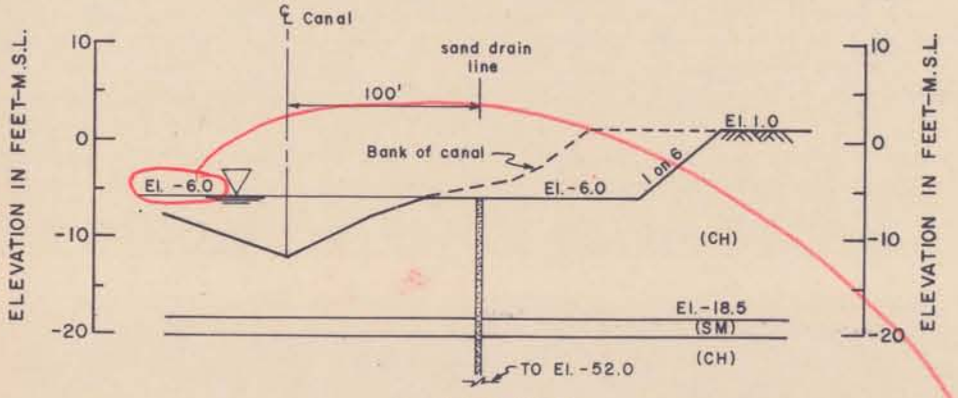
50 0 50 100



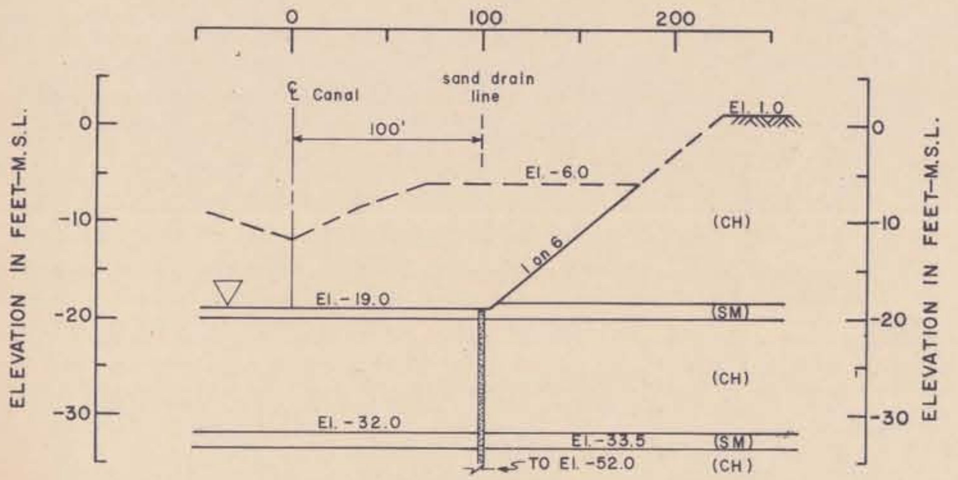
RADIAL FLOW NET PLAN

SCALE IN FEET

100 0 100 200

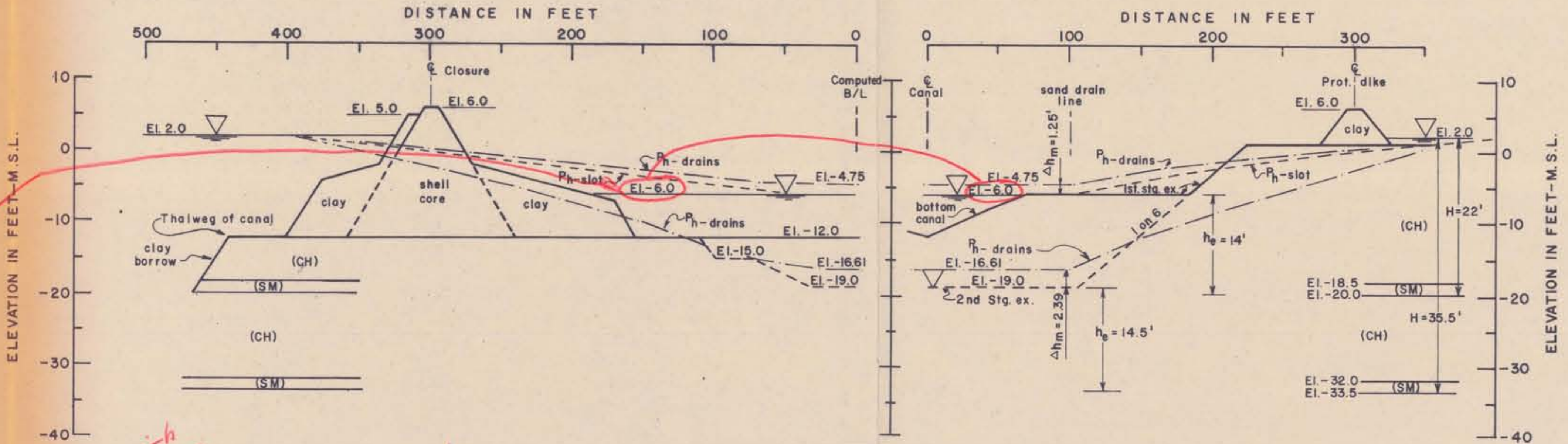


℄ SECTION
FIRST STAGE EXCAVATION



℄ SECTION
SECOND STAGE EXCAVATION

Does not agree with ℄ section plate III-2. Reconcile..!!



PRESSURE RELIEF SECTIONS

* FIRST STAGE EXCAVATION PRESSURE RELIEF

GIVEN: A line of drain holes, one foot in diameter, filled with concrete, sand, spacing = a = 10 feet permeability of concrete sand = K_D = 1000 X 10^{-4} CM. per sec. = 0.2 ft. per min., area of drain = A_D = 0.785 feet, K of (SM) layer = 100 X 10^{-4} CM. per sec. = 0.02 ft. per min., thickness of (SM) layer = D = 1.5 feet, radius of drain = r_w = 0.5 feet, distance from the bottom of the (SM) layer to the flood water surface = H = 22 feet, distance to the drain discharge face = he = 14 feet, distance from sand drain to source of flow = L = 300ft.

DETERMINE: Total flow to slots = Q_T c.f.m., average flow per foot of slot x a = Q_W c.f.m., head loss due to convergence of flow to sand drain = Δh_w in feet, head loss at mid point between sand drain lines = Δh_m in feet, assume 1 ft. hydraulic loss in drain and develop piezometric lines along the ℄ of the structure and along the ℄ of the canal, the capacity of the drain for carrying flow = Q_D in c.f.m.

CALCULATIONS:

EQ. 3-30: $q = K(H-h_e)\frac{n_f}{n_e}$, $Q_T = DK(H-he)\frac{n_f}{ne} = 1.5' \times 0.02 \times 8' \times 4.5 = 1.08$ c.f.m.

q per drain (a=10') $= Q_w = \frac{Q_T\, a'}{4 \times 110'} = 0.025$ c.f.m.

EQ. 3-32: $\Delta h_w = \frac{Q_w}{2\pi KD} \ln \frac{a}{2\pi r_w} = \frac{0.025}{0.188} \ln \frac{10}{3.14} = 0.133 \ln 3.18 = 0.15$ feet (El. -5.85)

EQ. 3-35: $\Delta h_m = \frac{Q_w}{2\pi KD} \ln \frac{a}{\pi r_w} = 0.133 \ln 6.36 = 0.133 \times 1.85 = 0.25$ feet (El. -5.75)

Assuming one foot head loss in drain, total loss at center line of canal = 1.00' + 0.25' = 1.25' = El. -4.75

EQ. 3-1a: $Q_D = KiA = K\frac{h}{l}A = 0.2 \times \frac{8'}{14'} \times 0.785 = 0.09$ c.f.m. > 0.025 (O'K')

* SECOND STAGE EXCAVATION PRESSURE RELIEF

GIVEN: K = 0.02 ft./min., D = 1.5 ft., H = 35.5 ft., h_e = 14.5', a = 10 ft. and n_f/n_e = 4.5'

CALCULATIONS:

EQ. 3-30: $Q_T = DK(H-h_e)\frac{n_f}{n_e} = 1.5' \times 0.02 \times 21.0' \times 4.5 = 2.83$ c.f.m., $Q_w = \frac{Q_T a}{4 \times 110'} = \frac{28.3}{440} = 0.064$ c.f.m.

EQ. 3-32: $\Delta h_w = \frac{Q_w}{2\pi KD} \ln \frac{a}{2\pi r_w} = \frac{0.064}{0.188} \ln \frac{10}{3.14} = 0.34 \ln 3.18 = 0.34 \times 1.16 = 0.39$ ft.

EQ. 3-35: $\Delta h_m = \frac{Q_w}{2\pi KD} \ln \frac{a}{\pi r_w} = 0.34 \ln 6.36 = 0.34 \times 1.85 = 0.63$ ft. = El. -18.37

Assume 2 foot head loss in drain, total loss at ℄ canal = 2.39 ft. = El. -16.61

EQ. 3-1a: $Q_D = KiA = 0.02 \times \frac{21}{14.5} \times 0.785 = 0.02 \times 1.145 \times 0.785 = 0.18$ c.f.m. > 0.064 (O'K')'

O.K.

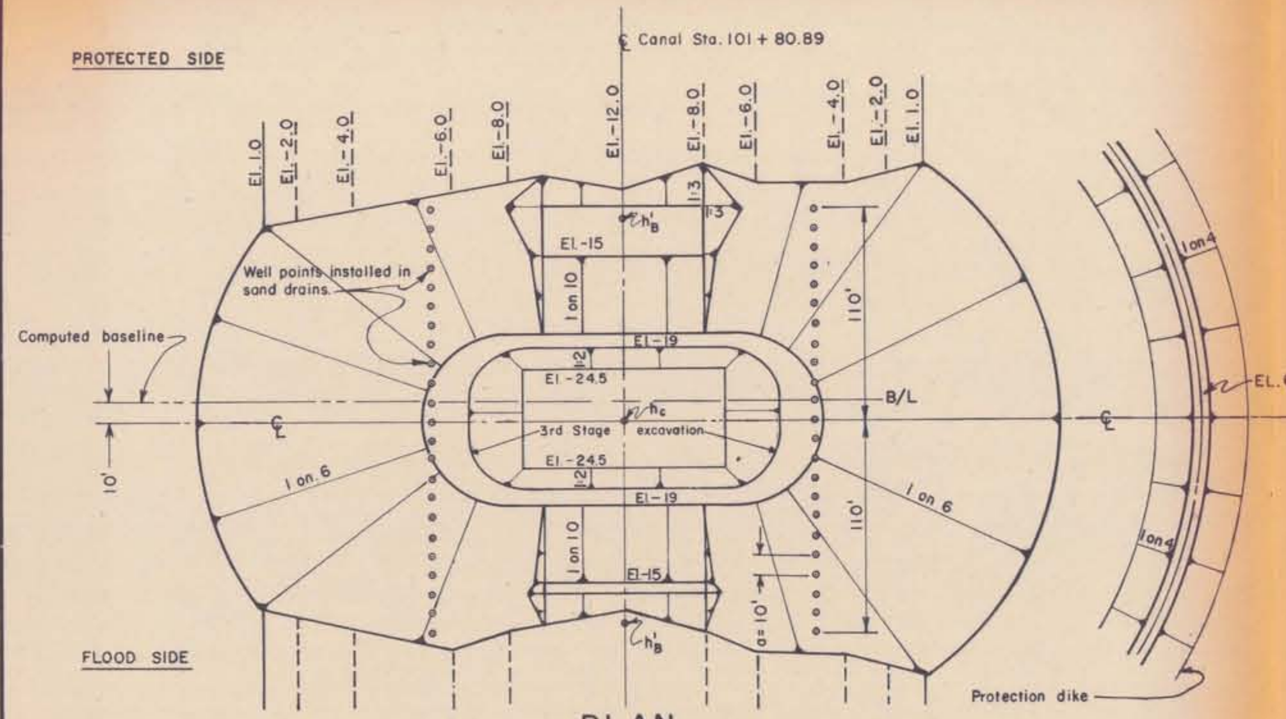
* NOTE: Procedure from Chapter No. 3 "Dewatering," Foundation Engineering Text, Leonards Editor, Authors — Mansur and Kaufman

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
SAND DRAIN PRESSURE RELIEF

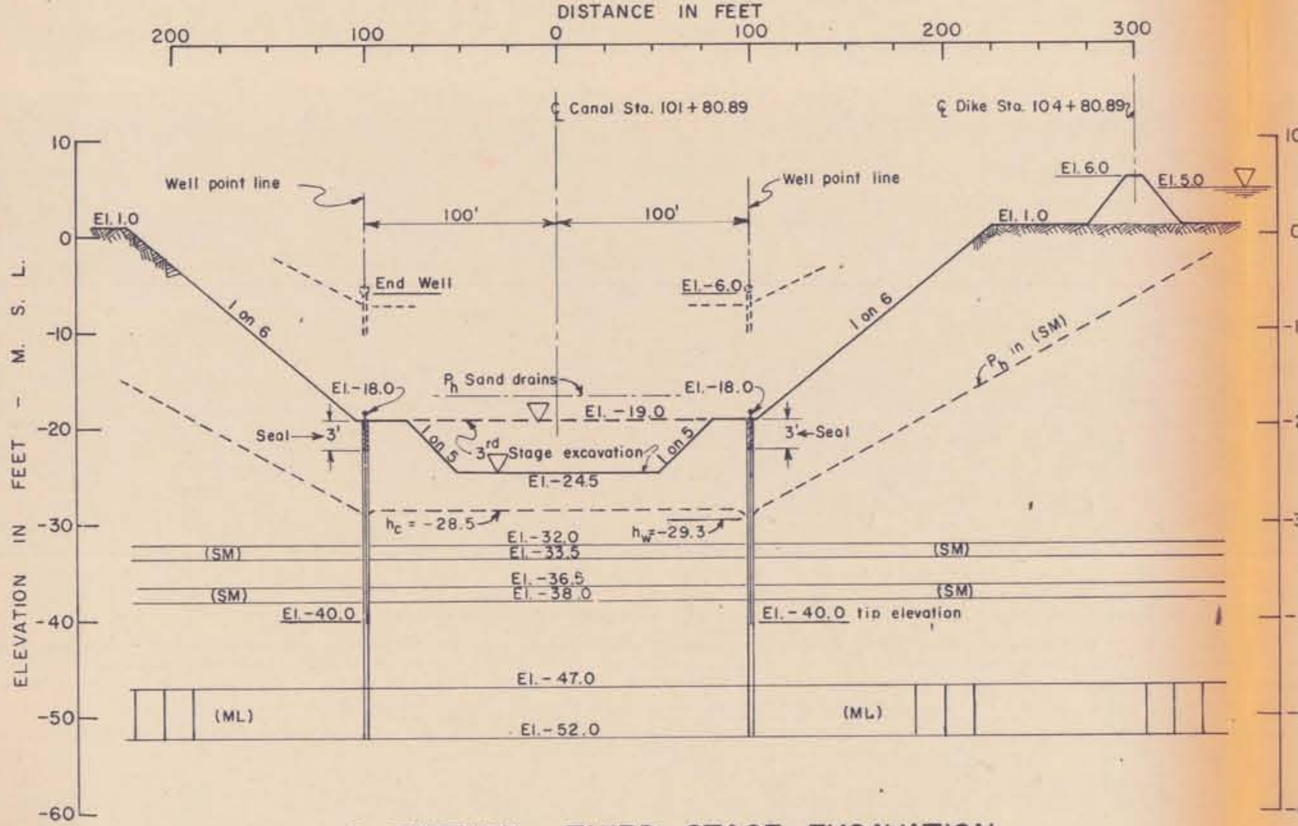
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048



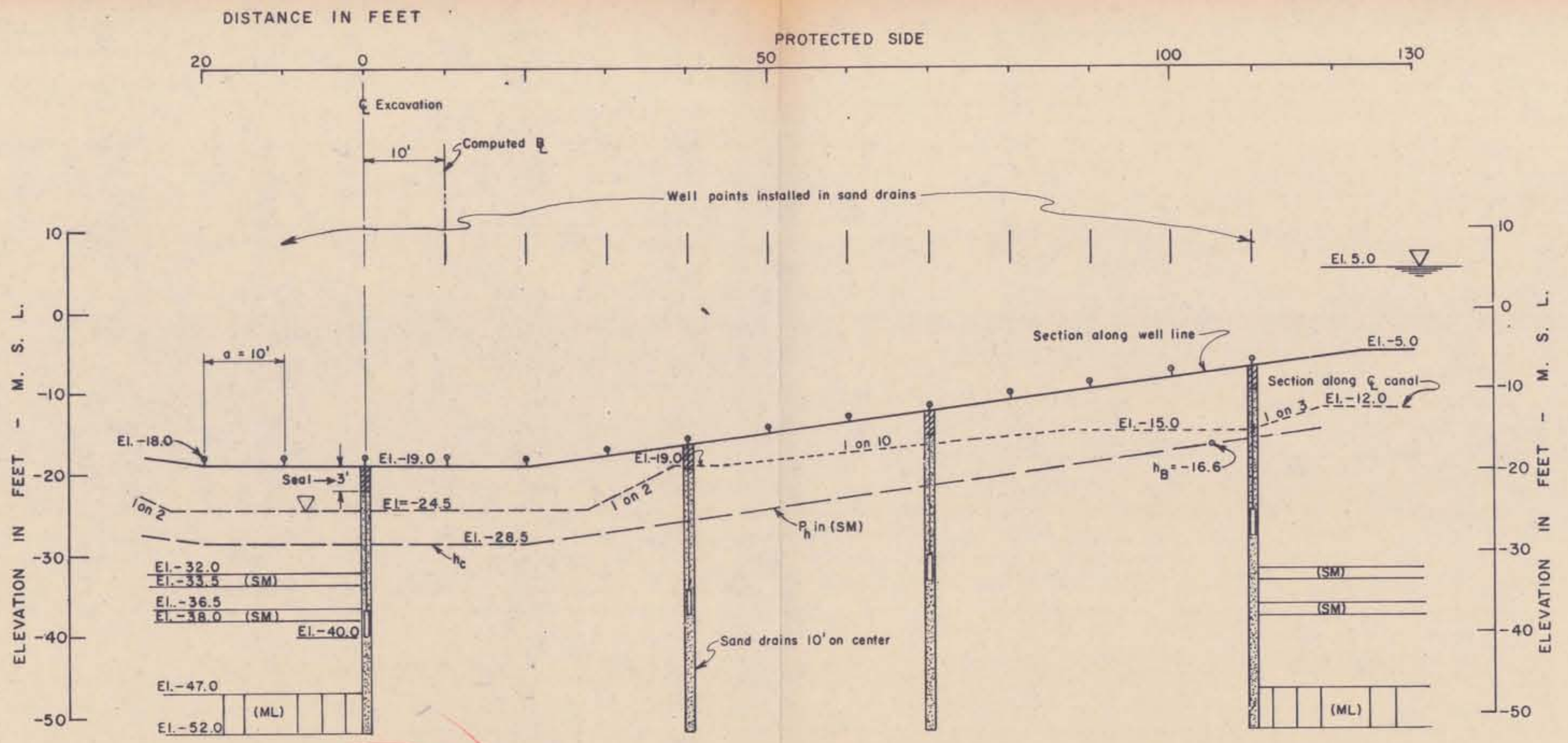
PLAN

SCALE IN FEET



℄ SECTION - THIRD STAGE EXCAVATION

(Install well points in sand drains and operate prior to excavation)



SECTION ALONG WELL LINE

Well array as shown in figure 3-36 (c); 2 lines of well points a distance of B = 200' apart, R for (SM) strata = 400', R for (ML) = 200', K(SM) = 0.02'/Min., K(ML) = 0.001'/Min., D(SM) = 1.5' each stratum, D(ML) = 5'

EQ. 3-90: $F_c = 4Q_w \sum_{i=1}^{i=n/4} \ln \frac{R}{\frac{1}{2}\sqrt{a^2(2i-1)^2+B^2}}$, n = 46 (Say 48), R/4 = 12, a = 10', Q_w = 1 c.f.m. = 7.5 g.p.m.

(SM) $F_c = 4 \times \ln\left[\left(\frac{2R}{a}\right)^{n/4} \times \frac{1}{\sqrt{\prod_{i=1}^{i=n/4}\left[(2i-1)^2+\left(\frac{B}{a}\right)^2\right]}}\right] = 4 \times \ln\left[\left(\frac{2B}{a}\right)^{n/4} \times \phi\right] = 4 \times \ln\left[\left(\frac{2 \times 400}{10}\right)^{12} \times (2.887 \times 10^{-18})\right] = 4 \times 12.2 = 48.8$

EQ. 3-77: $H - h_c = \frac{F_c}{2\pi KD} = \frac{48.8}{2\pi \times 0.02 \times 3'} = \frac{48.8}{0.377} = 129$ (1 c.f.m.); for Q_w = 2 g.p.m., $H - h_c = \frac{2}{7.5} \times 129 = 0.26 \times 129 = 33.5'$, Head water at elevation 5.0, H = 38.0 + 5.0 = 43.0', h_c = 43.0' − 33.5' = 9.5' = El. [−38.0 − (−9.5)] = El. −28.5

(ML) $F_c = 4 \ln 48.44 = 4 \times 6.18 = 24.72$

$H - h_c = \frac{24.72}{2\pi \times 0.001 \times 5} = \frac{24.72}{0.031} = 797$ (1 c.f.m), flow from (ML) stratum $= \frac{5 \times 0.001}{3 \times 0.02} \times 100 = 0.08 \times 100 = 8\%$ of (SM) flow, 8% of 2 g.p.m. = 0.16 g.p.m., $H - h_c = \frac{0.16}{7.5} \times 797 = 0.021 \times 797 = 16.7'$, h_c(ML) = 57.0' − 16.7' = 40.3 = El. −11.7

EQ. 3-37: $\Delta h_D = \frac{Q_w}{2\pi KD} \ln \frac{a}{2\pi r_w} = \frac{0.267}{2\pi \times 0.02 \times 3'} \ln \frac{10'}{2\pi \times 0.5} = \frac{0.267}{0.377} \ln \frac{10}{3.14} = 0.71 \ln 3.18 = 0.71 \times 1.16 = 0.82'$, $h_w = h_c - \Delta h_D = 9.5 - 0.8 = 8.7'$ = El. −29.3

EQ. 3-35: $\Delta h_m = \frac{Q_w}{2\pi KD} \ln \frac{a}{\pi r_w} = 0.71 \ln 6.36 = 0.71 \times 1.85 = 1.3'$, head midway between wells = $h_w + \Delta h_w$ = El. −28.0

Hydraulic head loss in 2½" ID. self-jetting well point with 2" ID. riser pipe 20' long

EQ. 3-101: $H_w = H_e + H_s + H_r + H_v = 0.1 + 0.5 + 0.4 = 1.0'$

Drawdown in well ≦ Effective vacuum (V = 20')

D. D. = 29.3 + 1.00 − 18.0 ≦ 20.00; D. D. = 12.3' < 20.0' (O'K')

EQ. 3-91: $F_B = 2Q_w \sum_{i=1}^{i=n/2} \ln \frac{R}{\frac{1}{2}\sqrt{a^2(2i-3)^2+B^2}} = 2 \times 0.267 \times 23.018 = 12.3$

$H - h_B = \frac{F_B}{2\pi KD} = \frac{12.3}{0.377} = 32.6'$, h_B = 43.0 − 32.6 = 10.4' = El. −27.6

Adjust h_B by + 11' for rise in slope, h_B' = −27.6 + 11.0 = El. −16.6

NOTE:
Procedure from Chapter No. 3 "Dewatering," Foundation Engineering Text, Leonards Editor, Authors — Mansur and Kaufman.

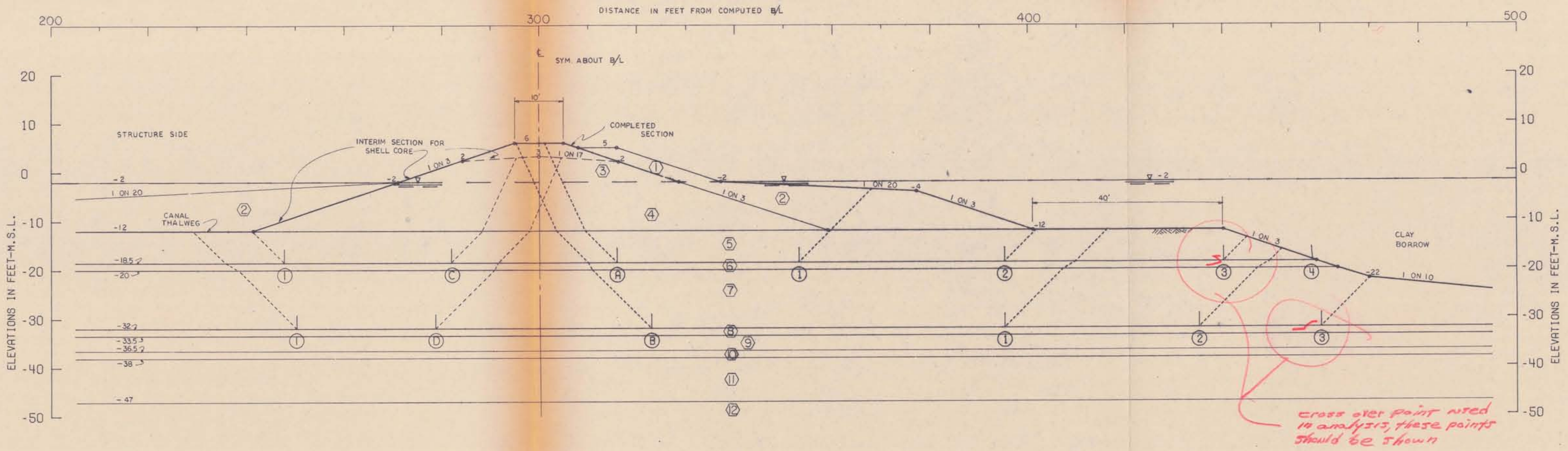
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B I - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
WELL POINT PRESSURE RELIEF

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

OCTOBER 1970 FILE NO. H-2-25048

PLATE III - 4



GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III-24 AND III-25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	DEGREES
1	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
2	CH	40.0	40.0	100.0	100.0	100.0	100.0	0.
3	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
4	SHELL	30.0	30.0	0.	0.	0.	0.	40.0
5	CH	40.0	40.0	150.0	150.0	150.0	150.0	0.
6	SM	55.0	55.0	0.	0.	0.	0.	30.0
7	CH	40.0	40.0	250.0	250.0	350.0	350.0	0.
8	SM	55.0	55.0	0.	0.	0.	0.	30.0
9	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
10	SM	55.0	55.0	0.	0.	0.	0.	30.0
11	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
12	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT. 1 - VERT. 2 - BORING 1-SEU

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A 1	-18.50	10738	5550	3640	18317	4633	19928	13684	1.456
A 2	-18.50	10738	11850	1950	18317	1083	24538	17234	1.424
A 3	-18.50	10738	18600	1452	18317	628	30790	17689	1.741
A 4	-18.50	10738	20022	76	18317	1	30836	18315	1.684
B 1	-32.00	18449	25200	8853	38804	8523	52503	30281	1.734
B 2	-32.00	18449	39200	7281	38804	7087	64931	31717	2.047
B 3	-32.00	18449	47879	5000	38804	2650	71329	36154	1.973
C 1	-18.50	7095	5100	1950	11802	1055	14145	10747	1.316
D 1	-32.00	14519	9975	8852	29090	8682	33346	20408	1.634

NOTES

- Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

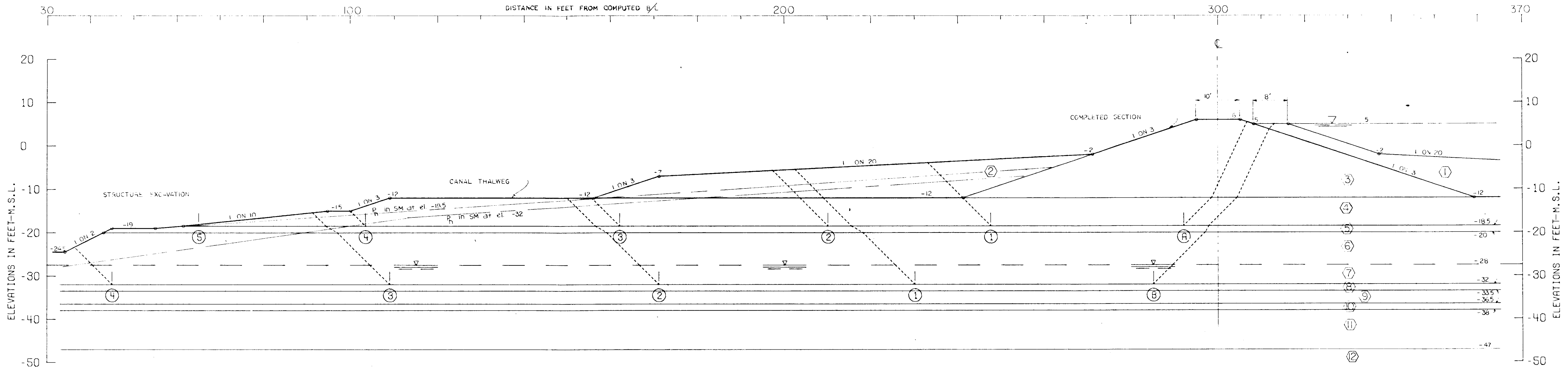
$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE STREAM CLOSURE SECTION STABILITY (Q)

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

OCTOBER 1970 FILE NO. H-2-25048



GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III 24 AND III 25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	DEGREES
1	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
2	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
3	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
4	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.
5	SM	117.0	117.0	0.	0.	0.	0.	30.0
6	CH	102.0	102.0	212.0	212.0	275.0	275.0	0.
7	CH	40.0	40.0	312.0	312.0	350.0	350.0	0.
8	SM	55.0	55.0	0.	0.	0.	0.	30.0
9	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
10	SM	55.0	55.0	0.	0.	0.	0.	30.0
11	CH	400	400	525.0	525.0	600.0	600.0	0.
12	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT 1 - VERT 2 - BORING 1 - SEU

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	-18.50	13546	6675	3569	27601	11330	23790	16271	1.462
A 2	-18.50	13546	12300	3211	27601	8780	29058	18820	1.544
A 3	-18.50	13546	19500	1950	27601	2763	34996	24838	1.409
A 4	-18.50	13546	28275	1050	27601	830	42871	26770	1.601
A 5	-18.50	13546	32745	112	27601	7	46403	27593	1.682
B 1	-32.00	21321	19250	13731	65440	37515	54303	27925	1.945
B 2	-32.00	21321	39900	10362	65440	23811	71583	41629	1.720
B 3	-32.00	21321	61600	8254	65440	15688	91175	49752	1.833
B 4	-32.00	21321	84000	4716	65440	5145	110037	60295	1.825

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

NEW ORLEANS TO VENICE, LA.

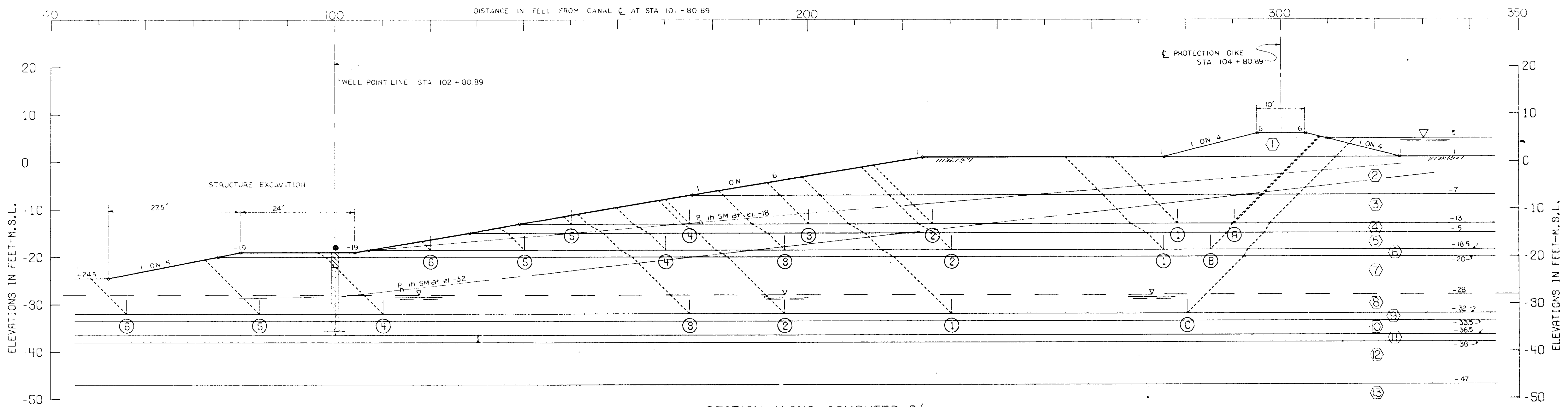
DESIGN MEMORANDUM NO.2 - DETAIL DESIGN

REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE STREAM CLOSURE SECTION STABILITY (Q)

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

FILE NO. H-2-25048



SECTION ALONG COMPUTED B/L

GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III-24 AND III-25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	CH	80.0	80.0	120.0	120.0	120.0	120.0	0.
2	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.
3	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
4	SM	117.0	117.0	0.	0.	0.	0.	30.0
5	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.
6	SM	117.0	117.0	0.	0.	0.	0.	30.0
7	CH	102.0	102.0	212.0	212.0	275.0	275.0	0.
8	CH	40.0	40.0	312.0	312.0	350.0	350.0	0.
9	SM	55.0	55.0	0.	0.	0.	0.	30.0
10	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
11	SM	55.0	55.0	0.	0.	0.	0.	30.0
12	CH	40.0	40.0	525.0	525.0	600.0	600.0	0
13	ML	55.0	55.0	2000	2000	2000	2000	12.0

VERT. 1 = VERT. 2 = BORING 1-SEU

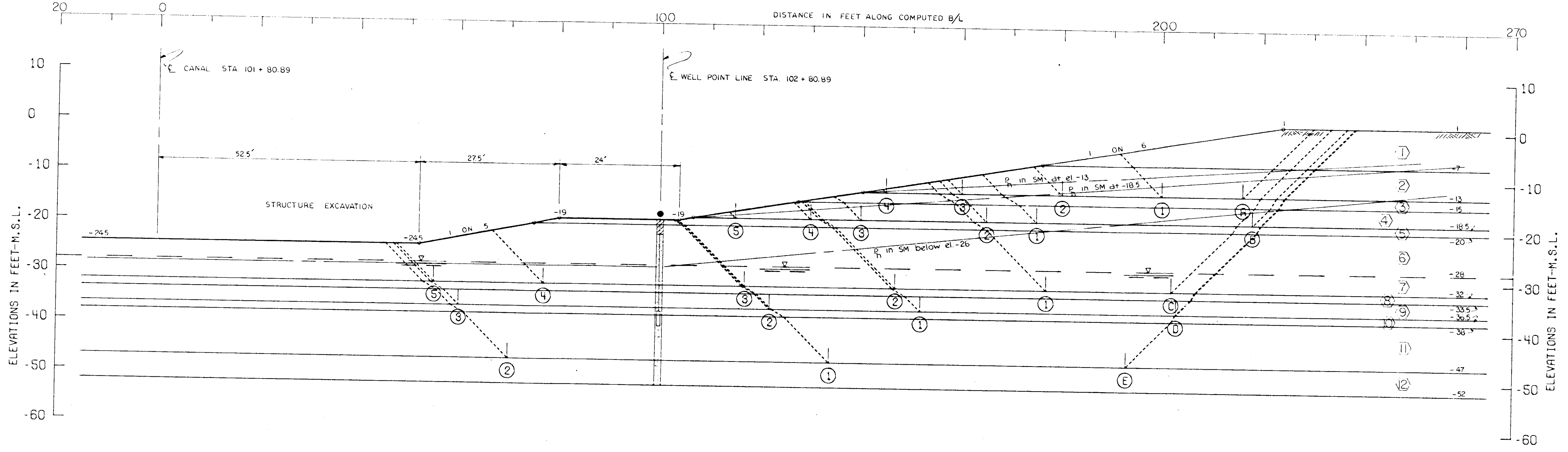
ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	-13.00	4608	1200	3600	14304	8160	9408	6144	1.531
A 2	-13.00	4608	6400	3094	14304	7282	14102	7022	2.008
A 3	-13.00	4608	9000	2000	14304	3835	15608	10469	1.491
A 4	-13.00	4608	11500	1028	14304	1571	17136	12733	1.346
A 5	-13.00	4608	14000	314	14304	146	18922	14158	1.336
B 1	-18.50	7855	1500	10060	24154	16544	19416	7610	2.551
B 2	-18.50	7855	8250	9423	24154	15321	25529	8833	2.890
B 3	-18.50	7855	13500	6213	24154	8874	27568	15280	1.804
B 4	-18.50	7855	17250	3955	24154	4991	29060	19163	1.516
B 5	-18.50	7855	21750	1261	24154	1489	30866	22664	1.362
B 6	-18.50	7855	24724	557	24154	204	33136	23950	1.384
C 1	-32.00	15787	17500	19463	61172	45866	52751	15306	3.446
C 2	-32.00	15787	29750	14966	61172	34231	60504	26941	2.246
C 3	-32.00	15787	36750	12109	61172	27403	64647	33769	1.914
C 4	-32.00	15787	59500	6004	61172	8645	81292	52527	1.548
C 5	-32.00	15787	68600	5679	61172	7528	90067	53644	1.679
C 6	-32.00	15787	78398	3980	61172	2529	98166	58643	1.674

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
PROTECTION DIKE
STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III-24 AND III-25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.
2	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
3	SM	117.0	117.0	0.	0.	0.	0.	30.0
4	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.
5	SM	117.0	117.0	0.	0.	0.	0.	30.0
6	CH	102.0	102.0	212.0	212.0	275.0	275.0	0.
7	CH	40.0	40.0	312.0	312.0	350.0	350.0	0.
8	SM	55.0	55.0	0.	0.	0.	0.	30.0
9	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
10	SM	55.0	55.0	0.	0.	0.	0.	30.0
11	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
12	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT. 1 - VERT. 2 - BORING 1-SEU

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A 1	-13.00	3600	1600	1971	7665	3770	7171	3894	1.842
A 2	-13.00	3600	3600	1142	7665	1907	8342	5757	1.449
A 3	-13.00	3600	5600	571	7665	484	9771	7180	1.361
A 4	-13.00	3600	6967	142	7665	30	10710	7635	1.403
B 1	-18.50	6421	6450	4341	16305	5618	17213	10687	1.611
B 2	-18.50	6421	7950	3376	16305	4124	17747	12180	1.457
B 3	-18.50	6421	11700	1239	16305	1421	19360	14883	1.301
B 4	-18.50	6421	13200	1007	16305	674	20629	15631	1.320
B 5	-18.50	6421	15224	364	16305	87	22010	16218	1.357
C 1	-32.00	14046	8750	12257	47034	27761	35053	19273	1.819
C 2	-32.00	14046	19250	8623	47034	17895	41920	29139	1.439
C 3	-32.00	14046	29750	6017	47034	9924	49814	37110	1.342
C 4	-32.00	14046	43750	5194	47034	6055	62991	40979	1.537
C 5	-32.00	14046	51437	3980	47034	2460	69464	44574	1.558
D 1	-36.50	19385	21675	16730	61156	26772	57790	34384	1.681
D 2	-36.50	19385	34425	12555	61156	16459	66365	44696	1.485
D 3	-36.50	19385	60775	8275	61156	5742	88435	55413	1.596
E 1	-47.00	31772	35400	27094	93422	37213	94267	56208	1.677
E 2	-47.00	31772	72414	20372	93422	18180	124559	75242	1.655

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

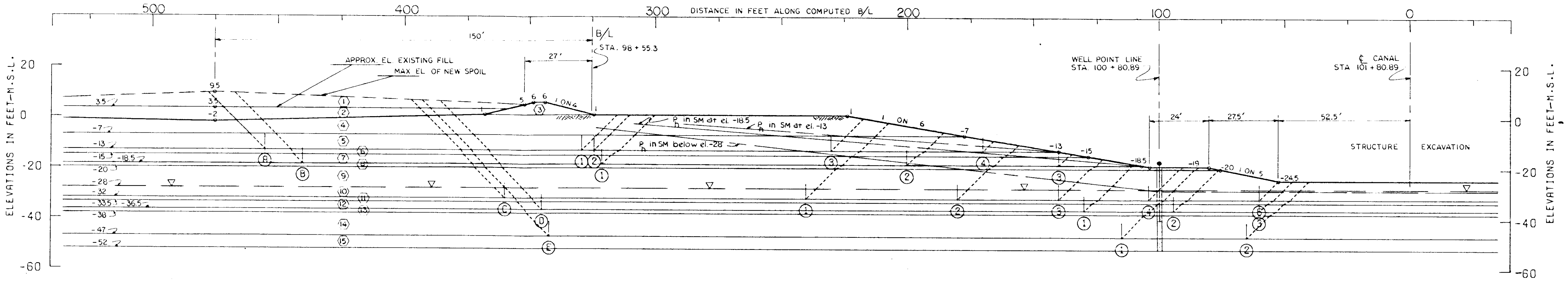
$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
STRUCTURE EXCAVATION
STABILITY (Q)

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

OCTOBER 1970 FILE NO. H-2-25048



GENERAL NOTES

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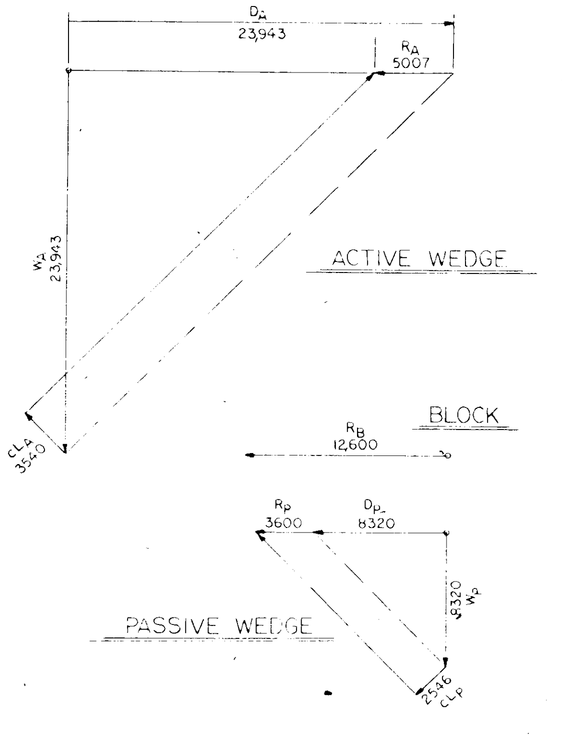


STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.0
2	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.0
3	CH	80.0	80.0	120.0	120.0	120.0	120.0	0.0
4	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.0
5	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.0
6	SM	117.0	117.0	0.0	0.0	0.0	0.0	30.0
7	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.0
8	SM	117.0	117.0	0.0	0.0	0.0	0.0	30.0
9	CH	102.0	102.0	212.0	212.0	275.0	275.0	0.0
10	CH	40.0	40.0	312.0	312.0	350.0	350.0	0.0
11	SM	55.0	55.0	0.0	0.0	0.0	0.0	30.0
12	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.0
13	SM	55.0	55.0	0.0	0.0	0.0	0.0	30.0
14	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.0
15	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT. 1 - VERT. 2 - BORING 1 - SEU

$$FS = \frac{5007 + 12600 + 3600}{23943 - 8320}$$

$$FS = \frac{21207}{15623} = 1.357$$

ASSUMED FAILURE SURFACE (A) (1)

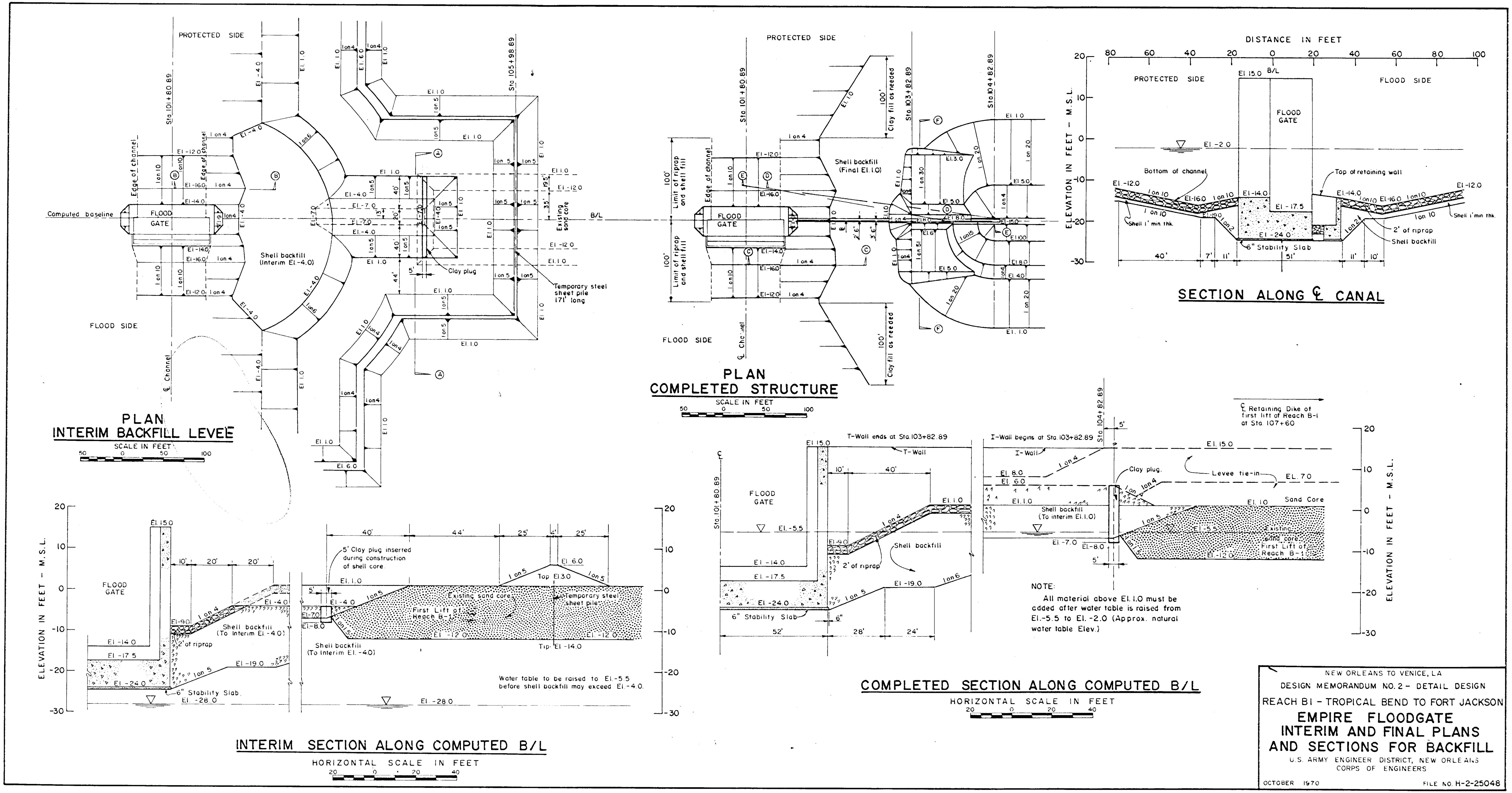
ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
(A) (1)	-13.00	5007	12600	3600	23943	8320	21207	15622	1.357
(A) (2)	-13.00	5007	13100	3600	23943	8071	21707	15872	1.368
(A) (3)	-13.00	5007	22500	3251	23943	7708	30758	16235	1.895
(A) (4)	-13.00	5007	28500	837	23943	1037	34344	22906	1.499
(B) (1)	-18.50	8919	17850	10061	35880	16544	36830	19336	1.905
(B) (2)	-18.50	8919	36000	6546	35880	9469	51465	26412	1.949
(B) (3)	-18.50	8919	45000	1231	35880	1413	55150	34467	1.600
(C) (1)	-32.00	16489	42000	20635	68287	48194	79124	20093	3.938
(C) (2)	-32.00	16489	63000	12614	68287	28558	92102	39729	2.318
(C) (3)	-32.00	16489	77000	7957	68287	15690	101446	52597	1.929
(C) (4)	-32.00	16489	89600	6004	68287	8306	112093	59980	1.869
(C) (5)	-32.00	16489	105000	4015	68287	3006	125503	65280	1.923
(D) (1)	-36.50	22085	91375	13492	81512	18887	126952	62625	2.027
(D) (2)	-36.50	22085	106675	11697	81512	13507	140457	68005	2.065
(D) (3)	-36.50	22085	121125	8276	81512	5743	151486	75769	1.999
(E) (1)	-47.00	35199	136200	25334	122582	30956	196732	91627	2.147
(E) (2)	-47.00	35199	164388	19948	122582	16789	219535	105793	2.075

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SECTION ALONG BASELINE
EXISTING FILL STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



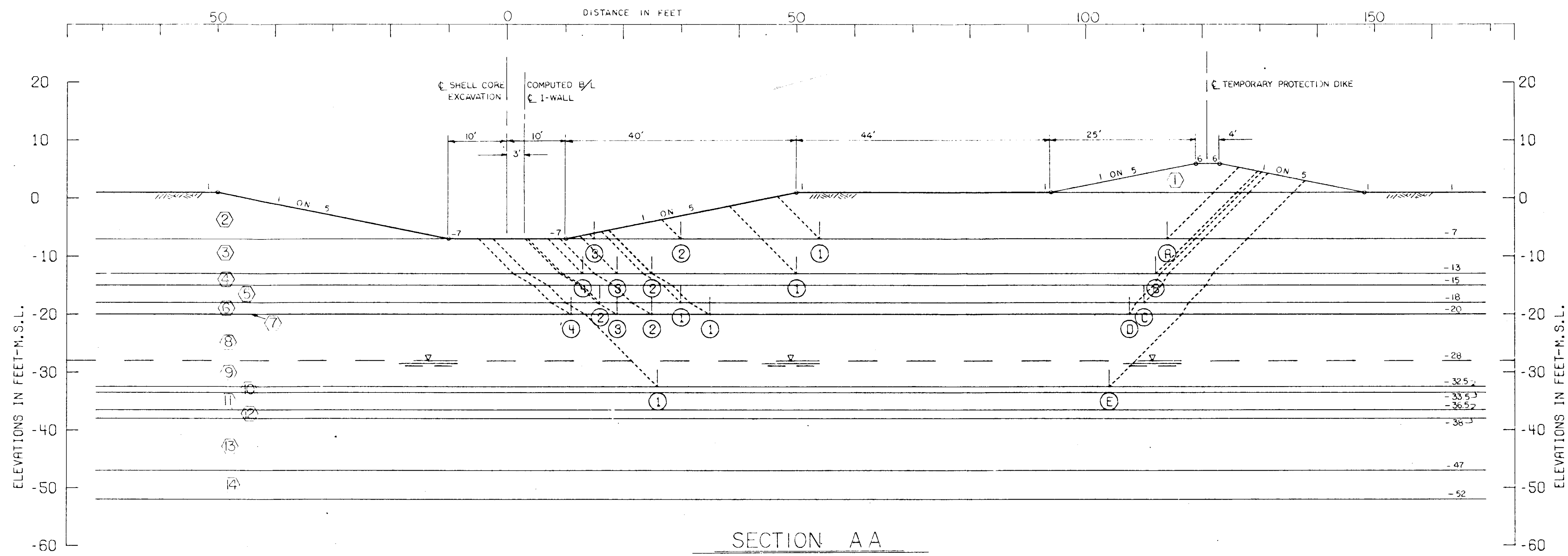
NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
INTERIM AND FINAL PLANS AND SECTIONS FOR BACKFILL

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

OCTOBER 1970 FILE NO. H-2-25048

PLATE III-10



SECTION AA

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F. VERT. 1	EFFECTIVE UNIT WT. P.C.F. VERT. 2	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	CENTER OF STRATUM VERT. 2	BOTTOM OF STRATUM VERT. 1	BOTTOM OF STRATUM VERT. 2	FRICTION ANGLE DEGREES
1	CH	80.0	80.0	120.0	120.0	120.0	120.0	0.
2	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.
3	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
4	SM	117.0	117.0	0.	0.	0.	0.	30.0
5	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.
6	SM	117.0	117.0	0.	0.	0.	0.	30.0
7		0.	0.	0.	0.	150.0	150.0	0.
8	CH	102.0	102.0	214.0	214.0	278.0	278.0	0.
9	CH	40.0	40.0	314.0	314.0	350.0	350.0	0.
10	SM	55.0	55.0	0.	0.	0.	0.	30.0
11	CH	40.0	40.0	400.0	400.0	425.6	425.0	0.
12	SM	55.0	55.0	0.	0.	0.	0.	30.0
13	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
14	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT. 1 = VERT. 2 = BORING 1-SEU

ASSUMED FAILURE NO.	SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	-7.00	3440	9000	2200	6393	2389	14640	4004	3.656
A 2	-7.00	3440	12600	1000	6393	518	17040	5875	2.900
A 3	-7.00	3440	14850	250	6393	32	18540	6361	2.914
B 1	-13.00	4480	6200	2900	13864	6796	13580	7067	1.921
B 2	-13.00	4480	8700	1650	13864	3060	14830	10803	1.373
B 3	-13.00	4480	9300	1350	13864	2406	15130	11458	1.320
B 4	-13.00	4480	9900	1200	13864	1903	15580	11960	1.303
C 1	-18.00	7516	12000	6344	22852	8687	25860	14164	1.826
C 2	-18.00	7516	14100	5095	22852	6537	26712	16314	1.637
D 1	-20.00	10367	10875	13043	27033	12247	34285	14786	2.319
D 2	-20.00	10367	12375	11341	27033	10119	34083	16914	2.015
D 3	-20.00	10367	13275	10600	27033	9268	34242	17765	1.927
D 4	-20.00	10367	14475	10098	27033	8828	34940	18205	1.919
E 1	-32.50	16593	27300	16449	62378	35441	60343	26937	2.240

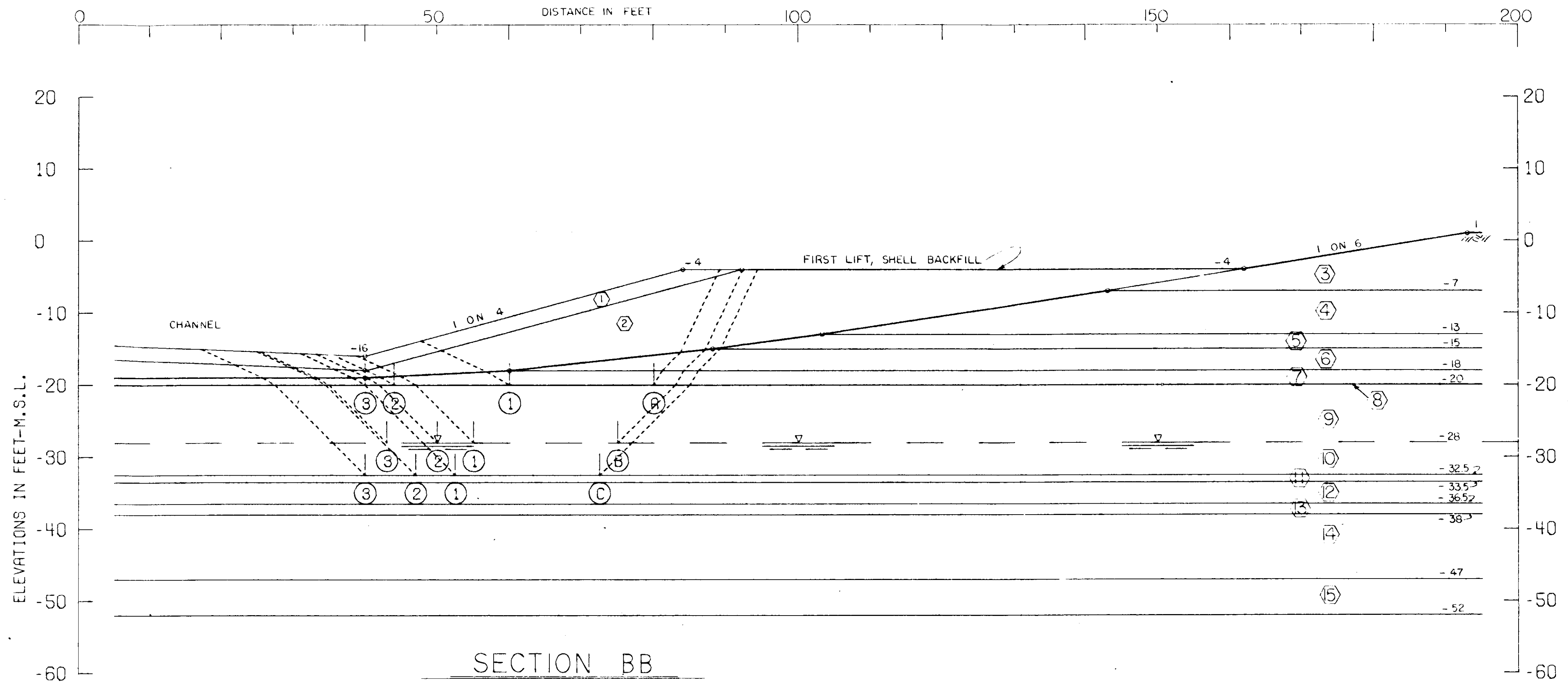
NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

SEE PLATE III-10 FOR LOCATION OF SEC. AA

NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SECTION AA
SHELL CORE EXCAVTION STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



SECTION BB

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	RIPRAP	125.0	125.0	0.	0.	0.	0.	40.0
2	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
3	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.
4	CH	102.0	102.0	100.0	100.0	100.0	100.0	0.
5	SM	117.0	117.0	0.	0.	0.	0.	30.0
6	CH	102.0	102.0	150.0	150.0	150.0	150.0	0.
7	SM	117.0	117.0	0.	0.	0.	0.	30.0
8		0.	0.	0.	0.	150.0	150.0	0.
9	CH	102.0	102.0	214.0	214.0	278.0	278.0	0.
10	CH	40.0	40.0	314.0	314.0	350.0	350.0	0.
11	SM	55.0	55.0	0.	0.	0.	0.	30.0
12	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
13	SM	55.0	55.0	0.	0.	0.	0.	30.0
14	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
15	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT 1 = VERT 2 = BORING 1-SEU

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A 1	-20.00	7819	3000	8708	12519	3105	19527	9414	2.074
A 2	-20.00	7819	5400	3060	12519	1083	16279	11436	1.423
A 3	-20.00	7819	6000	3070	12519	1032	16889	11487	1.470
B 1	-28.00	10908	5560	6997	27611	10552	23466	17059	1.376
B 2	-28.00	10908	6950	6424	27611	9228	24283	18383	1.321
B 3	-28.00	10908	8896	6838	27611	8237	26642	19374	1.375
C 1	-32.50	13540	7000	9320	38083	16172	29860	21911	1.363
C 2	-32.50	13540	8925	9699	38083	14928	32165	23154	1.389
C 3	-32.50	13540	11375	10211	38083	14684	35127	23399	1.501

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

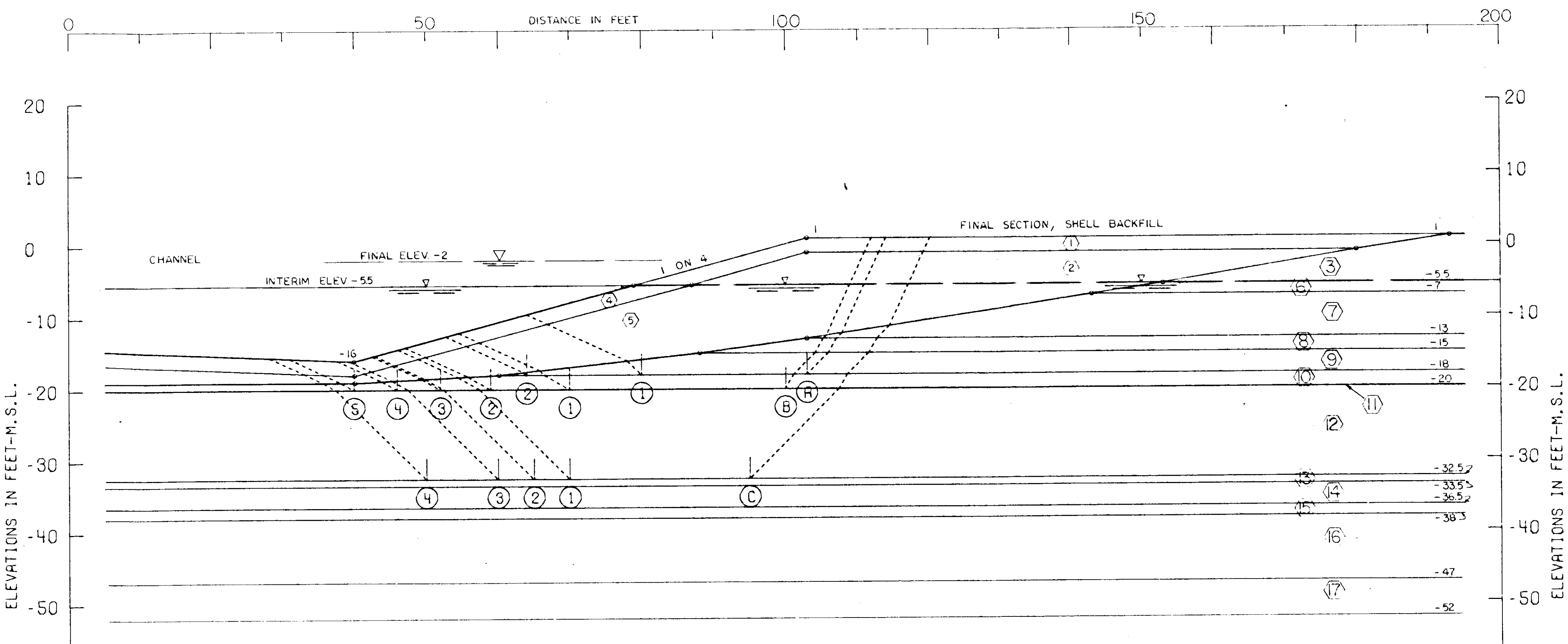
SEE PLATE III-10 FOR LOCATION OF SECTION BB

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
SECTION BB
INTERIM BACKFILL STABILITY (Q)

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

OCTOBER [illegible] FILE No. H-2-25048



SECTION CC

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	RIPRAP	125.0	125.0	0.	0.	0.	0.	40.0
2	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
3	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0.
4	RIPRAP	63.0	63.0	0.	0.	0.	0.	40.0
5	SHELL	30.0	30.0	0.	0.	0.	0.	40.0
6	CH(O)	16.0	16.0	150.0	150.0	150.0	150.0	0.
7	CH	40.0	40.0	100.0	100.0	100.0	100.0	0.
8	SM	55.0	55.0	0.	0.	0.	0.	30.0
9	CH	40.0	40.0	150.0	150.0	150.0	150.0	0.
10	SM	55.0	55.0	0.	0.	0.	0.	30.0
11		0.	0.	0.	0.	150.0	150.0	0.
12	CH	40.0	40.0	250.0	250.0	350.0	350.0	0.
13	SM	55.0	55.0	0.	0.	0.	0.	30.0
14	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
15	SM	55.0	55.0	0.	0.	0.	0.	30.0
16	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
17	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT. 1 = VERT. 2 = BORING 1-SEU

ASSUMED FAILURE NO.		ASSUMED SURFACE ELEV.	RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
			R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A	1	-18.00	8178	3450	5686	13225	2253	17314	10971	1.578
A	2	-18.00	8178	5850	3366	13225	1025	17394	12199	1.426
B	1	-20.00	9807	4500	5392	15377	2031	19699	13346	1.476
B	2	-20.00	9807	6150	3447	15377	1217	19404	14160	1.370
B	3	-20.00	9807	7200	2297	15377	815	19304	14562	1.326
B	4	-20.00	9807	8100	1528	15377	544	19436	14833	1.310
B	5	-20.00	9807	8926	1472	15377	491	20205	14886	1.357
C	1	-32.50	15549	8750	9432	33252	9677	33732	23575	1.431
C	2	-32.50	15549	10500	8622	33252	8811	34671	24441	1.419
C	3	-32.50	15549	12250	7922	33252	7979	35722	25273	1.413
C	4	-32.50	15549	15750	7796	33252	6748	39096	26504	1.475

NOTES

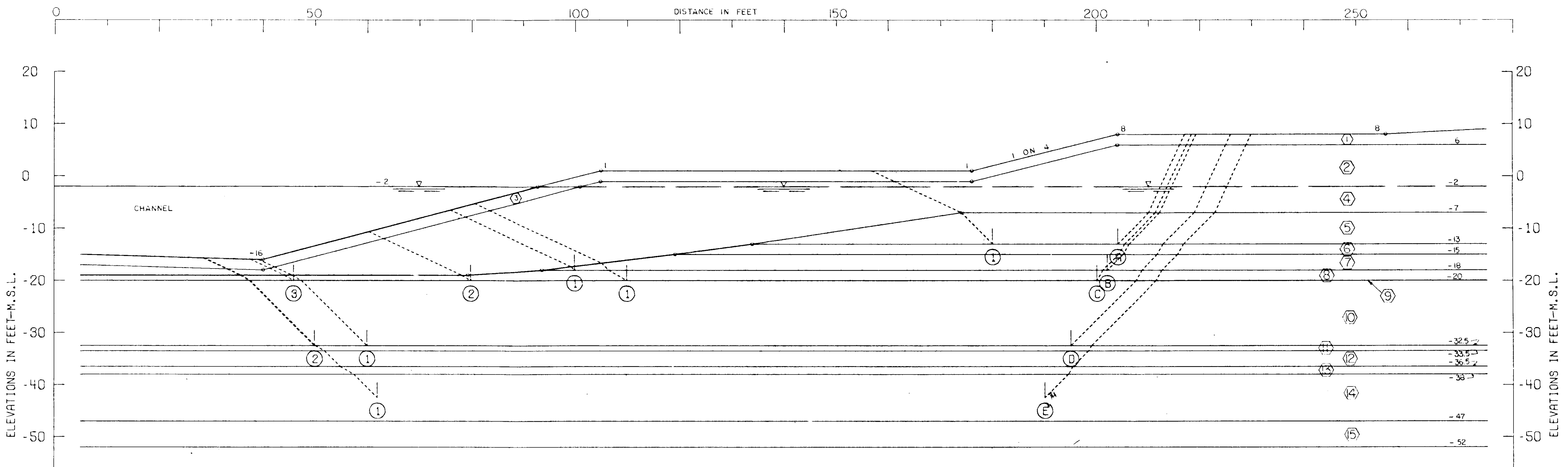
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

SEE PLATE III-10 FOR LOCATION OF SECTION CC

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SECTION CC
FINAL BACKFILL STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048



SECTION DD

GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III-24 AND III-25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	RIPRAP	125.0	125.0	0.	0.	0.	0.	40.0
2	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
3	RIPRAP	63.0	63.0	0.	0.	0.	0.	40.0
4	SHELL	30.0	30.0	0.	0.	0.	0.	40.0
5	CH	40.0	40.0	100.0	100.0	100.0	100.0	0.
6	SM	55.0	55.0	0.	0.	0.	0.	30.0
7	CH	40.0	40.0	150.0	150.0	150.0	150.0	0.
8	SM	55.0	55.0	0.	0.	0.	0.	30.0
9		0.	0.	0.	0.	150.0	150.0	0.
10	CH	40.0	40.0	250.0	250.0	350.0	350.0	0.
11	SM	55.0	55.0	0.	0.	0.	0.	30.0
12	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
13	SM	55.0	55.0	0.	0.	0.	0.	30.0
14	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
15	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

VERT 1 - VERT. 2 - BORING 1-SEU

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	-13.00	9424	2400	10665	18047	6484	22490	11562	1.945
B 1	-18.00	12233	15300	13220	25502	4066	40754	21436	1.901
C 1	-20.00	14343	13500	19495	28620	7186	47338	21434	2.209
C 2	-20.00	14343	18000	8794	28620	2664	41138	25956	1.585
C 3	-20.00	14343	23100	1558	28620	532	39002	28087	1.389
D 1	-32.50	20698	47250	7966	52516	7714	75915	44802	1.694
D 2	-32.50	20698	50750	7718	52516	6652	79166	45863	1.726
E 1	-42.50	31461	76800	19662	75432	17346	127924	58085	2.202

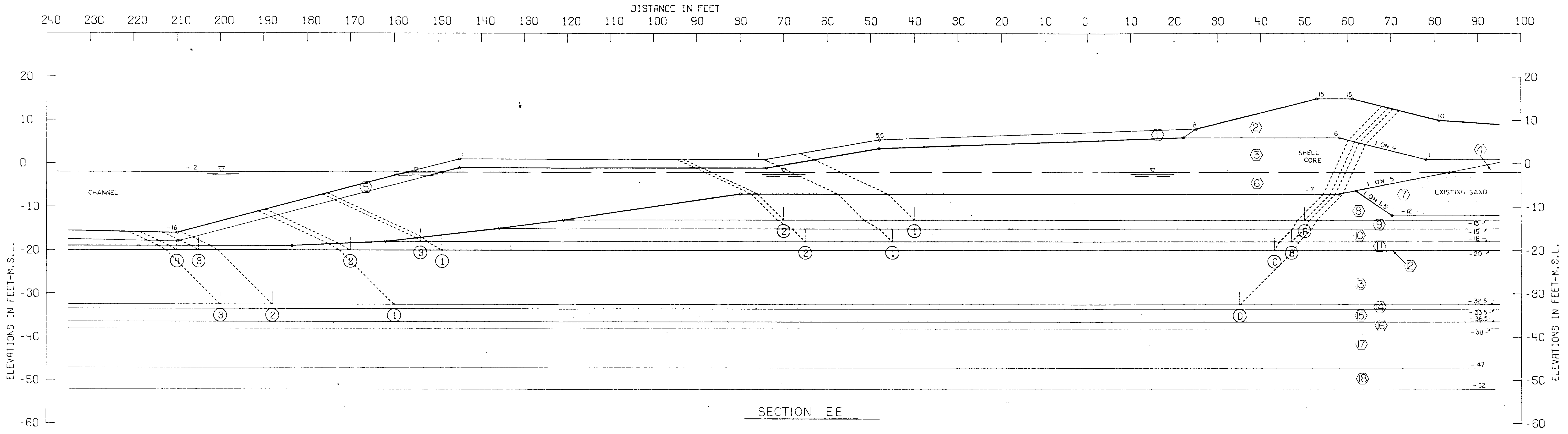
NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

SEE PLATE III-10 FOR LOCATION OF SECTION DD

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B-1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SECTION DD
I-WALL LEVEE STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1972 FILE NO. H-2-25048



SECTION EE

GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTHS, AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS. SEE BORING DATA PLATES III-24 AND III-25

SHEAR STRENGTHS BETWEEN VERTICALS 1 AND 2 WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	RIPRAP	125.0	125.0	0.	0.	0.	0.	40.0
2	CH	100.0	100.0	200.0	200.0	200.0	200.0	0.
3	SHELL	92.0	92.0	0.	0.	0.	0.	40.0
4	SP	117.0	117.0	0.	0.	0.	0.	30.0
5	RIPRAP	63.0	63.0	0.	0.	0.	0.	40.0
6	SHELL	30.0	30.0	0.	0.	0.	0.	40.0
7	SP	55.0	55.0	0.	0.	0.	0.	30.0
8	CH	40.0	40.0	100.0	100.0	100.0	100.0	0.
9	SM	55.0	55.0	0.	0.	0.	0.	30.0
10	CH	40.0	40.0	150.0	150.0	150.0	150.0	0.
11	SM	55.0	55.0	0.	0.	0.	0.	30.0
12		0.	0.	0.	0.	150.0	150.0	0.
13	CH	40.0	40.0	250.0	250.0	350.0	350.0	0.
14	SM	55.0	55.0	0.	0.	0.	0.	30.0
15	CH	40.0	40.0	400.0	400.0	425.0	425.0	0.
16	SM	55.0	55.0	0.	0.	0.	0.	30.0
17	CH	40.0	40.0	525.0	525.0	600.0	600.0	0.
18	ML	55.0	55.0	200.0	200.0	200.0	200.0	12.0

ASSUMED FAILURE NO.	SURFACE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A 1	-13.00	18302	9000	20337	32999	11565	47640	21433	2.223
A 2	-13.00	18302	12000	10665	32999	6428	40968	26571	1.542
B 1	-18.00	21660	13800	19956	43058	15502	55416	27556	2.011
B 2	-18.00	21660	16800	14984	43058	11125	53444	31933	1.674
B 3	-18.00	21660	30150	11170	43058	3531	62980	39527	1.593
C 1	-20.00	24850	28800	14608	47273	5273	68258	42000	1.625
C 2	-20.00	24850	31950	8517	47273	2698	65317	44574	1.465
C 3	-20.00	24850	37198	1480	47273	507	63529	46765	1.358
C 4	-20.00	24850	37869	1391	47273	468	64111	46805	1.370
D 1	-32.50	30617	68250	14105	76199	12858	112972	63340	1.784
D 2	-32.50	30617	78050	8249	76199	7999	116917	68200	1.714
D 3	-32.50	30617	82250	7673	76199	6639	120541	69560	1.733

VERT 1 = VERT 2 = BORING 1-SEU

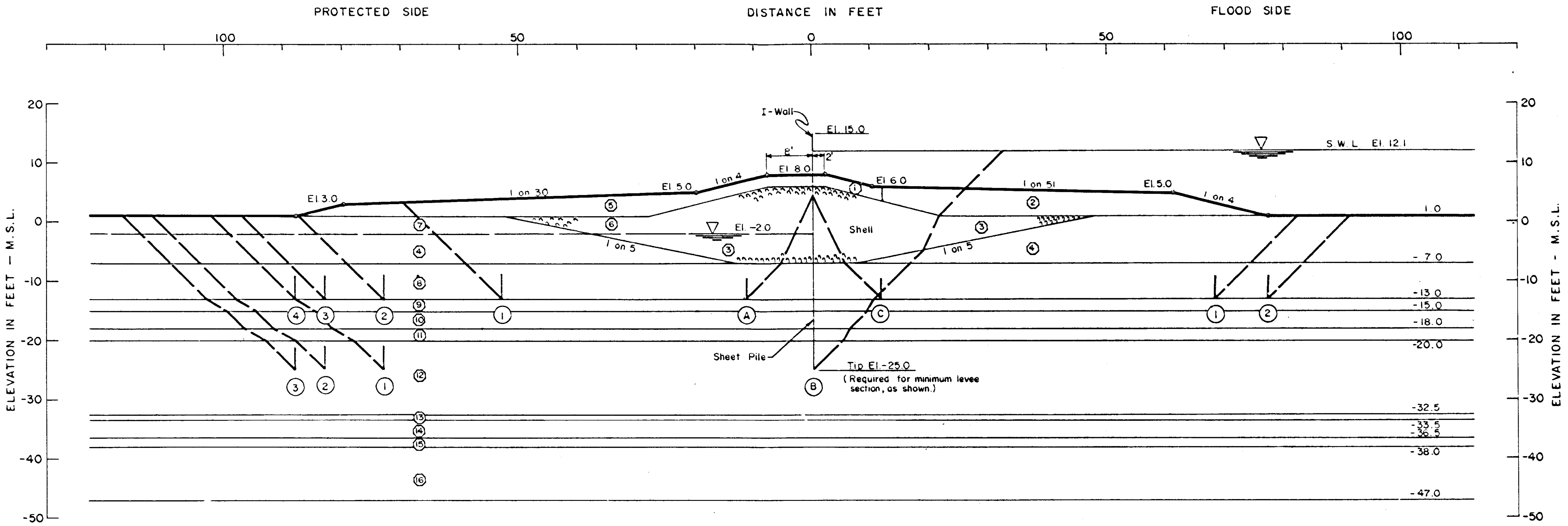
NOTES

ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

SEE PLATE III-10 FOR LOCATION OF SECTION EE

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SECTION EE
TIE IN LEVEE STABILITY (Q)
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



SECTION FF

STRATUM NO.	SOIL TYPE	EFFECTIVE UNIT WT. P.C.F.		C - UNIT COHESION - P.S.F.				FRICTION ANGLE DEGREES
				CENTER OF STRATUM		BOTTOM OF STRATUM		
		VERT. 1	VERT. 2	VERT. 1	VERT. 2	VERT. 1	VERT. 2	
1	RIPRAP	63.0	63.0	0	0	0	0	40.0
2	CH	40.0	40.0	100.0	100.0	100.0	100.0	0
3	SHELL	30.0	30.0	0	0	0	0	40.0
4	CH(O)	16.0	16.0	150.0	150.0	150.0	150.0	0
5	CH	100.0	100.0	100.0	100.0	100.0	100.0	0
6	SHELL	92.0	92.0	0	0	0	0	40.0
7	CH(O)	78.0	78.0	150.0	150.0	150.0	150.0	0
8	CH	40.0	40.0	100.0	100.0	100.0	100.0	0
9	SM	55.0	55.0	0	0	0	0	30.0
10	CH	40.0	40.0	150.0	150.0	150.0	150.0	0
11	SM	55.0	55.0	0	0	0	0	30.0
12	CH	40.0	40.0	250.0	250.0	350.0	350.0	0
13	SM	55.0	55.0	0	0	0	0	30.0
14	CH	40.0	40.0	400.0	400.0	425.0	425.0	0
15	SM	55.0	55.0	0	0	0	0	30.0
16	CH	40.0	40.0	525.0	525.0	600.0	600.0	0

Vert 1 = Vert 2 = Boring 1- SEU

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV	R_A	R_B	R_P	D_A	D_P	RESISTING	DRIVING	
A 1	-13.00	8879	4150	4060	172 64	8200	17089	9064	1.885
A 2	-13.00	8879	6150	3600	172 64	6225	18629	11039	1.689
A 3	-13.00	8879	7150	3600	172 64	4650	19629	12614	1.556
A 4	-13.00	8879	7650	3600	172 64	4325	20129	12939	1.556
A 3'	-13.00	7511	7650	3600	172 64	5012	18761	12252	1.531
B 1	-25.00	9214	16790	12788	448 41	15960	38792	28881	1.343
B 2	-25.00	9214	19090	11869	448 41	14501	40173	30340	1.324
B 2'	-25.00	9214	19090	10369	448 41	15970	38673	28871	1.339
B 3	-25.00	9214	20240	11869	448 41	14201	41323	30640	1.349
C 1	-13.00	8555	5650	3600	160 76	5225	17805	10851	1.641
C 2	-13.00	8555	6550	3600	160 76	4325	18705	11751	1.592
C 2'	-13.00	7564	6550	3600	160 76	4325	17714	11751	1.507

NOTES:

Failure surfaces A1, A2, A3, A4: Water table at El. -2.0.

Failure surface A3': Water table at El. 2.0.

Failure surfaces B1, B2, B3: Water table at El. -2.0 on protected side, El. 12.1 on flood side (active wedge).

Failure surface B2': Water table at El. 2.0 on protected side, El. 12.1 on flood side (active wedge)

Failure surface C1, C2: Water table at El. -2.0 } Non-Flood Condition

Failure surface C2': Water table at El. 1.0. } Non-Flood Condition

See Plate III-10 for location of section FF.

NEW ORLEANS TO VENICE, LA

DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN

REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

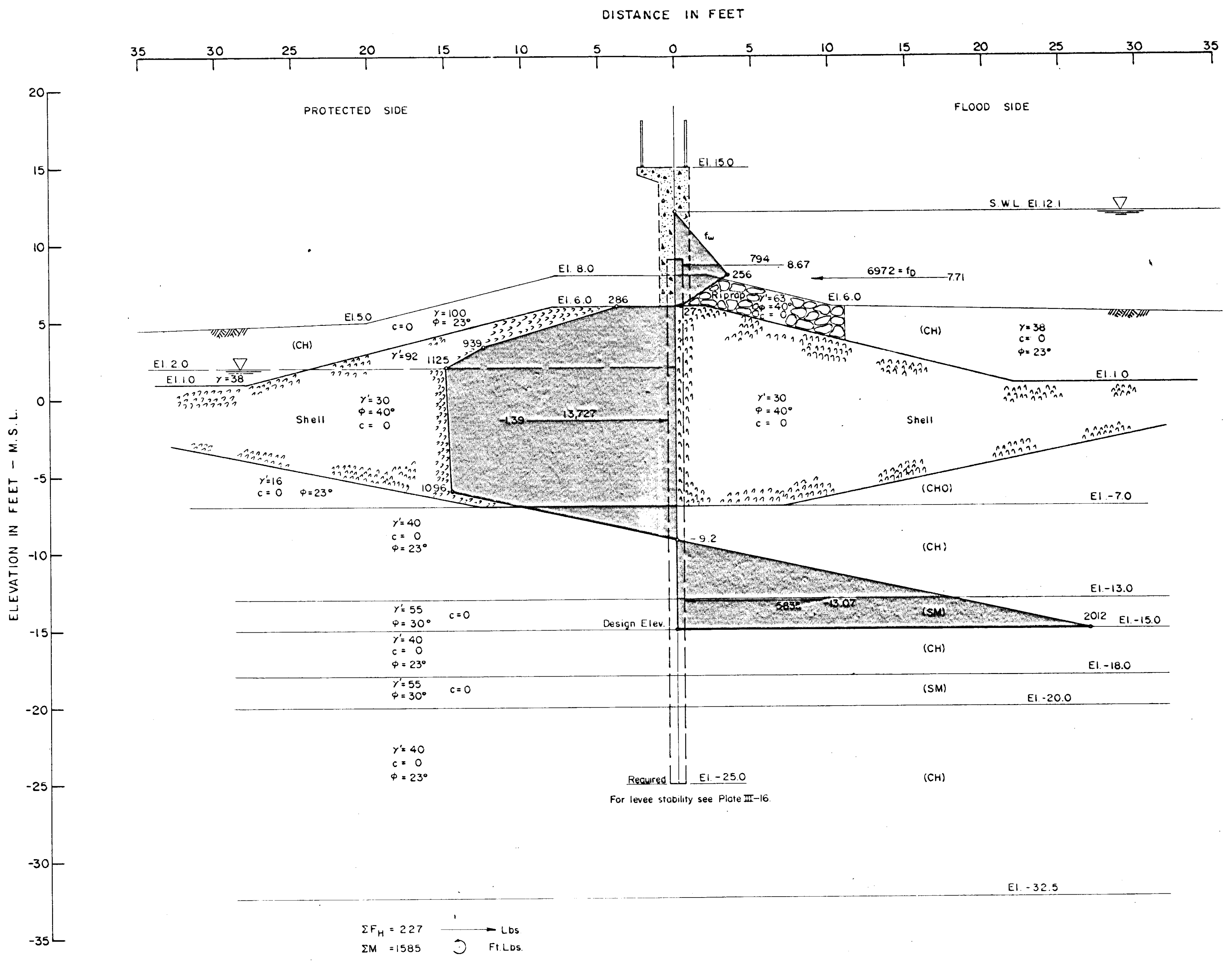
SECTION F.F

STABILITY (Q)

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS

CORPS OF ENGINEERS

OCTOBER 1970 FILE NO H-2-25048



(S) CASE F.S. = 1.25
WITH WAVE

GENERAL NOTES

(S) - Shear strength case governed for design. Stability analysis by the method of planes with surfaces $45 \pm \frac{\phi_d}{2}$ and F.S. = 1.25 applied to shear strength of the soil.

ϕ_A - Available angle of internal friction in degrees.

ϕ_D - Developed angle of internal friction $= \tan^{-1}\left(\frac{\tan \phi_A}{F.S.}\right)$

C_A - Unit cohesion available.

C_D - Unit cohesion developed $= C_A \div F.S.$

(S) - Consolidated-drained shear strength of soil. For undisturbed shear test data see plates:

f_ω - Net lateral water pressure. (Water pressure from waves effective to top of impervious clay layer).

ΣF_H - Summation of horizontal forces.

ΣM - Summation of moments about the sheet pile tip.

γ, γ' - Unit weights P.C.F.

S.W.L. - Still water level.

f_D - Dynamic wave force, effective to top of impervious clay layer. (El. -7.0)

NOTE:

Any uplift pressures developed in shell core on the protected side will be relieved through shell backfill near T-wall and flat area. (El. 1.0) See plate III-10.

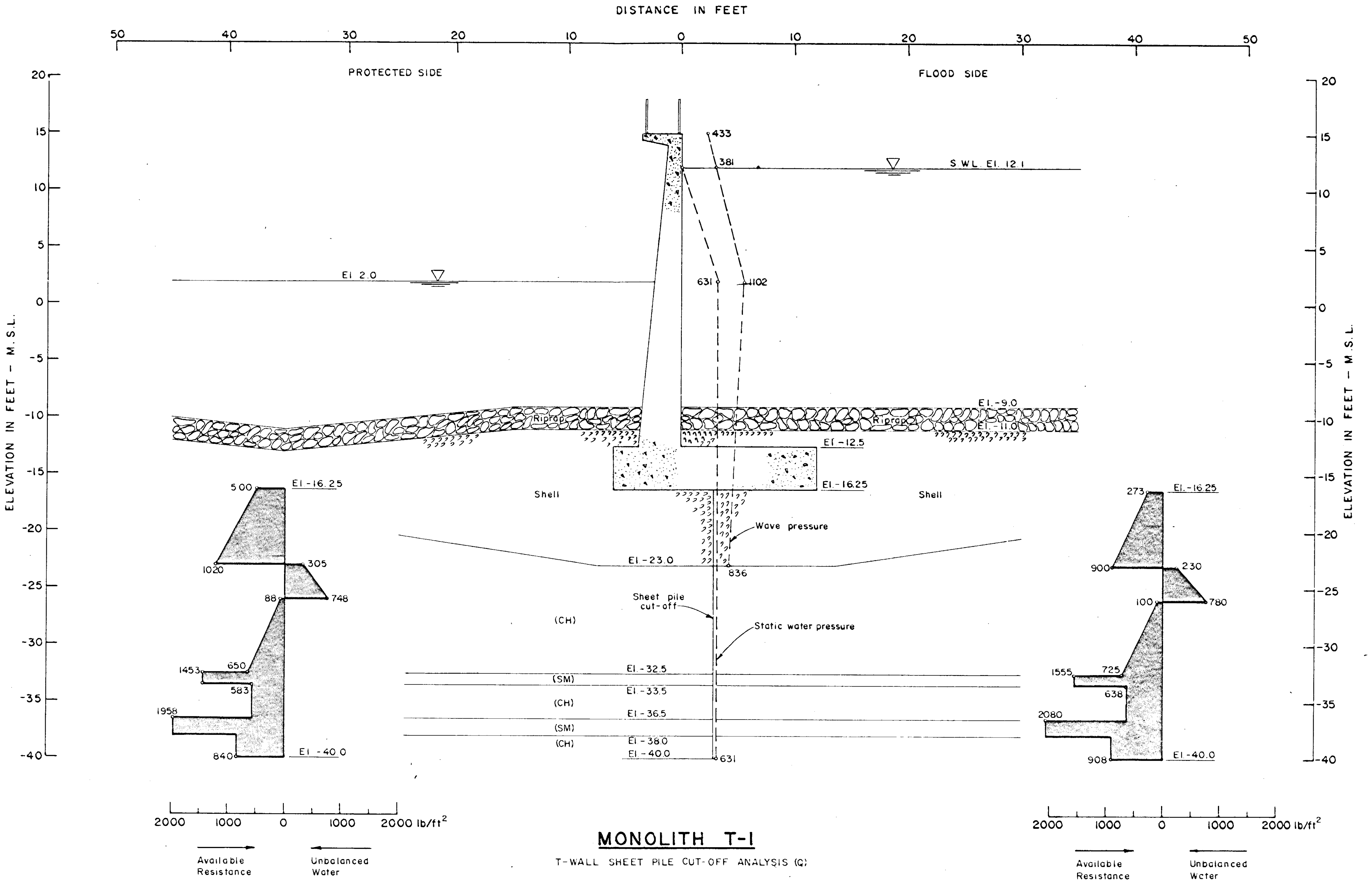
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
CANTILEVER SHEET PILE FLOODWALL
STABILITY (S)

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

OCTOBER 1970 FILE NO. H-2-25048

PLATE III-17



MONOLITH T-1

T-WALL SHEET PILE CUT-OFF ANALYSIS (G)

F.S. = 1.30
NO WAVE

F.S. = 1.25
WITH WAVE

NOTES:
Resistance due to bearing piles supporting T-Wall was neglected.
Available resistance shown is that in excess of the resistance developed to balance the water load.

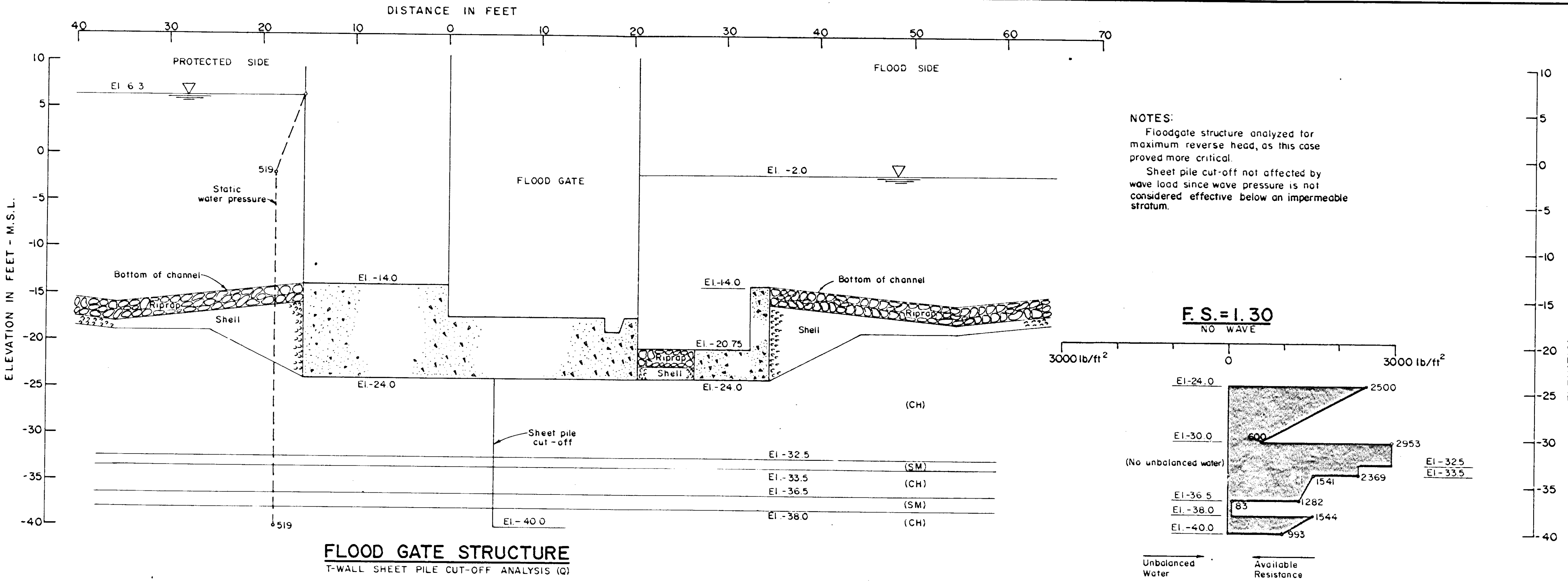
NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
UNBALANCED WATER LOAD ANALYSIS
MONOLITH T-1

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

OCTOBER 1970 FILE NO. H-2-25048

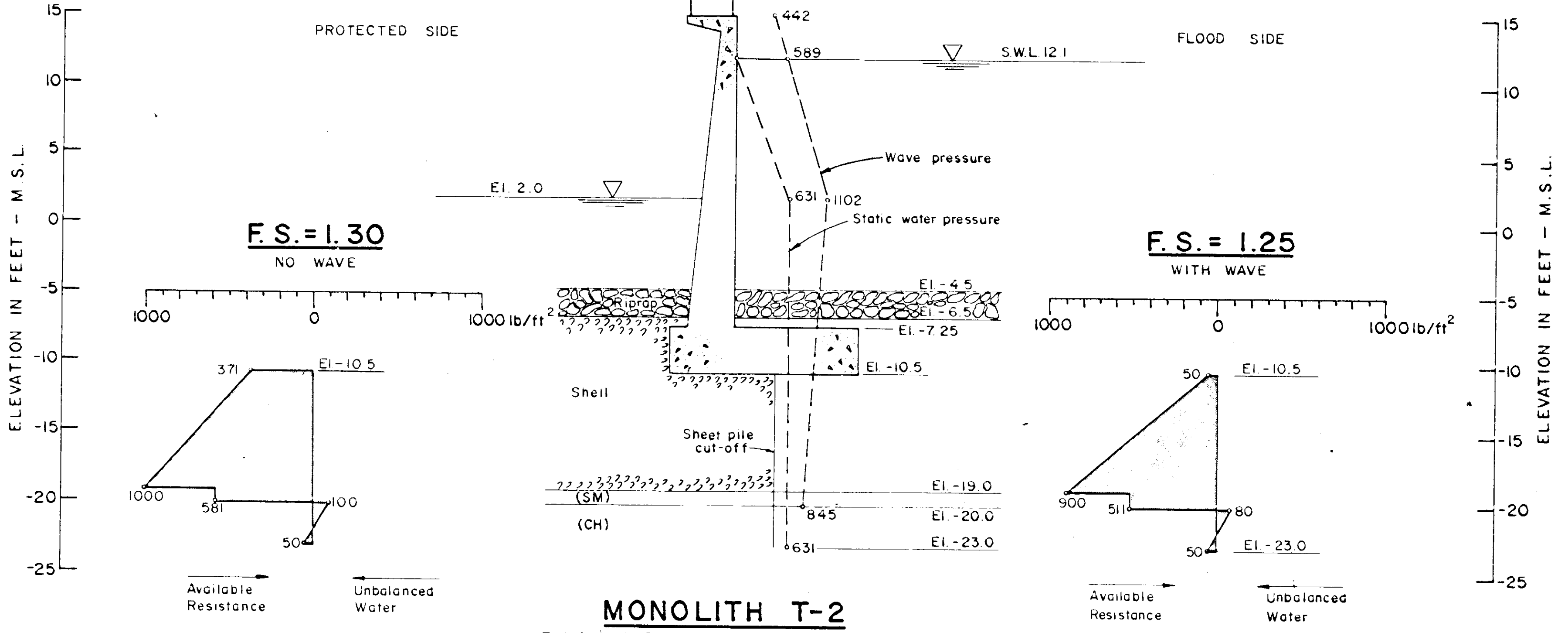
PLATE III-18



FLOOD GATE STRUCTURE

T-WALL SHEET PILE CUT-OFF ANALYSIS (Q)

NOTES:
Floodgate structure analyzed for maximum reverse head, as this case proved more critical.
Sheet pile cut-off not affected by wave load since wave pressure is not considered effective below an impermeable stratum.



MONOLITH T-2

T-WALL SHEET PILE CUT-OFF ANALYSIS (Q)

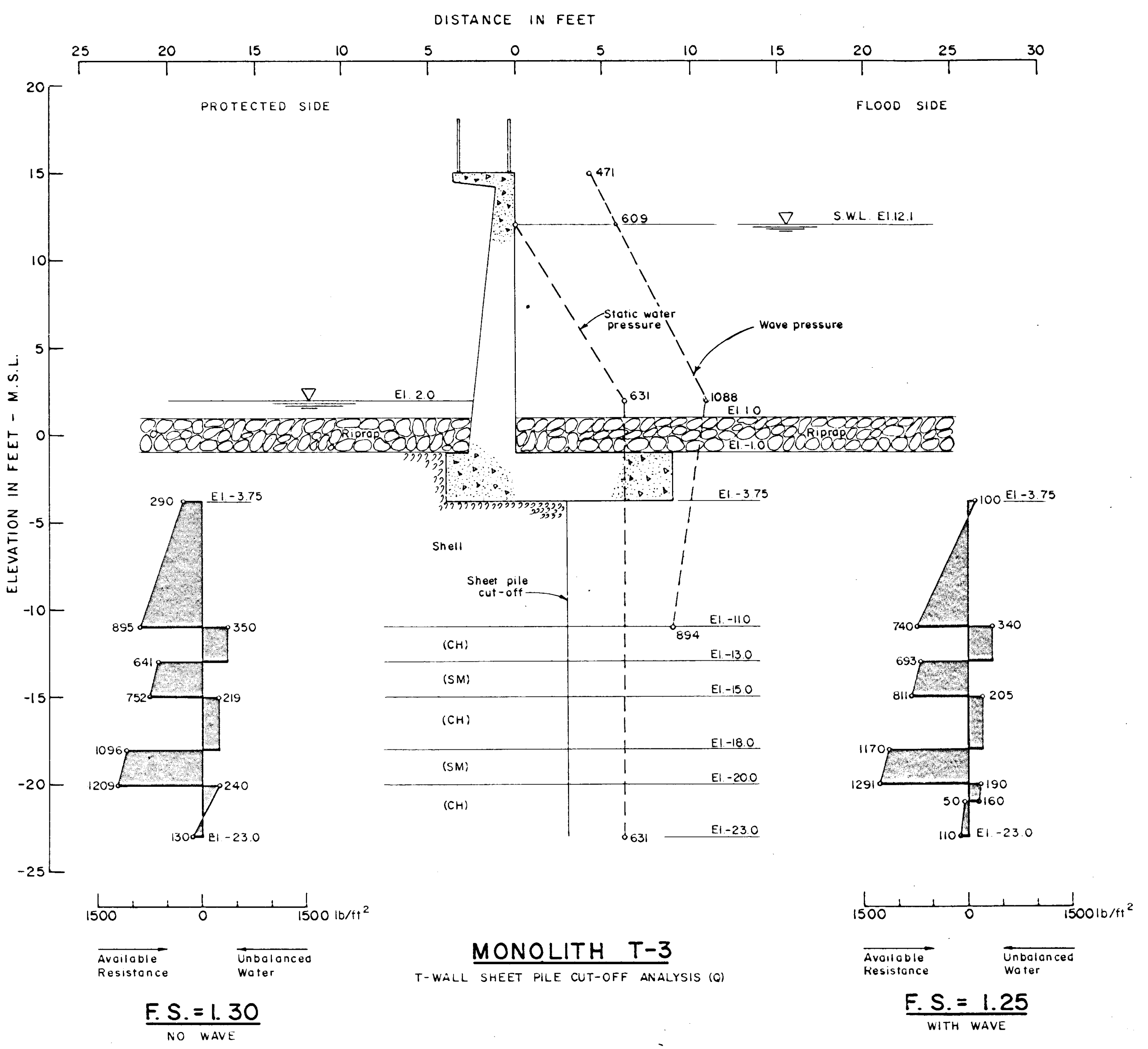
NOTES:
Resistance due to bearing piles supporting floodgate and T-Wall was neglected.
Available resistance shown is that in excess of the resistance developed to balance the water load.

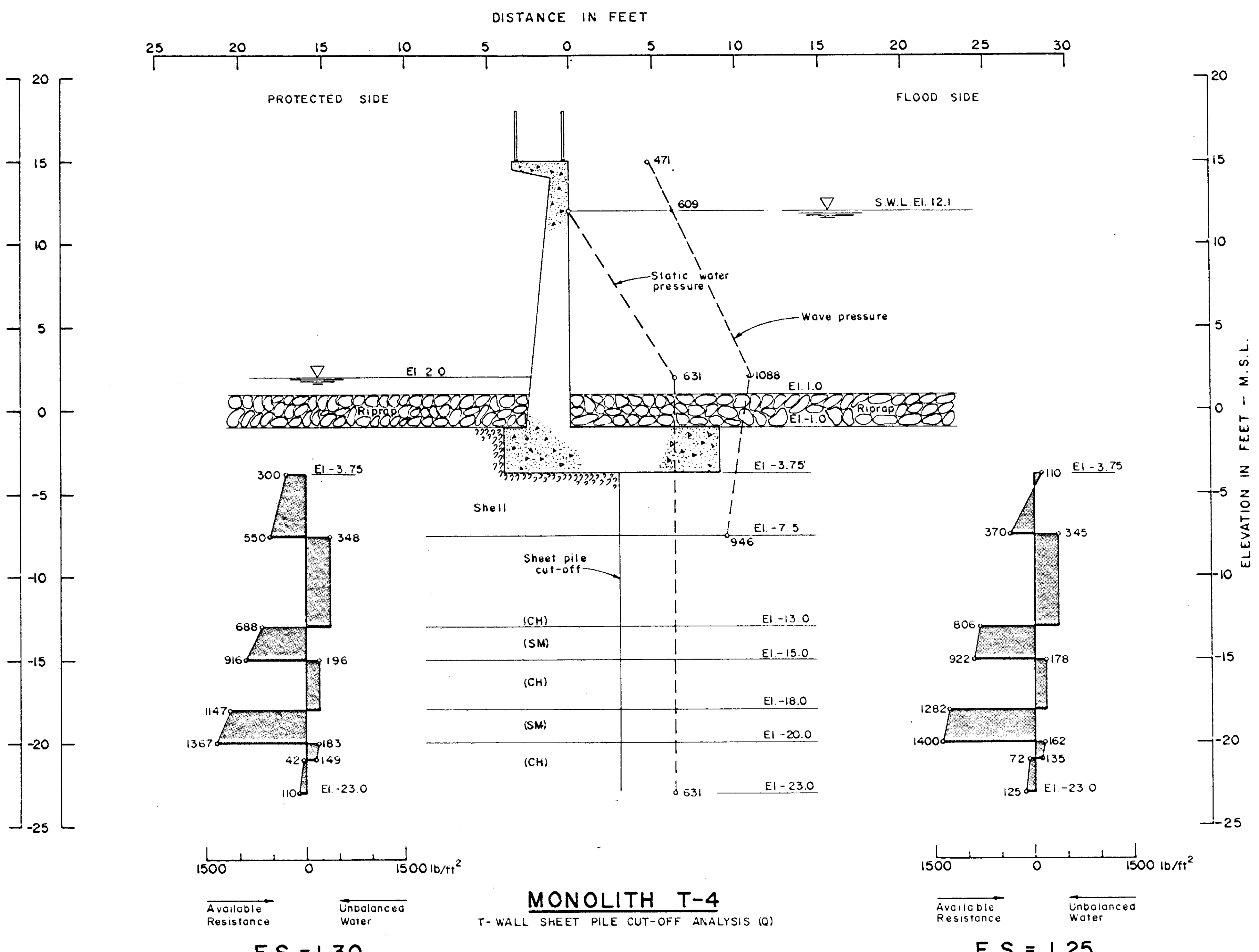
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
UNBALANCED WATER LOAD ANALYSIS
STRUCTURE AND T-WALL

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

OCTOBER 1970 — FILE NO. H-2-25048





NOTES:

Resistance due to bearing piles supporting T-Wall was neglected.

Available resistance shown is that in excess of the resistance developed to balance the water load.

NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
UNBALANCED WATER LOAD ANALYSIS
MONOLITH T-3 AND T-4

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

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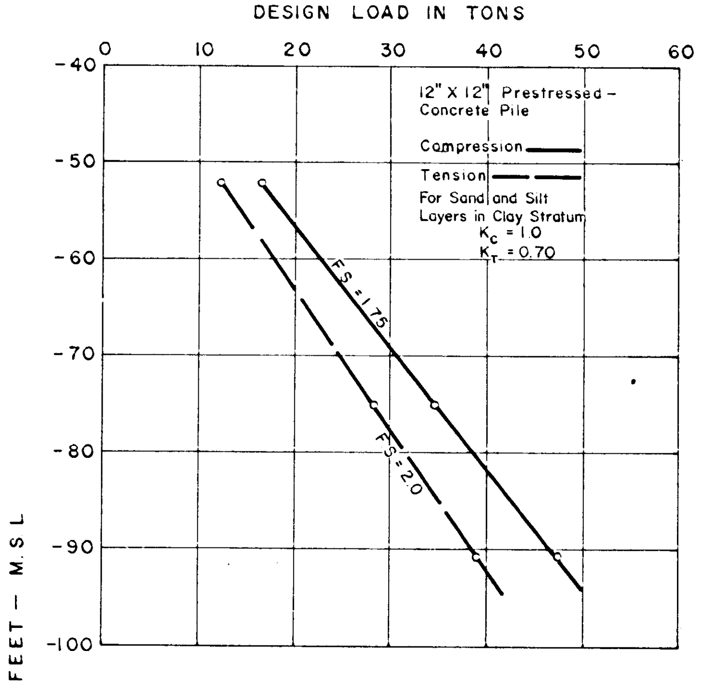
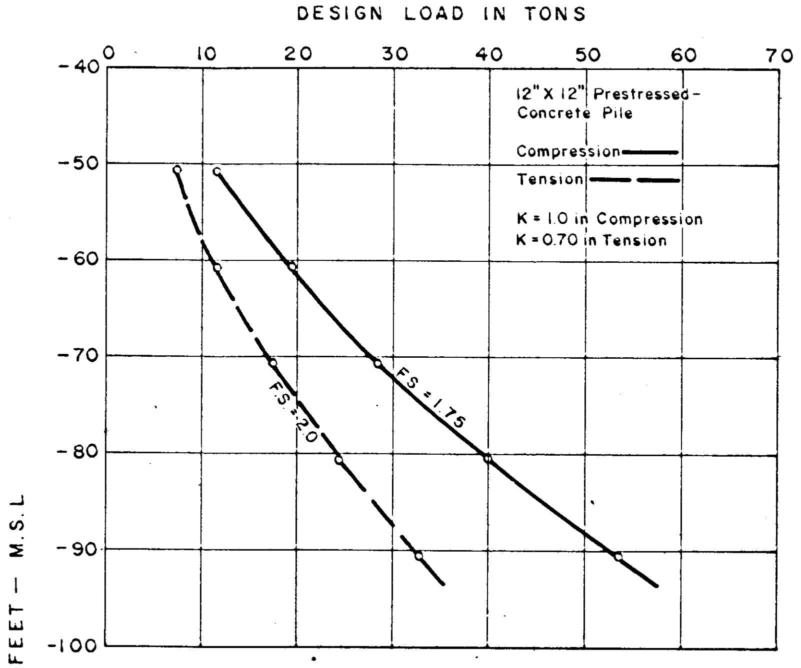
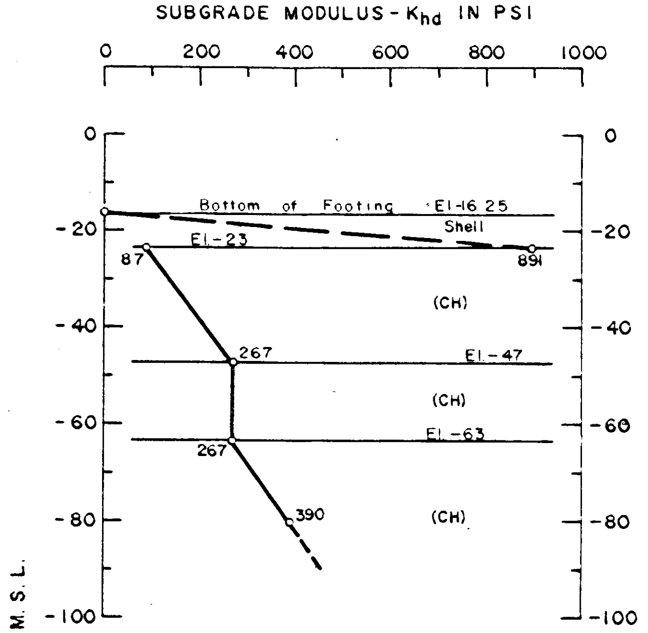


MONOLITH T-1
PILE DESIGN (Q)



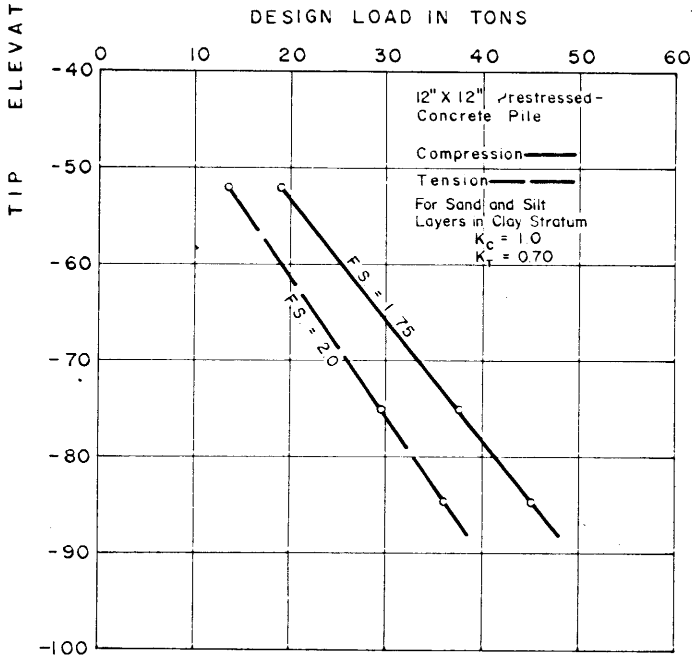
MONOLITH T-1
PILE DESIGN (S)



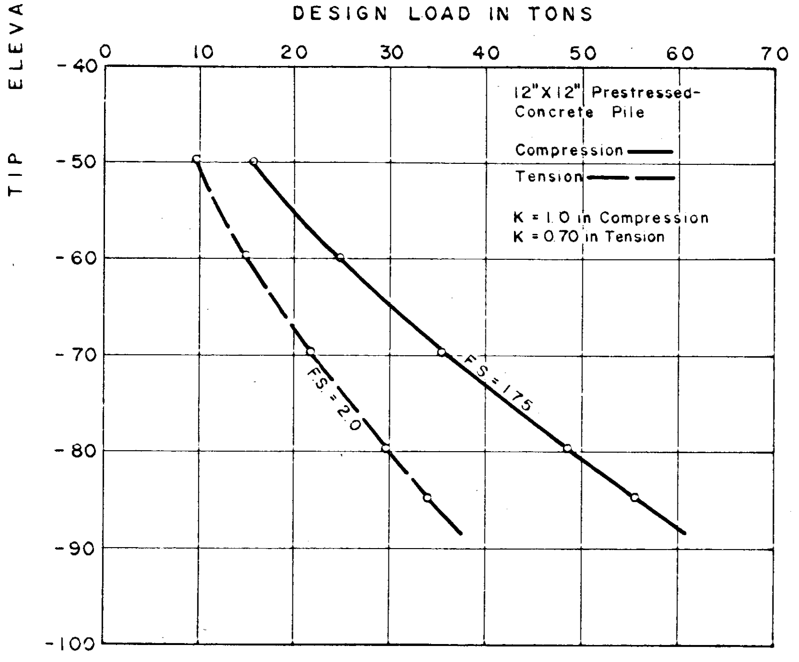
MONOLITH T-1
SUBGRADE MODULUS

GENERAL NOTES

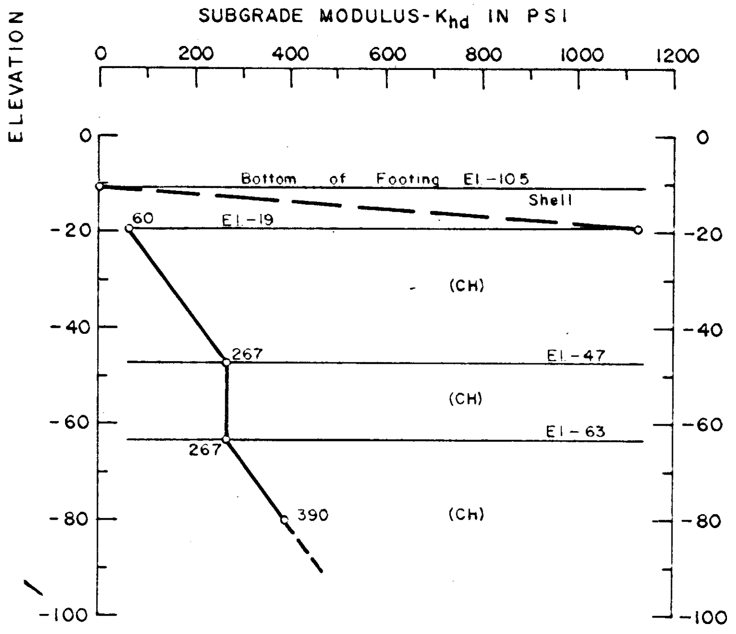
N_h — 22 PCI for shell
Δd_o — Thickness of shell (ft)
d — Projected pile diameter (in)
b — Unit conversion factor = 12"
K_o — 80 q_u (PSF)
q_u — Unconfined compressive Strength = 2C
C — Cohesion
K_{hd} — 0.4 $K_o d/b$ (for cohesive soils)
0.4 — Reduction factor for cyclic loading.
K_{hd} — 0.5 $N_h \Delta d_o d/b(12)$ [for noncohesive soils]
K_c — Lateral earth pressure Coefficient (Compression)
K_T — Lateral earth pressure Coefficient (Tension)
K — Conjugate stress ratio



MONOLITH T-2
PILE DESIGN (Q)



MONOLITH T-2
PILE DESIGN (S)



MONOLITH T-2
SUBGRADE MODULUS

NOTE:

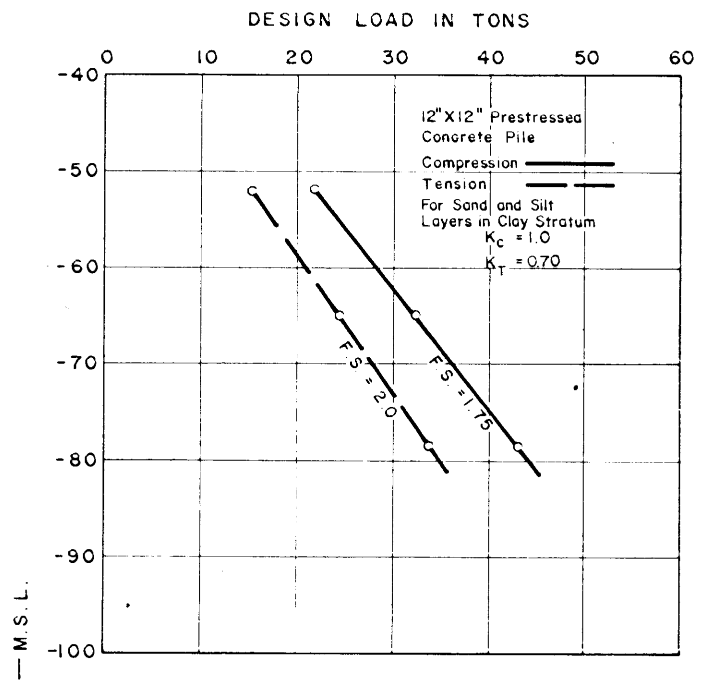
See plate III-22 (Pile Design, Monolith T-3, T-4, Floodgate) for illustration of calculations for pile design based on (S) strengths.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 — DETAIL DESIGN
REACH B1 — TROPICAL BEND TO FORT JACKSON

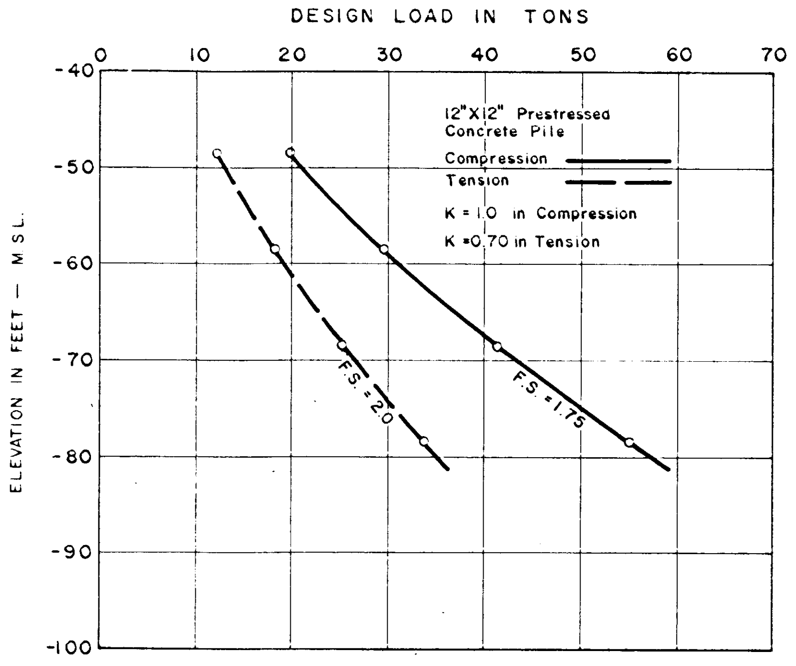
EMPIRE FLOODGATE
PILE DESIGN
MONOLITH T-1 & T-2

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

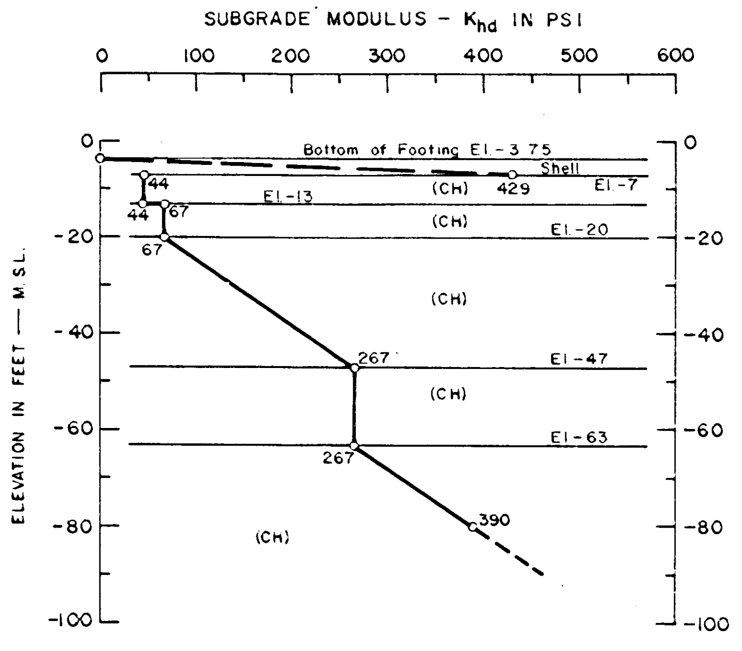
OCTOBER 1970 FILE NO. H-2-25048



MONOLITH T-3&T-4
PILE DESIGN (Q)



MONOLITH T-3 & T-4
PILE DESIGN (S)



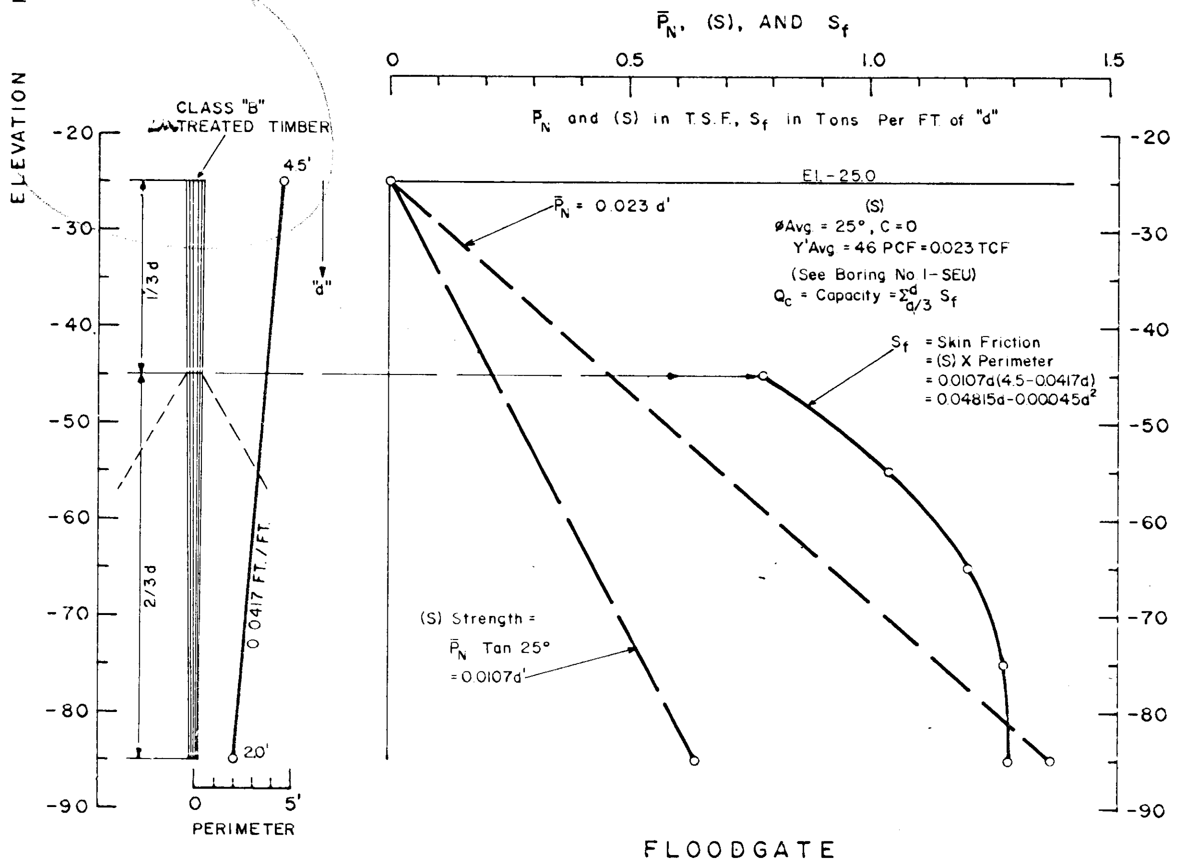
MONOLITH T-3 &T-4
SUBGRADE MODULUS

GENERAL NOTES

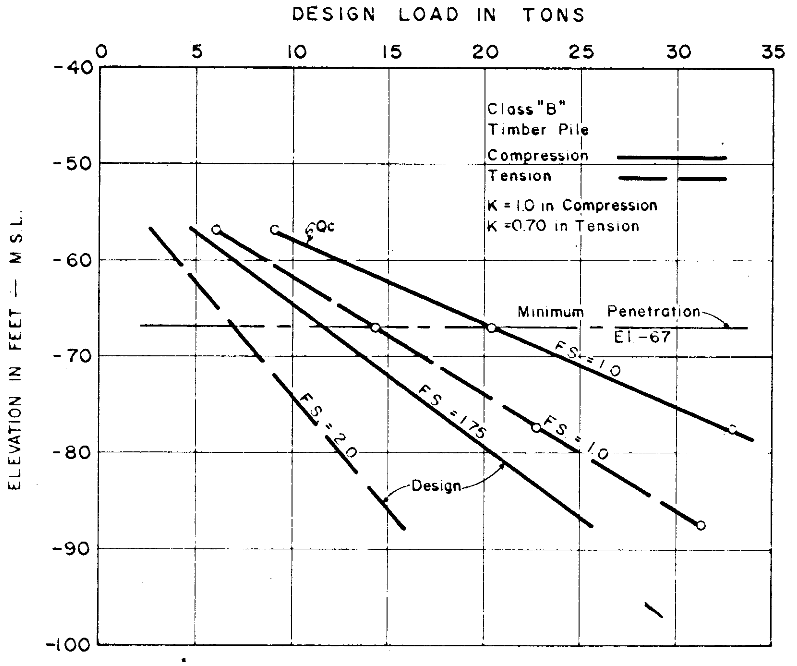
See plate III-21 onpile design, Monolith T-1 and T-2.

NOTES:

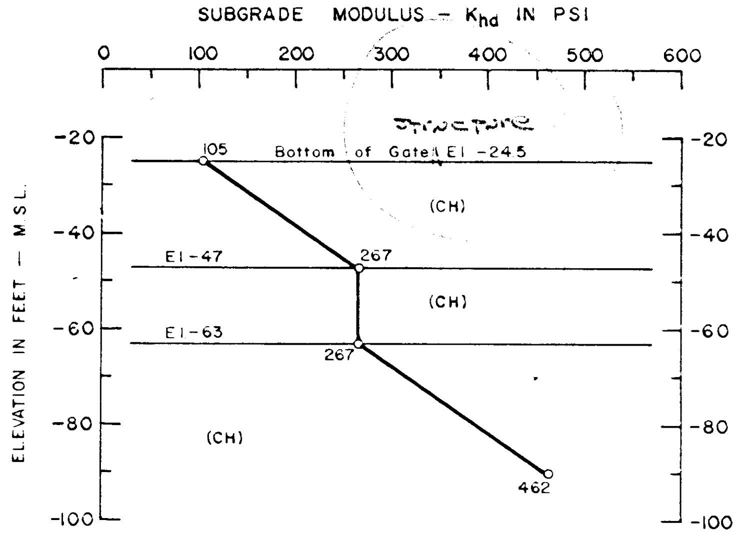
Settlement of structure due to consolidation is not a problem since the major loads are caused by hurricane water heads of insufficient duration for consolidation of the foundation clays to ensue.
See plate IV-8 for Tip elevation and location of Test Piles.



FLOODGATE
(S) CASE GOVERNED FOR DESIGN



FLOODGATE
PILE DESIGN (S)



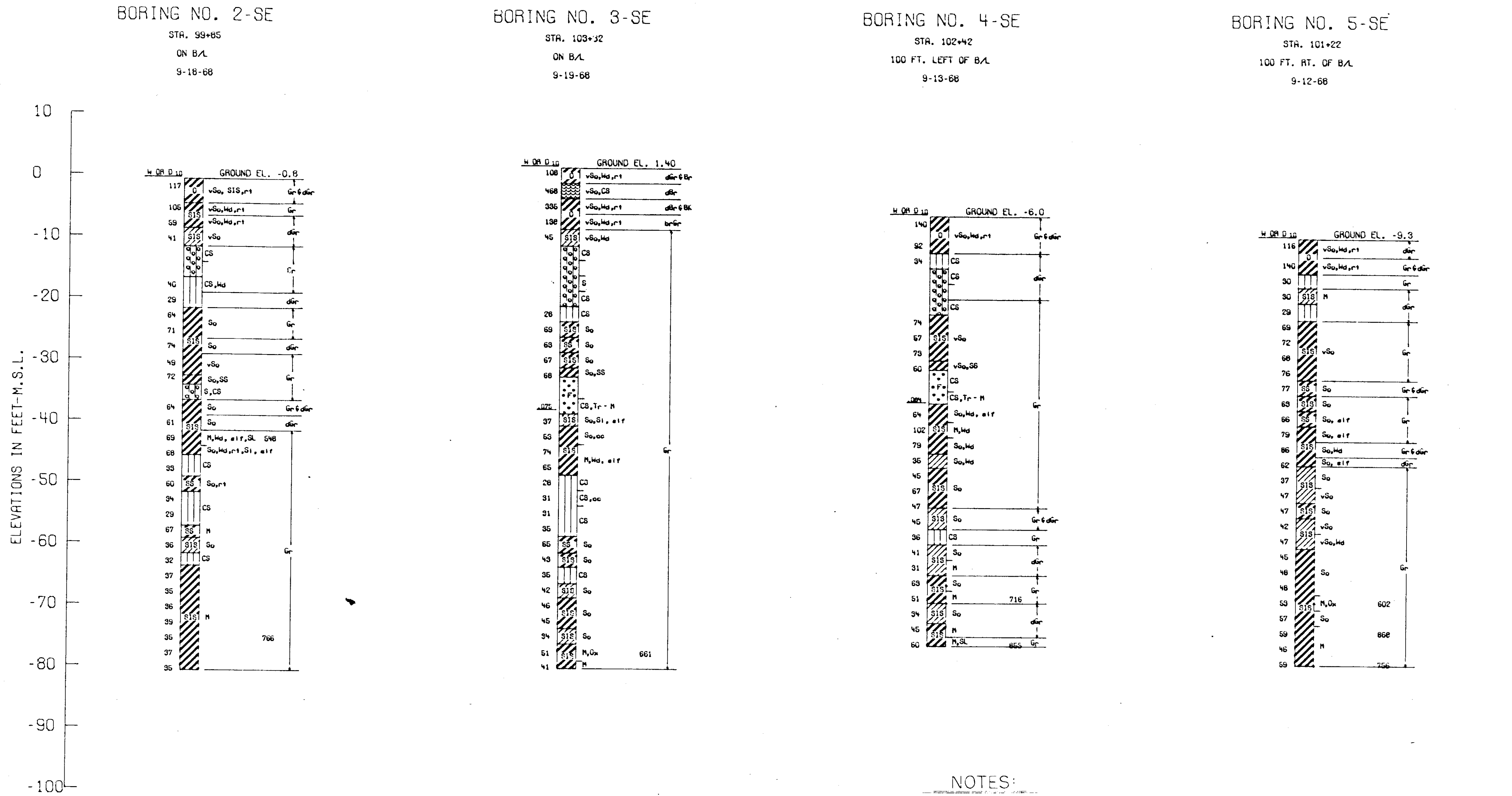
FLOODGATE
BEARING PILE SUBGRADE MODULUS

NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
PILE DESIGN
MONOLITH T-3 & T-4, FLOODGATE

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

OCTOBER 1970 FILE NO H-2-25048



NOTES:

GENERAL TYPE BORINGS TAKEN WITH A 1⅞" I.D. CORE BARREL SAMPLER.

FOR SOIL BORING LEGEND, SEE PLATE A.

FOR LOCATION OF BORINGS SEE Plate IV-1.

NEW ORLEANS TO VENICE, LA

DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN

REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

GENERAL TYPE BORINGS

Nos. 2-SE, 3-SE, 4-SE & 5-SE

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

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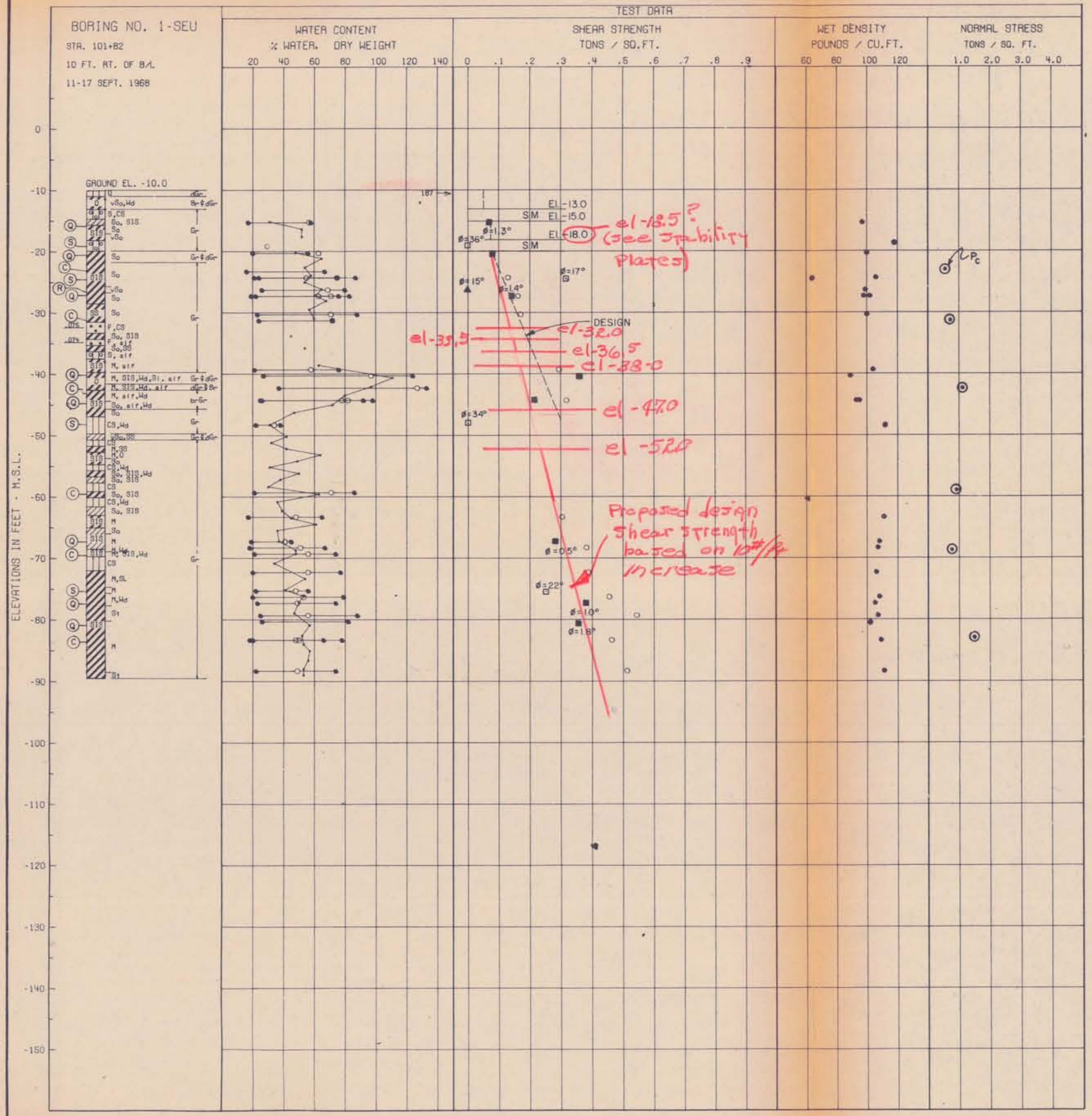
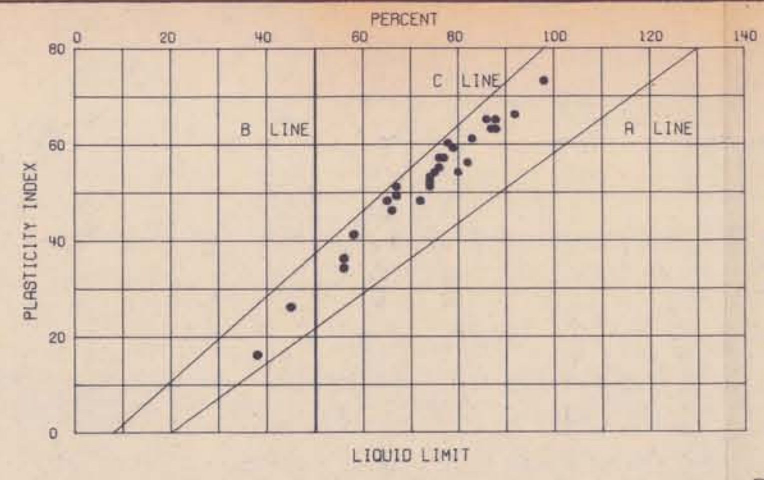
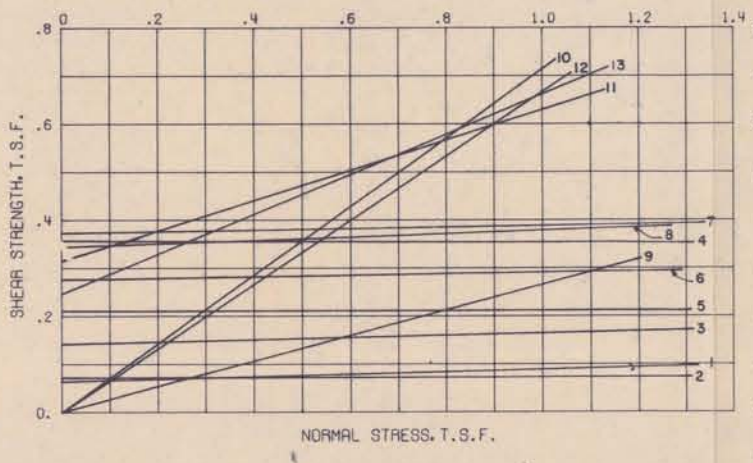


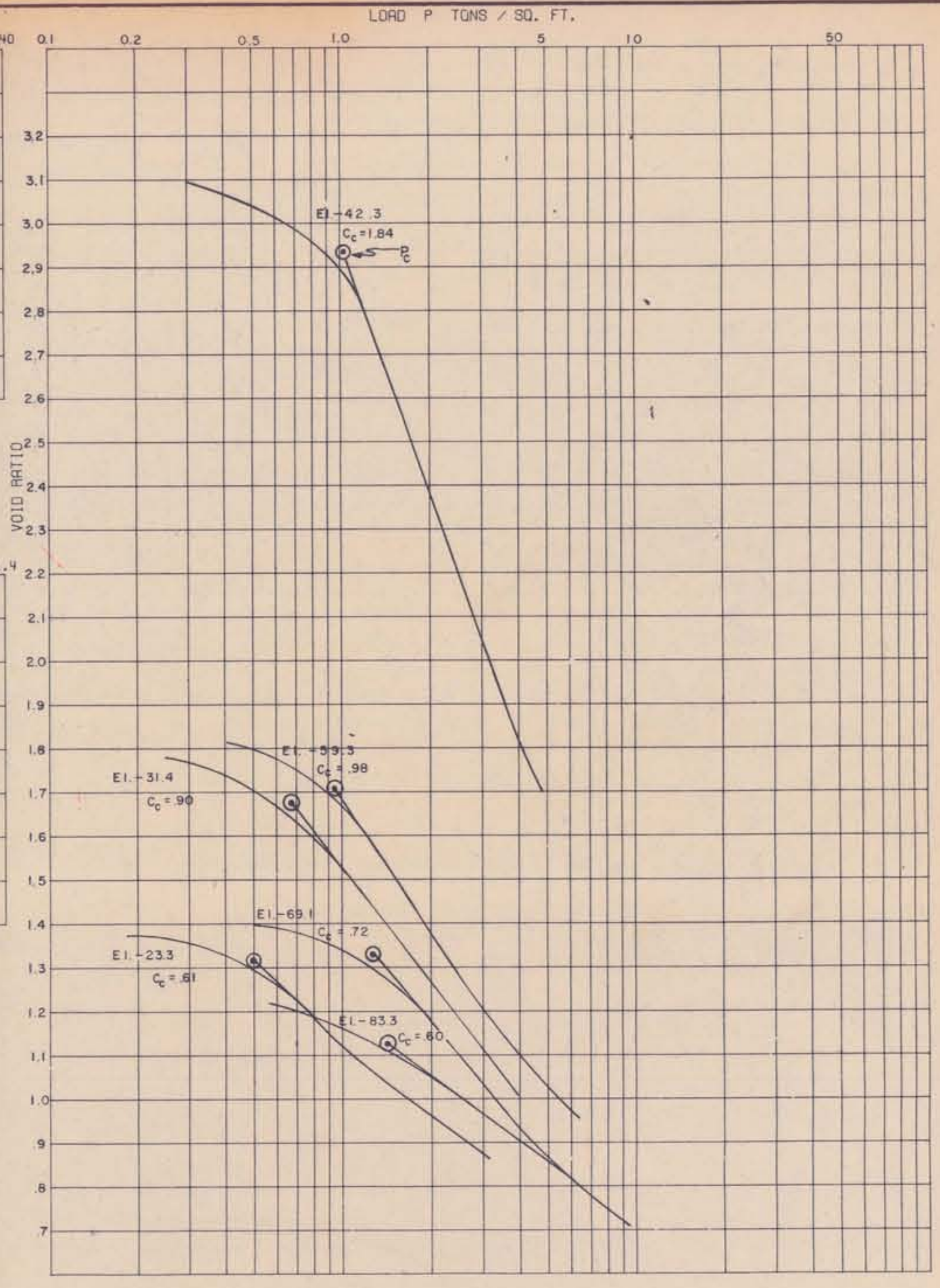


PLASTICITY CHART



SHEAR STRENGTH DATA

BORING NO.	ENVELOPE NO.	ENVELOPE EL.	TYPE	STRENGTH Φ	STRENGTH C - TSF	CLASS
I-SEU	1	- 15.8	Q	1.3°	0.07	CH
	2	- 20.9		0°	0.08	CH
	3	- 27.1		1.4°	0.14	CH
	4	- 40.1		0°	0.36	CH
	5	- 44.9		0°	0.22	CH
	6	- 67.3		0.5°	0.28	CH
	7	- 77.0		1.0°	0.38	CH
	8	- 80.9		1.8°	0.35	CH
	9	- 26.4	R	15°	0	CH
	10	- 19.2	S	36°	0	SM
	11	- 24.8		17°	0.32	CH
	12	- 48.9		34°	0	ML
	13	- 75.2		22°	0.25	CH



CONSOLIDATION DATA

○ - (UC) UNCONFINED COMPRESSION TEST
■ - (Q) UNCONSOLIDATED - UNDRAINED SHEAR TEST
▲ - (R) CONSOLIDATED - UNDRAINED SHEAR TEST
□ - (S) CONSOLIDATED - DRAINED SHEAR TEST

BORINGS WERE TAKEN WITH A 5 INCH DIAMETER STEEL TUBE PISTON TYPE SAMPLER

FOR SOIL BORING LEGEND SEE PLATE A

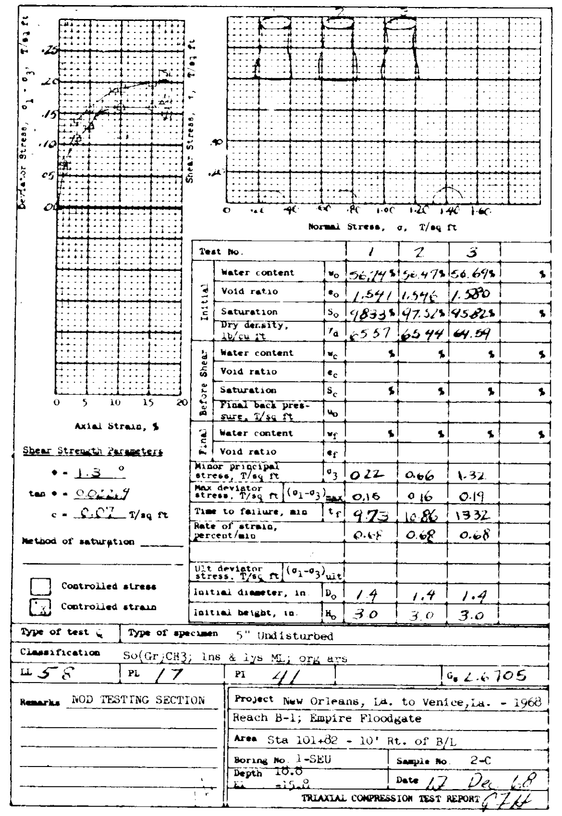
FOR LOCATION OF BORINGS SEE PLATE IV-1

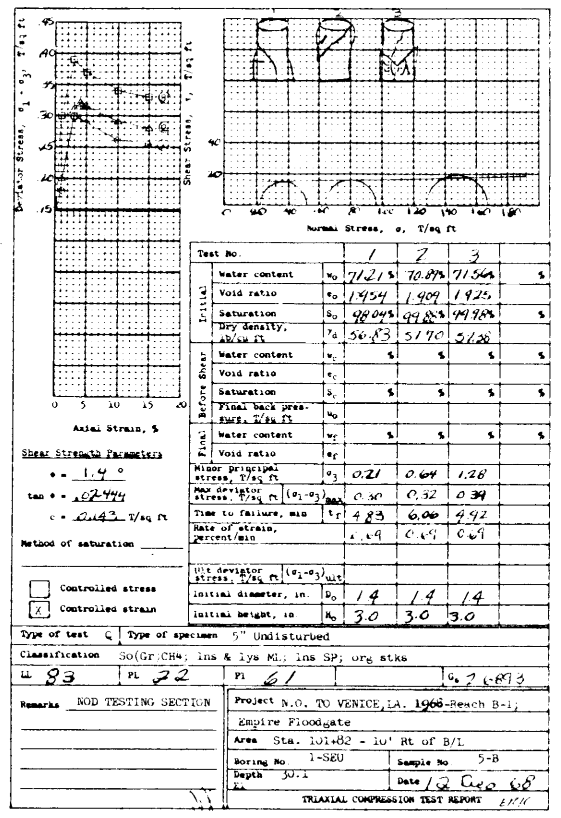
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 — DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON

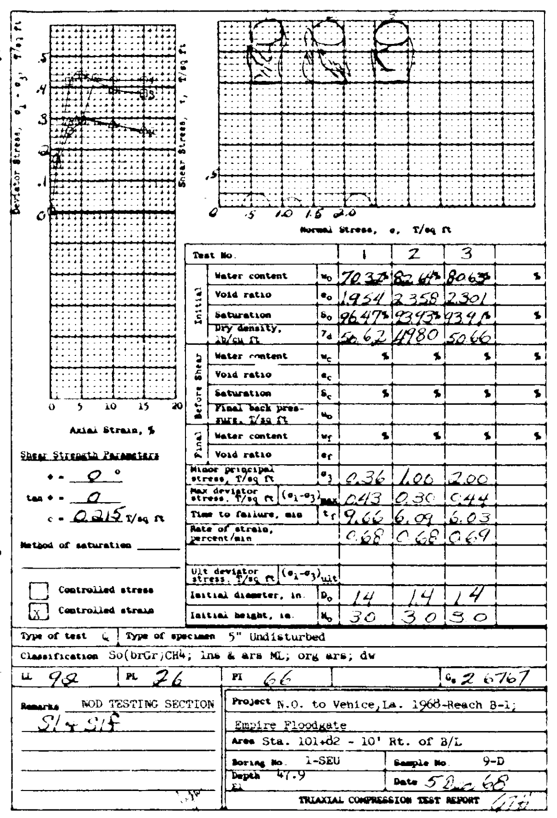
EMPIRE FLOODGATE
UNDISTURBED BORING
I-SEU DATA

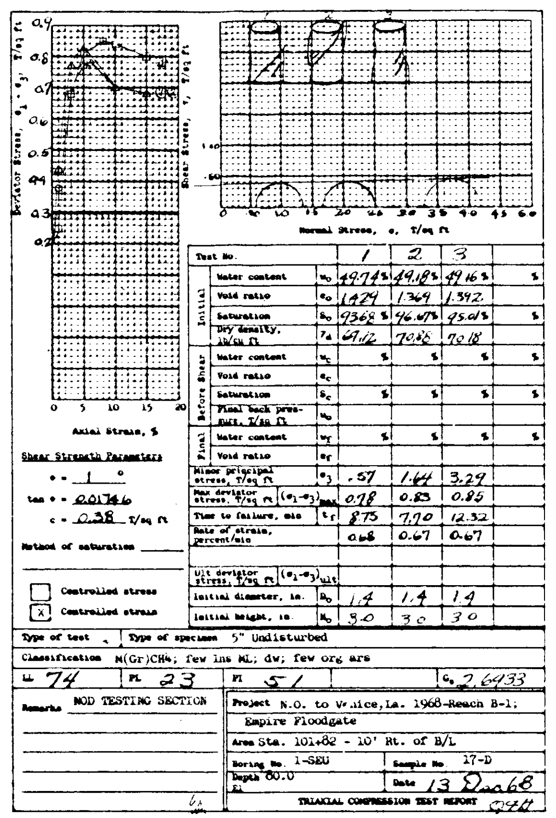
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

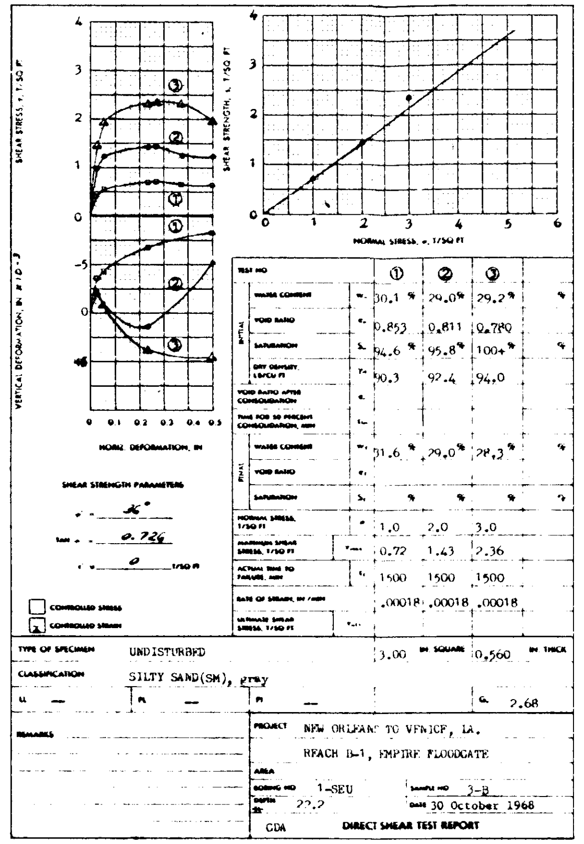
OCTOBER 1970 FILE NO. H-2-25048

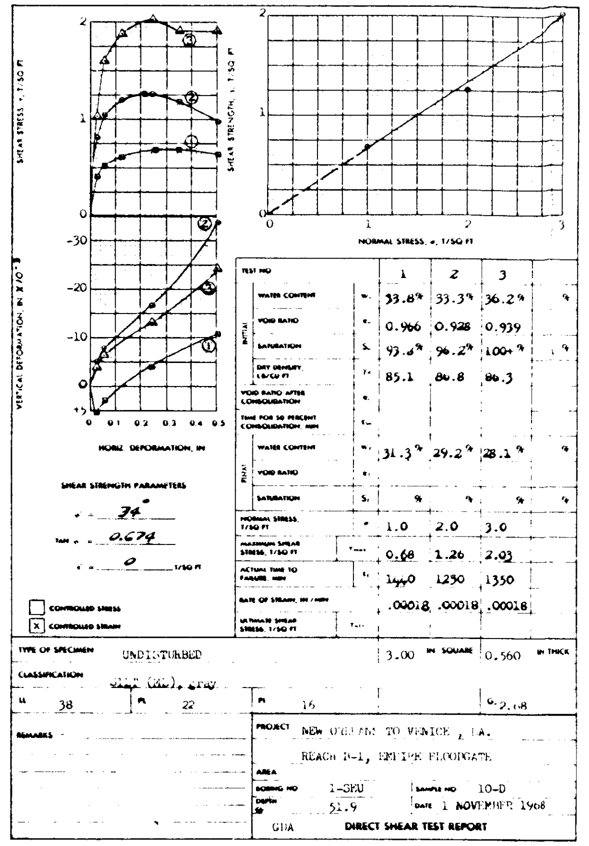


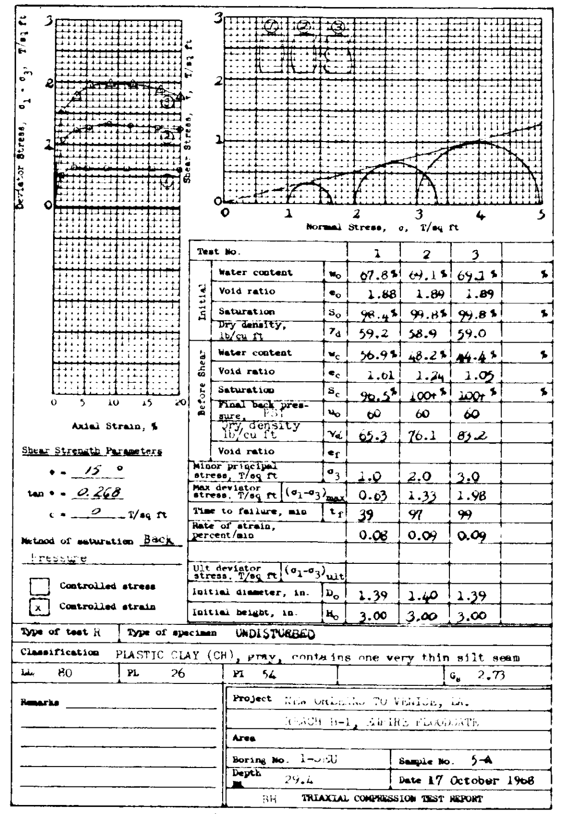


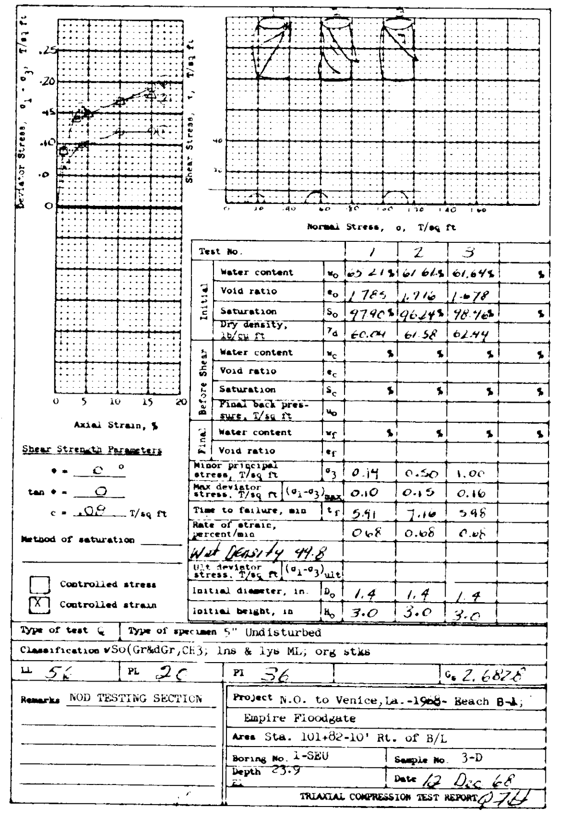


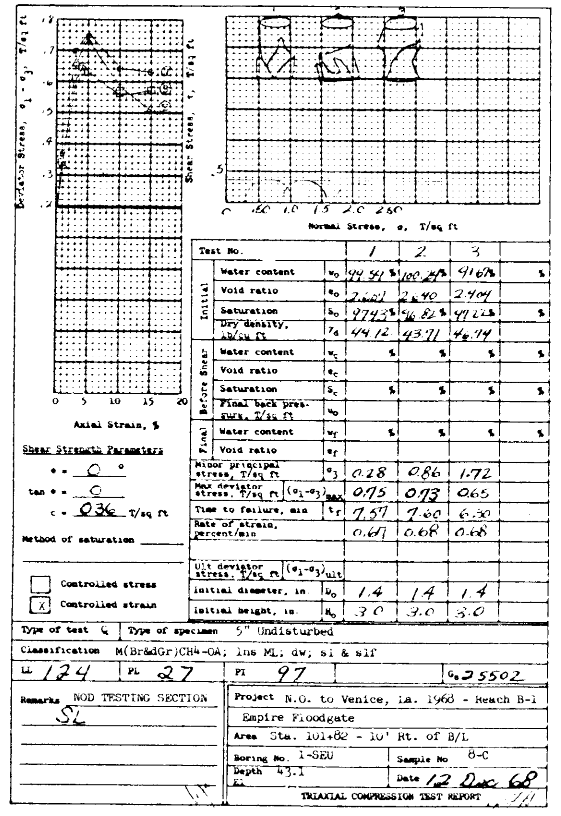


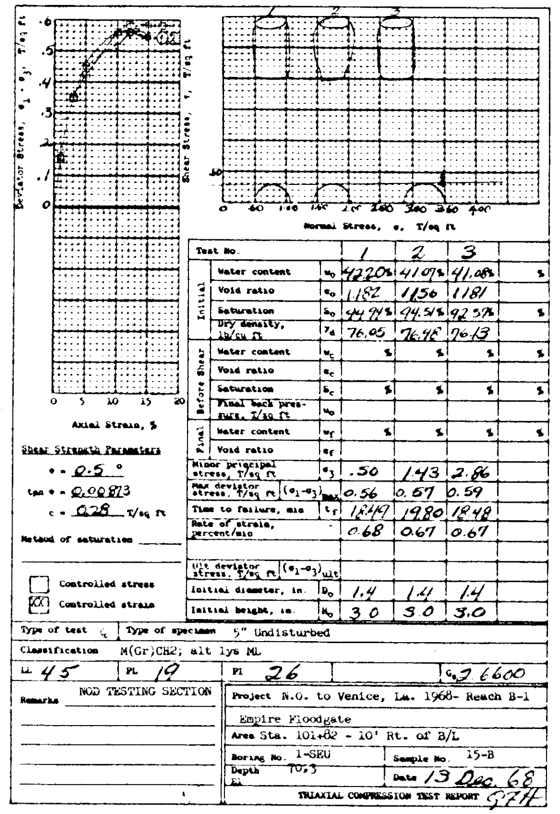


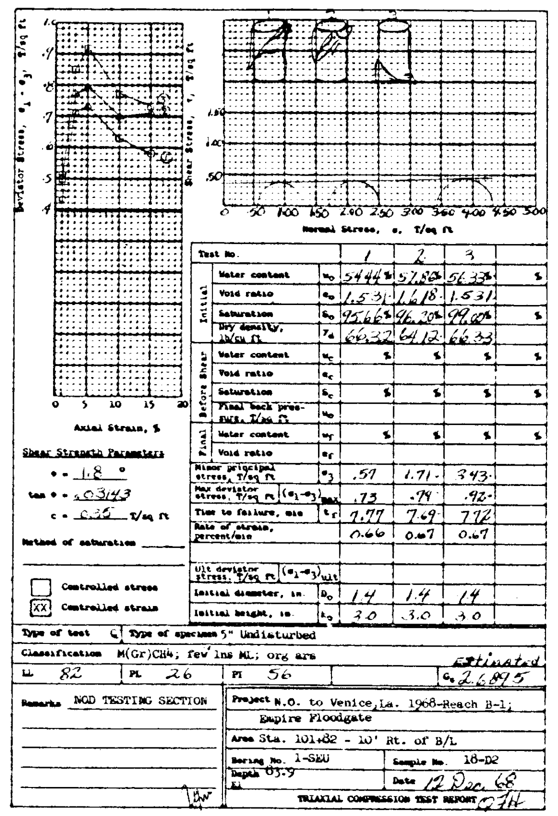


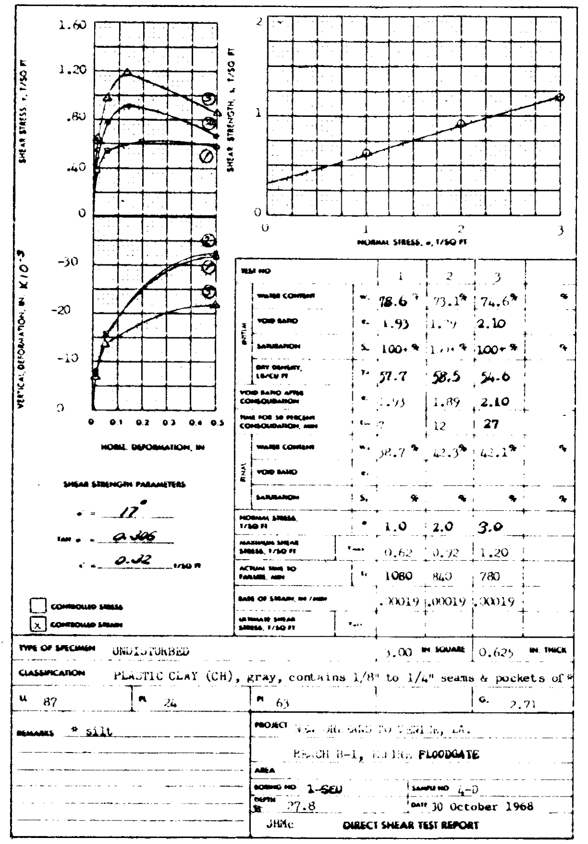


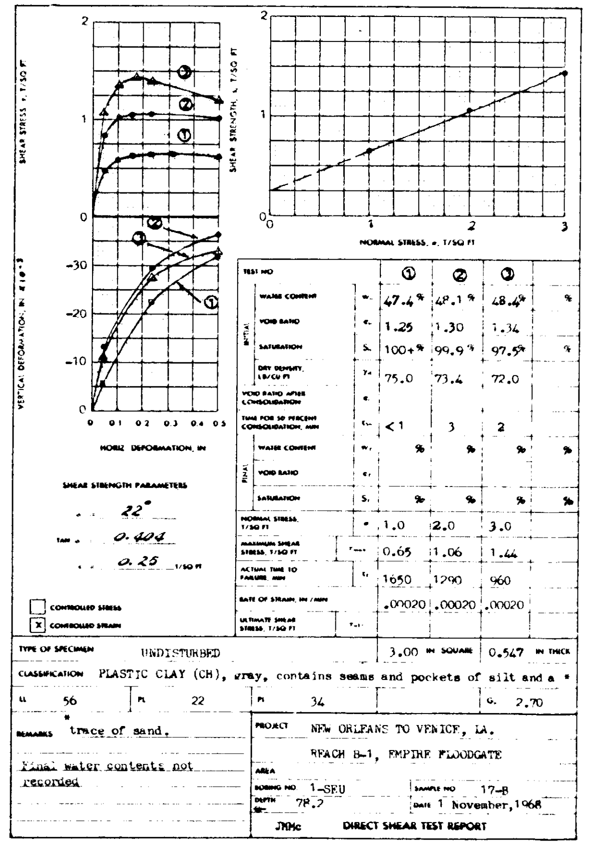












NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN

REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE

DETAIL SHEAR STRENGTH DATA
BORING 1-SEU

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

FILE NO. H-2-25048

PLATE III-25

UNIFIED SOIL CLASSIFICATION

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

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

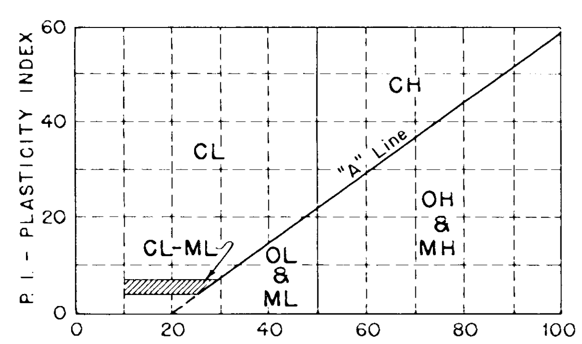
DESCRIPTIVE SYMBOLS

COLOR

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

CONSISTENCY FOR COHESIVE SOILS

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



PLASTICITY CHART

For classification of fine - grained soils

MODIFICATIONS

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

NOTES:

FIGURES TO LEFT OF BORING UNDER COLUMN "W OR D_{10}"
Are natural water contents in percent dry weight
When underlined denotes D_{10} size in mm *
FIGURES TO LEFT OF BORING UNDER COLUMNS "LL" AND "PL"
Are liquid and plastic limits, respectively
SYMBOLS TO LEFT OF BORING
▽ Ground-water surface and date observed
(C) Denotes location of consolidation test **
(S) Denotes location of consolidated-drained direct shear test **
(R) Denotes location of consolidated-undrained triaxial compression test **
(Q) Denotes location of unconsolidated-undrained triaxial compression test **
(T) Denotes location of sample subjected to consolidation test and each of the above three types of shear tests **
FW Denotes free water encountered in boring or sample
FIGURES TO RIGHT OF BORING
Are values of cohesion in lbs./sq.ft. from unconfined compression tests
In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler ($1\frac{3}{8}$" I.D., 2" O.D.) and a 140 lb. driving hammer with a 30" drop
Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample
Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

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

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

GENERAL NOTES:

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

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

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

REVISION	DATE	DESCRIPTION	BY
2	6-8-64	SYMBOL FW, NOTE REVISED	ORAL FROM L.M.V.G.G. 5 JUNE 1964
1	9-17-63	1ST. PAR. OF GENERAL NOTES REVISED	L.M.V.D. MULTIPLE LETTER, DATED 5 SEPT. 1963

SOIL BORING LEGEND

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

FILE NO. H-2-21800

SECTION IV - STRUCTURAL DESIGN

CRITERIA FOR STRUCTURAL DESIGN

1. General. Structural design has been made in accordance with standard engineering practice and with criteria set forth in Engineering Manual for Civil Works Construction published by the Office, Chief of Engineers.

2. Pertinent data. Pertinent data relevant to the hurricane design wave, to the elevations of the water surface, structure and channel, and to the dimensions of the structure and channel are shown in the following tabulation:

a. Design water elevations (feet m.s.l.)

	Gulfside	Landside
Direct head from hurricane	+12.1	+2.0
Reverse head from hurricane	- 2.0	+6.3
Direct head for maintenance	+ 5.0	-1.0
Reverse head for maintenance	- 2.0	+5.0

b. Structure elevations (feet m.s.l.)

Top of wall	+15.0
Top of timber guide walls & fenders	+ 9.5
Top of sill	-17.5/-14.0
Centerline of gate hinges	-15.54
Centerline of hoist wildcat	+17.75
Centerline of cwt, wildcats	+15.0/+21.0
Centerline of needle girders	+ 5.0
Bottom of channel outside limits of riprap	-12.0

c. Structure dimensions.

	Feet
Channel design width	84.0
Gate width (seal to seal)	84.5
Gate recesses	5.5

d. Hurricane design wave.

Fetch length	F	2 miles
Fetch width	W	1 mile
Ratio (from p27-TR4)	Fe/F	0.81
Effective fetch	Fe	8,554 feet
Windspeed	U	77 m.p.h.
Stillwater level elev.	swl	+12.1 feet
Avg. depth of fetch	d	15.7 feet
Depth at bottom seal of gate	d_t	26.85 feet
Min. depth (marsh +1.5 m.s.l.)	d_{Lim}	10.6 feet
Significant wave height	H_s	5.2 feet
Wave period	T	5.8 sec.
Deepwater wave length	L_o	172 feet
Relative depth	d/Lo	.091
Shoaling coef.	Hs/H'o	.9445
Deepwater wave height	H'o	5.5 feet
Wave steepness	$H'o/T^2$	.163
Design wave height	H.01	8.8 feet
Height of breaking wave= $.8d_{Lim}$	H_b	8.5 feet
Design depth at structure	d_d	24.1 feet

3. Unit weights. The following values of unit weights are used in design calculations:

Item	Lb. per cu. ft. Submerged	Saturated
Water	--	62.5
Concrete	87.5	150.0
Steel	427.5	490.0
Riprap	63.0	125.5
Shell	30.0	92.5
Earth	57.5	120.0

4. Design loads. The assumed design loads used in the design of the structure, gate, and abutment walls are tabulated below:

a. Lateral pressures (p.s.f./ft.)

	Submerged	Saturated
Earth	25.875	54.0
Shell	13.5	41.625
Riprap	28.35	56.475

b. Uniform live loads.

	Lbs. per sq.ft.
Walkways & stairs	100
Control building floor	200
Control building roof	20

c. Wind loads on exposed vertical surfaces and projected area of sloped surfaces. (Allowable stressed increased one-third) 30 p.s.f.

d. Wave loads--see figures IV-1 through IV-6 (Allowable stresses increased one-third)

5. Working stresses.

a. General. The allowable working stresses for structural steel and concrete are in accordance with those recommended in "Working Stresses for Structural Design," EM 1110-1-2101 of 1 November 1963. For convenient reference, allowable stresses are tabulated as follows:

b. Allowable working stresses structural steel, ASTM A-36.

Application	Group 1 loading psi	Group 2 loading psi
(1) Tension		
Structural steel net section except at pin holes	18,000	24,000
Net section at pin holes in eyebars, pin connected plates or built-up members	13,500	18,000
(2) Shear		
On the gross section of beam and plate girder webs	12,000	16,000
(3) Compression		
On gross section of axially loaded compression member for (KL/r) less than C_c	$0.83\ K_1 F_Y$	$1.11\ K_1 F_Y$

$$K_1 = \frac{\left(1 - \frac{(KL/r)^2}{2C_c^2}\right)}{F.S.} \qquad \text{where; } C_c = \sqrt{\frac{2\pi^2 E}{F_Y}} = 126.1$$

K = Effective length factor

$$F.S. = \frac{5}{3} + \frac{3}{8}\,\frac{(KL/r)}{C_c} - \frac{(KL/r)^3}{8C_c^3}$$

For axially loaded column with L/r greater than C_c	$\dfrac{124{,}000{,}000}{\left(\frac{KL}{r}\right)^2}$	$\dfrac{165{,}000{,}000}{\left(\frac{KL}{r}\right)^2}$
On secondary member when L/r > 120, modify the above values by multiplying by the following factor:	$\dfrac{1}{1.6-L/200r}$	$\dfrac{1}{1.7-L/200r}$ *
On gross area of plate girder stiffeners	18,000	24,000
On web rolled shapes at toe of fillet	22,500	30,000

Application	Group 1 loading psi	Group 2 loading psi
(4) Bending		
Tension and compression on extreme fibers of rolled sections, plate girders, and built-up members having axis of symmetry and meeting required dimension proportions	20,000	26,500
Tension and compression on extreme fibers of unsymmetrical members (with compression flange supported)	18,000	24,000
Tension and compression on extreme fibers of box type members not meeting required dimension proportions	18,000	24,000

*This modification factor is applied to secondary members for $L/r \geq 150$. For L/r between C_c and 150, a factor of 1.0 is applied.

Application	Group 1 loading psi	Group 2 loading psi
Tension on extreme fibers of other rolled shapes, built-up members and plate girders	18,000	24,000
Compression on extreme fibers of rolled shapes, plate girders, and built-up members having axis of symmetry in the plane of the web (Formula 4)	$0.50K_2F_Y$	$0.67\ K_2F_Y$

$$K_2 = 1 - \frac{(L/r)^2}{2C_c^2\ C_b}$$

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2, \text{ but not more than } 2.3$$

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length.

(Formula 5)	$\dfrac{10{,}000{,}000}{\dfrac{Ld}{A_f}}$	$\dfrac{12{,}000{,}000}{\dfrac{Ld}{A_f}}$
Use larger value computed by Formulas 4 or 5 but not more than basic stress. Where L/r is less than 40, Formula 4 may be neglected. For allowable stresses based on the use of Formula 4, see appendix 1 of EM 1110-1-2101.		
Compression on extreme fibers of channels. Value computed by Formula 5, but not more than	18,000	24,000
Tension and compression on extreme fibers of large pins (max. for Group 2 loading, 0.90 F_Y)	27,000	32,500

Application	Group 1 loading psi	Group 2 loading psi
Tension and compression on extreme fibers of rectangular bearing plates (max. for Group 2 loading, 0.90 F_Y)	22,500	30,500
(5) Bearing		
Milled surfaces and pins in reamed, drilled, or bored holes (max. for Group 2 loading, 0.90 F_Y)	27,000	32,500
Finished stiffeners (max. for Group 2 loading 0.80 F_Y)	24,000	29,000
Expansion rollers and rockers (lbs./lin. inch)	0.83 $K_3 d$	1.11 $K_3 d$

$$K_3 = \left(\frac{F_Y - 13{,}000}{20{,}000}\right) 660$$

d = Diameter of roller or rocker in inches.

Application	Group 1 loading psi	Group 2 loading psi
(6) Bolts (tension)		
A307 bolts	11,500	15,500
A325 bolts	33,500	44,500
A~~354~~ 490 bolts (grade BC)	~~41,500~~ 50,000	~~55,500~~ 66,500
(7) Bolts (shear) (bearing type connections)		
A307 bolts	8,500	11,000
A325 bolts when threading is not excluded from shear planes	12,500	16,500
A325 bolts when threading is excluded from shear planes	18,000	24,000
A~~354~~ 490 bolts (Grade BC) when threading is not excluded from shear planes	18,600 ~~16,500~~	25,000 ~~22,000~~
A~~354~~ 490 bolts (Grade BC) when threading is excluded from shear planes	26,600 ~~20,000~~	35,500 ~~26,500~~

Application	Group 1 loading psi	Group 2 loading psi
(8) Bolts (shear) (friction type connections)		
A325 bolts	12,500	16,500
A~~354~~ 490 bolts (Grade BC)	~~16,500~~ 18,600	~~22~~ 25,000
(9) Bolts (bearing) (bearing type connections)		
Bearing on projected area (max. for Group 2 loading 1.35 F_Y)	1.13 F_Y	1.35 F_Y
(10) Welds		
Fillet, plug, slot, and partial penetration groove welds using A233 Class E-60 electrodes or submerged arc Grade SAW-1	11,500	15,000
Fillet, plug, slot, and partial penetration groove welds using A233 Class E-70 electrodes or submerged arc Grade SAW-2	13,000	17,500

Complete penetration groove welds shall have the same allowable for tension, compression, bending, shear, and bearing stresses as those allowed for the connected material.

(11) Combined stresses

(a) Axial compression and bending. Members subject to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

1. When $f_a/F_a \leq 0.15$,

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$$

2. When $f_a/F_a \geq 0.15$, F_e = Euler stresses divided by factor of safety

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{K_4 F_e}\right) F_b} \leq 1.0 \qquad F_e = \frac{149{,}000{,}000}{\left(\frac{Kl_b}{r_b}\right)^2}$$

and, in addition, at points braced in the plane of bending,

$$\frac{f_a}{K_5 F_Y} + \frac{f_b}{F_b} \leq 1$$

Where K_4 = 0.83 for Group 1 loading and 1.11 for Group 2 loading.

Where K_5 = 0.50 for Group 1 loading and 0.67 for Group 2 loading.

C_m = a coefficient--See Section 1.6 AISC Specifications in Manual of Steel Construction Sixth Edition.

(b) Shear and tension. Rivets and bolts subject to combined shear and tension shall be proportioned so that the tension stress from the force applied to the connected part does not exceed the following:

For A307 bolts	$F_t = 15{,}000 - 1.6\ f_v \leq 10{,}500$
For A325 bolts in bearing type joints	$F_t = 37{,}500 - 1.6\ f_v \leq 30{,}000$
For A~~354~~ 490 bolts (Grade BC) in bearing type joints	$F_t = 45{,}000 - 1.6\ f_v \leq 37{,}500$

where f_v, the shear produced by the same force, shall not exceed the value for shear given in sections (7) and (8) of this paragraph.

For bolts used in friction type joints, the allowable shear stresses shall be reduced to meet the following:

For A325 bolts	$F_v \leq 11{,}000\ (1 - f_t A_b/T_b)$
For A~~354~~ 490 bolts	$F_v \leq 15{,}000\ (1 - f_t A_b/T_b)$

T_b = the proof load of the bolt.

c. Allowable working stresses concrete (3,000 p.s.i. 28 days). Concrete which will be subjected to submergence, wave action, and spray will be designed with working stresses in accordance with ACI Building Code with the following modifications:

Flexure (f_c):

Extreme fiber stress in compression	$0.35\sqrt{f'_c}$
Extreme fiber stress in tension (plain concrete for footings and walls but not for other portions of gravity section)	$1.2\sqrt{f'_c}$
Extreme fiber stress in tension (for other portions of gravity sections)	$0.6\sqrt{f'_c}$

Types of structures to which those modifications apply are:

Floodwalls
Lock walls, guide, and guard walls
Retaining walls subject to contact with water

Allowable stresses in reinforcement will be in accordance with the ACI Building Code except for tension in deformed bars with a yield strength of 60,000 p.s.i. or more, the stress shall not exceed 20,000 p.s.i. based upon Group 1 loading.

For Group 2 loading the above stresses may be increased by 33 1/3%.

Minimum tensile reinforcement. The minimum area of tensile reinforcement steel should be .0025 bt, with a maximum of #9 bars at 12 inches.

Minimum temperature reinforcement. The minimum area of temperature reinforcement steel should be .0020 bt, half in each face, with a maximum of #6 bars at 12 inches.

d. Application of working stresses.

(1) Group 1 loading: Allowable working stresses as listed for structural steel and for reinforced concrete will be applied to the following loads:

Dead load
Live load
Buoyancy
Earth pressure
Water pressure

(2) Group 2 loading: Allowable working stresses as listed for structural steel and for reinforced concrete will be applied to the following loads when combined with Group 1 loads:

Wind loads
Wave loads

e. Prestressed concrete piles. Prestressed concrete piles shall conform to the requirements of the Joint Committee of the American Association of State Highway Officials and the Prestressed Concrete Institute for Standard 12"x12" solid concrete piles with a minimum ultimate design strength of 5,000 p.s.i. at 28 days.

f. Timber piles. Timber piles are Type I, Class B, Southern Pine or Douglas Fir, clean-peeled piles in accordance with the requirements of Federal Specification MM-P-371b, dated 25 April 1967.

6. Foundation.

a. General. The results of subsurface explorations, soils tests, and foundation studies are presented in Section III. The gate and outer sill structures will be founded on untreated timber piling and the abutment walls will be founded on prestressed concrete piling. Allowable pile loads and moduli of horizontal subgrade reaction are indicated on plates III-21 and III-22. Unbalanced lateral forces are resisted by batter piles.

b. Pile foundation and stability analysis. The pile foundations were designed in accordance with EM 1110-2-2906, July 1969, "Design of Pile Structures and Foundations." Computed pile loads were determined from the rational method of pile foundation analysis (method developed by A. Hrennikoff).† A GE-400 automatic data processing system with teletype time sharing and programs K29WL3 and K29022 were utilized for computing pile loads and comparing computed loads with allowable axial and transverse loads to determine the critical pile loads for all load cases on the structure and on each T-wall monolith. All piles were assumed to have a pinned end at the base of the structure and to be friction type piles. For plan and sections of pile foundations, see plates IV-8 and IV-2.

†Paper No. 2401 of ASCE Transactions, "Analysis of Pile Foundations with Batter Piles," by A. Hrennikoff.

c. Timber piles. Untreated, class "B" timber piles are used under the gate structure and outer sill for the following reasons:

(1) Pile heads are relatively deep and always below ground water level.

(2) Soil investigation indicates that driving should not damage piles.

(3) Loads are relatively small.

(4) Timber piles are available and easily handled in the relatively deep excavation.

d. Concrete piles. Precast, prestressed concrete piles are used under the inverted T-type abutment walls for the following reasons:

(1) Computed pile loads are too large for an economical length of timber pile.

(2) Prestressed concrete piles are available in the New Orleans, La., area and are economically feasible.

e. Cutoff wall. To prevent the flow of water under the structures an MA-22 steel sheet pile cutoff is located under the gate structures and adjacent T-type abutment walls as indicated on plate IV-1. A Z-32 steel sheet pile cutoff to elevation -25.0 is used for the I-type walls.

7. Wave loads. Net wave pressures have been computed from the hurricane design wave data in accordance with recommendations of "Shore Protection, Planning and Design," Technical Report No. 4, Third Edition, 1966, by the Coastal Engineering Research Center, Corps of Engineers. The hurricane design wave was assumed to approach the structure at a 90° angle. Design net wave pressure computations for the structure, gate, and abutment walls are shown on figures IV-1 through IV-6.

8. Gate bay.

a. General. The gate structure is a monolithic reinforced concrete U-frame with a recess in the base to allow for a steel gate with horizontal hinges at the bottom. The clear channel is 84 feet wide and the structure walls are 10 feet thick. The total structure width is 106 feet, top of walls are at elevation 15.0, and top of sill is at elevation -14.0. A control house is provided above one wall for operation of the gate, and needle dams are provided for unwatering the gate while the gate is in the closed position. (See plates IV-2 and IV-3 for plans, sections, and details of the gate structure.) The outer sill is a reinforced concrete, modified inverted T-wall which acts as a retaining wall for the gate recess, supports and protects the gate while the gate is open, and supports one end of each of the side retaining walls. The top of the outer sill is at elevation -14 and is 99 feet wide. (See plates IV-2 and IV-3 for plan and sections of the outer sill and the side retaining walls.)

b. Design loading conditions.

Case I - Operating conditions. Maximum direct head (hurricane). Gate closed; flood side water at elevation +12.1, protected side water at elevation +2.0; uplift with sheet pile cutoff considered impervious--no wave force.

Case II - Same as Case I, except uplift with sheet pile cutoff considered pervious.

‡Case III - Maximum direct head with wave forces (hurricane). Gate closed; flood side water at elevation +12.1, protected side water at elevation +2.0; uplift with sheet pile cutoff considered impervious.

‡Case IV - Same as Case III except uplift with sheet pile cutoff considered pervious.

Case V - Maximum reverse head. Gate closed; flood side water at elevation -2.0, protected side water at elevation +6.3; uplift with sheet pile cutoff considered impervious.

Case VI - Same as Case V except uplift with sheet pile cutoff considered pervious.

Non-operating conditions

Case VII - Gate dewatered. Gate removed; needle beams and girders in place; flood side water at elevation +5.0; protected side water at elevation +5.0; full uplift.

Case VIII - Construction condition. Gate closed; no uplift.

‡Cases III and IV are considered Group 2 loadings. All other cases considered Group 1 loadings.

c. Base slab.

(1) The base slab has been treated as a monolithic unit and has been designed to resist bending moments in both the longitudinal and transverse directions for the various loading conditions described in paragraph 8b. (See plates IV-9 through IV-13).

(2) The longitudinal and transverse bending moment diagrams were developed with the assumption that the total amount of all forces producing bending were uniformly distributed over the width of the base in each direction of bending.

(3) The total bending moment in the longitudinal direction has been distributed equally across the width of the base. However, because it was assumed that large moments will be concentrated at the walls the reinforcing steel required per foot of width (as determined from the longitudinal moment diagrams) will be doubled in a 20-foot strip under and adjacent to the walls.

(4) Because the base slab has a depth of 10 feet (15.5 feet wide) on the protected side and a depth of 6.5 feet (20.5 feet wide) on the flood side, it was assumed that the total transverse moment would not be equally distributed on a per foot of width basis. Since the deflections of both sides of the slab are equal, it was assumed that the total transverse moment would be distributed according to the relative stiffnesses (bd^3) of each side. Therefore, each side of the slab was designed for the transverse moment proportional to its stiffness. The transverse moments for each side of the slab were also checked by computing the moments for the flood side and the protected side independently. The moments obtained by this method were compatible with the moments obtained by using the relative stiffnesses of each side of the slab.

(5) Cases III and IV were found to be critical for design in the longitudinal direction and Case VIII was critical in the transverse direction.

(6) The base under Case VII has a factor of safety of 1.18 against uplift if the tension capabilities of the piles are disregarded and 1.84 considering all piles to be active in tension.

d. Gate bay walls.

(1) The gate bay walls were designed to resist the moments and shears caused by water and wave combined with the reaction from the gate. Each wall was treated as a monolithic unit; the moment of inertia for the transformed concrete section was calculated and used in the design of the concrete. See plate IV-14.

(2) A steel grillage has been designed to distribute the large gate reaction into the gate bay wall. This was required because of the reduction in concrete section at the gate reaction due to the shock absorber and chain slot recesses. See figures IV-38 through IV-50.

9. Gate.

a. General. The gate is fabricated structural steel, mounted on horizontal hinges at the bottom and operated by lifting chains connected to each end of a horizontal girder at the top. This horizontal girder spans the full width of the gate and supports vertical beams at the top. Each vertical beam is supported by a hinge at the bottom and horizontal ribs span between the vertical beams to support the skinplate. (See plate IV-5 for elevation, sections, and details of the gate, and figures IV-7 through IV-37 for design analysis of the gate.)

b. Design loading conditions. (See loading conditions shown in par 8b.)

Case I Maximum direct head with no wave forces

§Case III Maximum direct head with wave forces

Case V Maximum reverse head

10. Counterweights.

a. Each of the two counterweights weighs approximately 40,000 pounds and consists of lead billets contained in a structural steel cage which is suspended from the counterweight chain in a vertical recess in each concrete wall of the structure as shown on plate IV-3.

b. Each counterweight chain is supported by two idler wildcats, one located directly over the counterweight recess in each concrete wall and the other cantilevered over the flood side end of each wall. The idler wildcat that is located over the counterweight recess serves only to change the chain direction to permit vertical movement of the counterweight and is supported by steel beams over concrete supports. The idler wildcat that is cantilevered over the flood side end of each wall is positioned to cause the counterweight to perform the following functions:

(1) Exert a lifting force on the gate when the gate is in the open position in order to assist the hoist machinery to overcome the initial forces required to close the gate, i.e., silt on top of gate, hinge friction, inertia of gate, etc. (Weight of each counterweight will be transferred to support framing by two hangers with turnbuckles while gate is stored in the open position.)

(2) Exert a retarding force on the gate after it reaches an angle of 50° with the horizontal to prevent the gate from slamming shut while closing.

(3) Exert a horizontal force on the closed gate to assist in opening the gate while it still has a differential head acting to hold the gate in the closed position.

11. Needle dams. The needle dams consist of reinforced concrete needles supported by a single span steel needle girder with intermediate vertical supports to reduce moments and deflections due to weight of girder. Concrete needles are superior to wooden needles because they are permanent and flotation does not interfere with placement. The needle girder with intermediate supports and typical needle for the flood side needle dam are shown on plate IV-17 and design shown on plate IV-18. Both the span and height of the needle dam on the protected side are less than those shown for the flood side, but the design analysis will be similar.

12. Control house. The control house will be two-story, with operating floor at elevation 24.0 to enable the operator to view the operation of the gate over the sight obstruction of the gate machinery. The second floor will also house the air compressor and electric panels. The first floor will be used to house the engine generator and for storage.

13. Abutment walls.

a. General. An inverted T-type reinforced concrete floodwall abuts the structure wall and extends for 150 feet on each side of the structure to meet a minimum backfill final grade of elevation +8.0. An I-type reinforced concrete floodwall extends from this point into the final levee crown on each side of the structure. The tops of the concrete walls are at elevation +15.0. A reinforced concrete walkway is provided on top of the walls for access to the structure and the control house. For plan and profile of the abutment walls, see plate IV-1.

b. Inverted T-wall. The inverted T-wall is divided into two 25-foot and two 50-foot monoliths on each side of the structure. The bottom of the base varies from elevation -16.25 to elevation -3.75, based on final grades. This wall is supported by prestressed concrete piles. See plate IV-2 for typical sections, plate IV-8 for pile foundation plan, and figures IV-51 through IV-64 for design analysis of the deepest monolith, T1. (See plate IV-15 for design of stem.) Design analyses of the other T-wall monoliths are similar.

c. I-wall. The I-wall consists of Z-32 steel sheet piling driven to elevation -25.0 and a 2-foot thick reinforced concrete section on top which extends a minimum distance of 2 feet below final grade. The I-wall is divided into three equal monoliths 35 feet long at the levee tie-in on each side of the structure. See plate IV-16 for section and design analysis of the typical I-wall monolith. Due to anticipated settlement of the I-wall at the levee tie-ins, concrete capping of the sheet pile I-wall will not be accomplished during the construction of the structure. However, the steel sheet piling for the I-wall will be installed for interim protection. Steel sheet piling will be driven to the final design penetration with the top of the sheet piling extended to elevation 15.0. Later, after settlement of the levee has taken place, the sheet piling will be capped.

14. Timber guide walls and fenders. A 300-foot long timber guide wall and a 100-foot long timber fender are located on each side of the gate structure. The guide wall is on the west side of the channel and the fender is on the east side as indicated on plate IV-6. The tops of the guide wall and fender are at elevation +9.5. Braced piles, consisting of one vertical pile and one batter pile, are located 6 feet on centers with horizontal timber walls and fender timbers as shown on plate IV-6. Piling and timbers are creosoted with 25-pound treatment for protection against marine borers. Removable floating creosoted timber camels will be placed in the gate recesses for protection from marine traffic hitting the wall projections. Construction of the timber fenders is the same as the construction of the guide walls except that the maximum pile spacing is 8 feet.

15. Breakwater. A breakwater with top elevation of +2.0 will be provided to the southwest of the structure, as shown on plate I-3. The breakwater will cause the larger hurricane waves in the wave spectrum approaching the structure from Adams Bay to break on the breakwater during the closing operation, thus limiting the incident wave heights to those equal in height to the smaller waves which approach directly along the channel alignment. The breakwater will provide a quieted area and a substantial reduction in wave loads on the gate machinery due to slammings during closing operations. It is considered that in no case would it be necessary to delay closing the floodgate after the ingress of hurricane tides has produced an elevation of 5.0 on the landside of the structure. The width of the breakwater is one-half the incident deep water wave length and is of sufficient height to limit waves to a height of 78 percent of the depth, 3 feet over the breakwater.

16. Access road. The alignment of the access road has been changed from that shown in "Design Memorandum No. 1, New Orleans to Venice, Reach B1" to make use of the proposed four-lane highway (La. 23) which will parallel the levee on the east side of the structure. The access road will be constructed of shell and follow the general alignment plan shown on plate I-3. Work on the road cannot be started until the levee is completed between station 118+00 and the structure. This is necessary since the road is to be constructed on the landside berm of the levee.

BY: CWR 7-31-69

NEW ORLEANS TO VENICE, LA.

CKD. BY: [illegible]

DESIGN MEMORANDUM No. 2, DETAIL DESIGN

REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

WAVE LOADING - GATE & STRUCTURE $\qquad W = 62.5$ p.c.f.

DESIGN MAX. WAVE HT. (1%), $H_o = 1.7 \times 5.2 = 8.84'$

AVE. DEPTH OF FETCH, d_o = 15.2'

DEEPWATER WAVE LENGTH, L_o = 172'

STILLWATER SURGE EL., SWL = 12.1'

CHANNEL EL. = -12

$d_o/L_o = .08837$

$$d_b = \frac{1.28 H_o}{3.3\sqrt[3]{H_o/L_o}} = 9.22' \text{ (FORM. 1-36)}$$

$$d_b = \sqrt[3]{\left(\frac{H_o T}{1.837}\right)^2} = 9.20' \text{ (FORM 1-37)}$$

REF.

REF.: SHORE PROTECTION, PLANNING AND DESIGN - TECH. REPORT NO. 4, U.S. ARMY COASTAL ENGINEERING RESEARCH CENTER (3RD ED. 1966)

AT STRUCTURE, $d = 12.1 + 12 = 24.1 > d_b$ $\therefore$ USE UNBROKEN WAVE

FROM REF., TABLE D-1:

For d/L_o, $d/L = .1307$ $\qquad L = \frac{d}{d/L} = 184.3'$

$$h_o = \frac{\pi H^2}{L} \coth\left(\frac{2\pi d}{L}\right) = 1.97'$$

$$p_1 = \frac{wH}{\cosh \frac{2\pi d}{L}} = 407.2 \text{ \#/}' \qquad @ \text{ EL. } -12.0$$

TOP OF WAVE EL. $= 12.1 + h_o + H = 22.91'$

TOP OF WALL & GATE EL. = 15.0

WAVE HT. OVER GATE & WALL = 7.91'

NET WAVE PRESS. @ TOP OF GATE, EL +15.0

$$p_T = \frac{7.9105}{22.9105+12}(24.1 \times 62.5 + p_1) = 433.58 \text{ \#/}\square'$$

NET WAVE PRESS. @ EL. 12.1 $= \frac{10.81}{34.91}(1506.25 + p_1) = 592.53$ #/'

NET WAVE PRESS. @ EL. -14.0 $= \frac{36.91}{34.91}(1506.25 + p_1) - 26.1 \times 62.5 = 391.84$ #/'

NET WAVE PRESS. @ EL. -20.5 $= \frac{43.41}{34.91}(1506.25 + p_1) - 32.6 \times 62.5 = 341.86$ #/'

NET WAVE PRESS. @ EL. -24.0 $= \frac{46.91}{34.91}(1506.25 + p_1) - 36.1 \times 62.5 = 314.94$ #/'

NET WAVE PRESS. @ EL. +2.0 $= \frac{20.91}{34.91}(1506.25 + p_1) - 10.1 \times 62.5 = 514.87$ #/'

NET WAVE PRESS. @ EL. -14.5 $= \frac{37.41}{34.91}(1506.25 + p_1) - 26.6 \times 62.5 = 387.99$ #/'

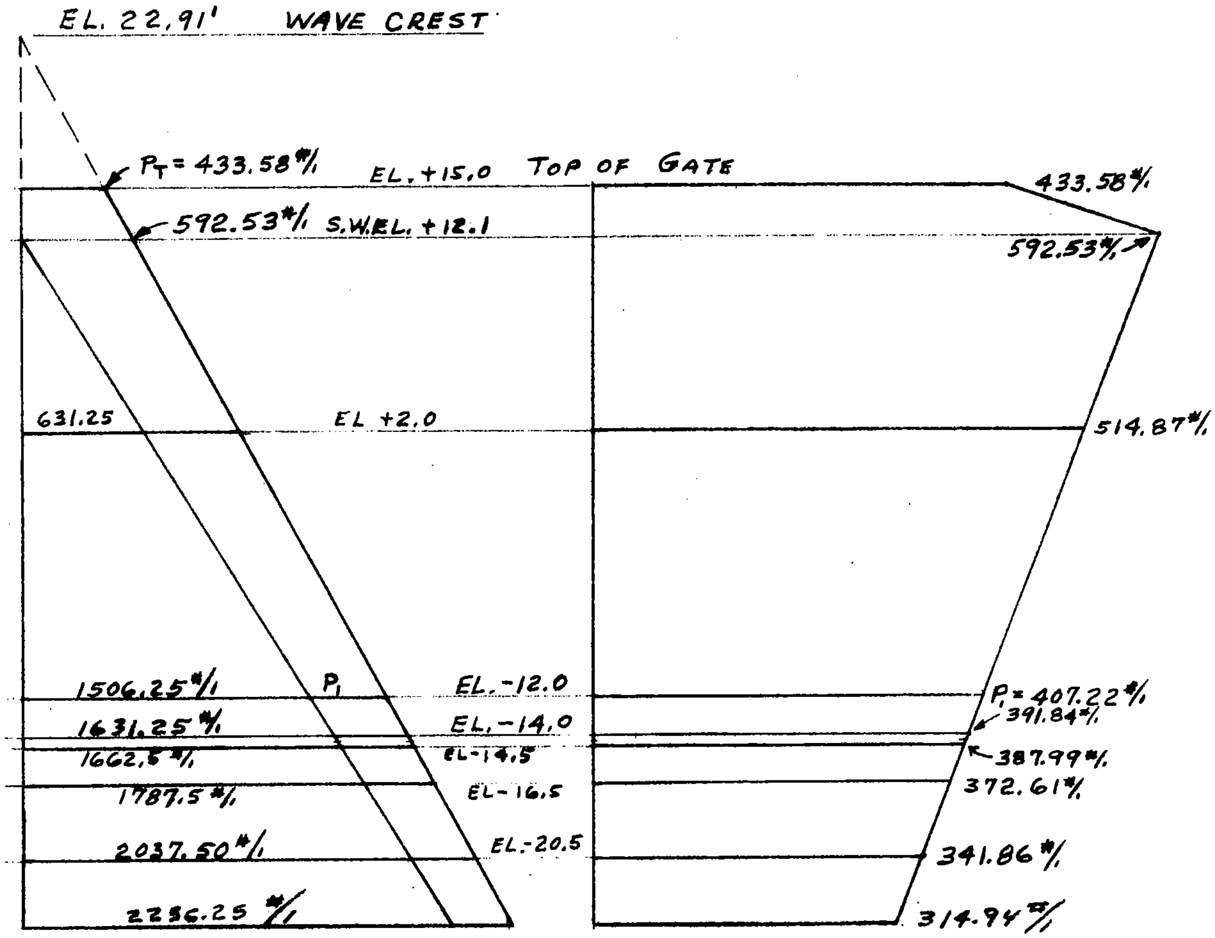
NET WAVE PRESS @ EL. -16.5 $= \frac{37.41}{34.91}(1506.25 + p_1) - 28.6 \times 62.5 = 372.61$ #/'

FIG IV-1

BY: CWR 8-1-69
CKD. BY: [signature]

EMPIRE FLOODGATE

WAVE LOADING - GATE & STRUCTURE (CONT.)



WAVE + HYDROSTATIC PRESSURE

NET WAVE PRESSURE

FIG. IV-2

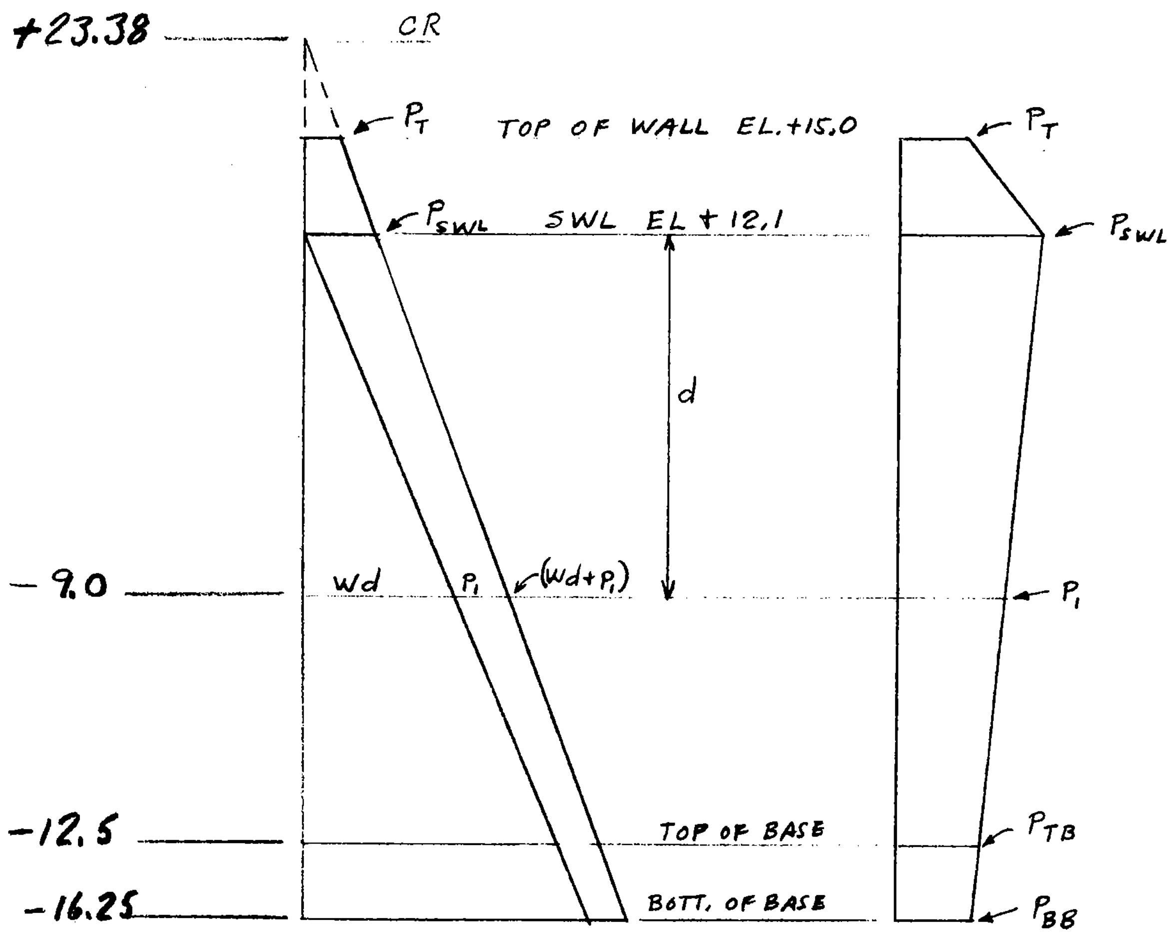
BY: CWR 4/8/70
CKD. BY: [signature]

EMPIRE FLOODGATE

WAVE LOADING - ABUTMENT WALL

$w = 62.5$ p.c.f.

$H_s = 5.2'$ $H_o' = 5.5'$ $T_o = 5.8$ SEC. $L_o = 172'$ DESIGN $H = 8.84'$



WAVE + HYDROSTATIC PRESSURE

NET WAVE PRESSURE

FROM REF: (FORMULA 1-36)

$$d_b = \frac{1.28}{3.3\sqrt[3]{H/L_o}} = 9.22'$$

(FORMULA 1-37)

$$d_b = \sqrt[3]{\left(\frac{HT_o}{1.837}\right)^2} = 9.20'$$

LARGEST $\underline{d_b = 9.22}$

EMPIRE FLOODGATE

WAVE LOADING - ABUTMENT WALLS (CONT.)

MONO. T-1 FIN. GRADE EL -9.0

$d = 12.1 + 9 = \underline{21.1} > d_b$ USE UNBROKEN WAVE

$d/L_o = \frac{21.1}{172} = .1226744$ FROM REF., TABLE D-1,

$d/L = .1604$ $L = 131.5'$

$$h_o = \frac{\pi H^2}{L} \coth\left(\frac{2\pi d}{L}\right) = \underline{2.440'}$$

$$P_1 = \frac{wH}{\cosh\left(\frac{2\pi d}{L}\right)} = \underline{355.9} \;\#/\square'$$

WAVE CREST EL. = CR = SWL + h_o + H = $\underline{23.38'}$

AT EL +15.0, $P_T = (Wd + P_1)\frac{CR-15}{CR-12.1+d} = \underline{433.40\;\#/\square'}$

AT EL +12.1, $P_{SWL} = (Wd + P_1)\frac{CR-12.1}{CR-12.1+d} = \underline{583.39\;\#/\square'}$

AT EL -12.5, $P_{TB} = (Wd + P_1)\frac{CR+12.5}{CR-12.1+d} - W(SWL+12.5) = \underline{318.17\;\#/\square'}$

AT EL -16.25, $P_{BB} = (Wd + P_1)\frac{CR+16.25}{CR-12.1+d} - W(SWL+16.25) = \underline{277.74\;\#/\square'}$

MONO. T-2 FIN. GRADE EL -5.25

$d = 12.1 + 5.25 = \underline{17.35} > d_b$ USE UNBROKEN WAVE

$d/L_o = \frac{17.35}{172} = .1009$ FROM REF, TABLE D-1,

$d/L = .1418$ $L = 122.4'$

$$h_o = \frac{\pi H^2}{L} \coth\left(\frac{2\pi d}{L}\right) = \underline{2.818'}$$

$$P_1 = \frac{wH}{\cosh\left(\frac{2\pi d}{L}\right)} = \underline{388.1} \;\#/\square'$$

WAVE CREST EL. = CR = SWL + h_o + H = $\underline{23.76'}$

AT EL. +15.0, $P_T = (Wd + P_1)\frac{CR-15}{CR-12.1+d} = \underline{444.55\;\#/\square'}$

AT EL. +12.1, $P_{SWL} = (Wd + P_1)\frac{CR-12.1}{CR-12.1+d} = \underline{591.75\;\#/\square'}$

AT EL. -7.25, $P_{TB} = (Wd + P_1)\frac{CR+7.25}{CR-12.1+d} - W(SWL+7.25) = \underline{364.57\;\#/\square'}$

AT EL. -10.5, $P_{BB} = (Wd + P_1)\frac{CR+10.5}{CR-12.1+d} - W(SWL+10.5) = \underline{326.42\;\#/\square'}$

BY: CWR 4/8/70
CKD. BY: [signature]

EMPIRE FLOODGATE

WAVE LOADING - ABUTMENT WALLS (CONT.)

MONO. T-3 & T-4 FIN. GRADE EL. +1.0

$d = 12.1 - 1 = \underline{11.1'} > d_b$ USE UNBROKEN WAVE

$d/L_0 = \frac{11.1}{172} = .06453$ FROM REF., TABLE D-1,

$d/L = .1087$ $L = 102.1'$

$$h_o = \frac{\pi H^2}{L} \coth\left(\frac{2\pi d}{L}\right) = \underline{4.05'}$$

$$p_1 = \frac{wH}{\cosh\left(\frac{2\pi d}{L}\right)} = \underline{444.63\ \#/\square'}$$

WAVE CREST EL. = CR = SWL + h_o + H = $\underline{24.99'}$

AT EL. +15.0, $P_T = (Wd + p_1)\frac{CR-15}{CR-12.1+d} = \underline{474.09\ \#/\square'}$

AT EL. +12.1, $P_{SWL} = (Wd + P_1)\frac{CR-12.1}{CR-12.1+d} = \underline{611.69\ \#/\square'}$

AT EL. -1.0, $P_{TB} = (Wd + p_1)\frac{CR+1}{CR-12.1+d} - W(SWL+1) = \underline{414.53\ \#/\square'}$

AT EL. -3.75, $P_{BB} = (Wd + p_1)\frac{CR+3.75}{CR-12.1+d} - W(SWL+3.75) = \underline{373.15\ \#/\square'}$

MONO. I-5, I-6 & I-7 MIN. FIN. GRADE EL +8.0

$d = 12.1 - 8.0 = 4.1 < d_b$ USE BREAKING WAVE

FROM REF., FORMULAS 4-8 & 4-10:

MAX. DYNAMIC PRESS. = $p_m = \frac{101\, w H_b}{L_D} \times \frac{d}{D}(D + d)$

MAX. NET HYDROSTATIC WAVE PRESS. = $P_s = WH_b/2$

FROM REF., PAR. 4.12, H_b IS DETERMINED BY WATER DEPTH AT A DISTANCE $7 \times H_b$ SEAWARD OF STRUCTURE. IF DESIGN DEPTH $\leq 1.3 H_{10}$, THEN $H_b = .78 \times$ DESIGN DEPTH (FORMULA 1-35). IF DESIGN DEPTH $> 1.3 H_{10}$, THEN $H_b = \frac{1.837}{T}(d_b)^{3/2}$ (FORMULA 1-37).

$H_{10} = 1.27 H_S = 1.27 \times 5.2 = 6.60'$ (FORMULA 1-22)

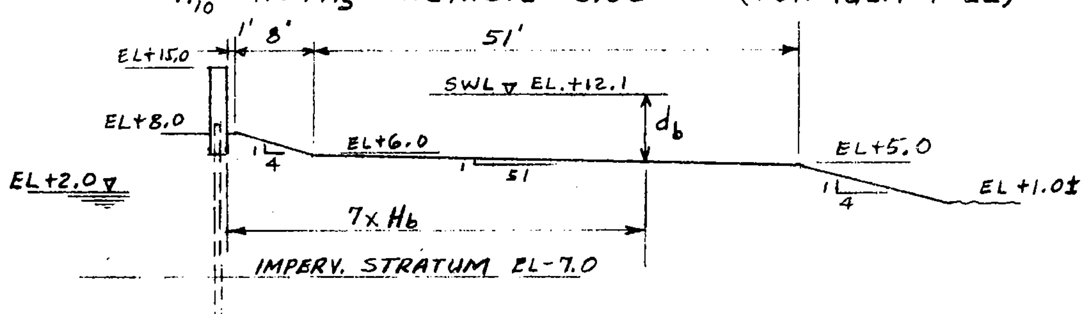


FIG. IV - 5

EMPIRE FLOODGATE

WAVE LOADING - ABUTMENT WALLS (CONT.)

MONO. I-5, I-6 & I-7 (CONT.) $1.3\,H_{10} = 8.59'$

AT A DISTANCE OF 36.2' FROM FACE OF WALL,

$d_b = (12.1-6) + (36.2-9)/51 = 6.63' < 1.3\,H_{10}$ (USE 1-35)

$H_b = .78\,d_b = \underline{5.17'}$ $7 \times H_b = 36.2'$ AS ASSUMED

FROM REF., TABLE D-1:

FOR $d/L_0 = \frac{4.1}{172} = .02384$ $d/L = .06317$ $L = 64.9'$

AT A DISTANCE 64.9021' FROM FACE OF WALL,

ACTUAL DEPTH, $D = (12.1-5) + (64.90 - 60)/4 = 8.33'$

FOR A MIN. AVE. SLOPE OF 1 ON 10 IN ACCORDANCE W/ PAR. 4.231 OF REF.; MIN. DEPTH, $D = d + L/10 = 10.59 > 8.33$

$\therefore$ USE $D = \underline{10.59'}$

FROM REF., TABLE D-1:

FOR $D/L_0 = \frac{10.59}{172} = .06157$ $D/L_D = .1059$ $L_D = \underline{100.0'}$

NET STATIC WAVE PRESS., $P_s = 62.5 \times 5.17/2 = \underline{161.56\ \#/\square'}$

MAX. DYNAMIC WAVE PRESS., $P_m = \frac{101 \times 62.5 \times 5.17 \times 4.1}{100.0 \times 10.59}(10.59+4.1)$

$P_m = \underline{1855.57\ \#/\square'}$

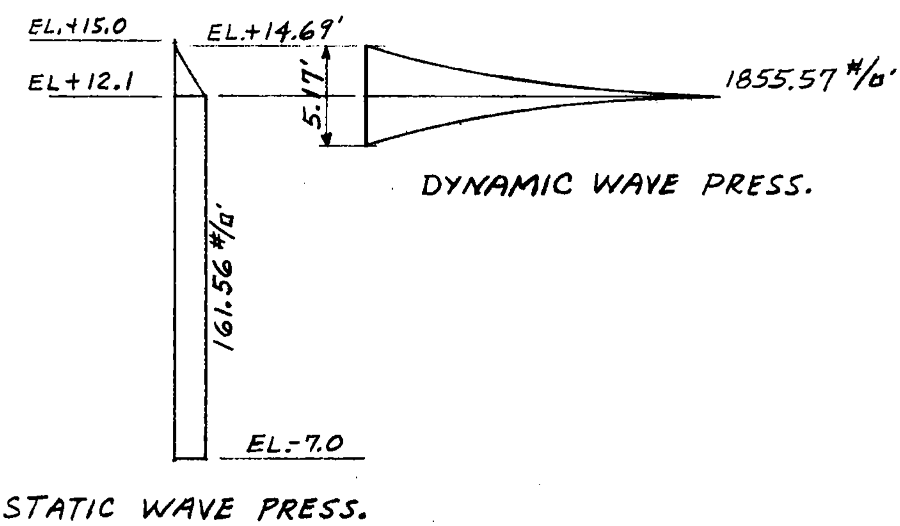
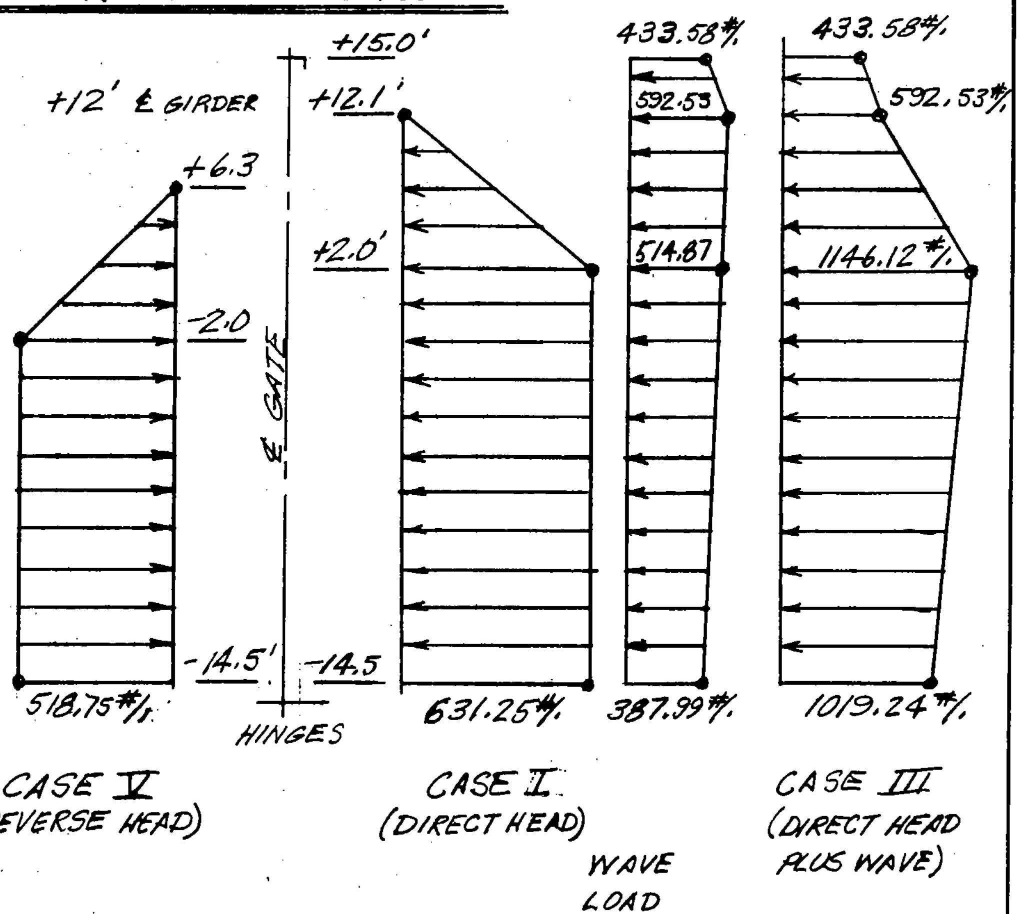


FIG IV-6

THE EMPIRE FLOOD GATE IS DESIGNED AS A BOTTOM-HINGED VERTICAL FLAP GATE.

THE GATE DESIGN CONSISTS OF VERTICAL BEAMS WHICH TRANSMIT THE WATER LOAD AT THE BASE SLAB THRU HINGES AND WHICH ARE SUPPORTED AT THE TOP BY A HORIZONTAL GIRDER THAT TRANSMITS THE WATER LOAD TO THE GATE BAY WALLS.

GATE LOADING CASES



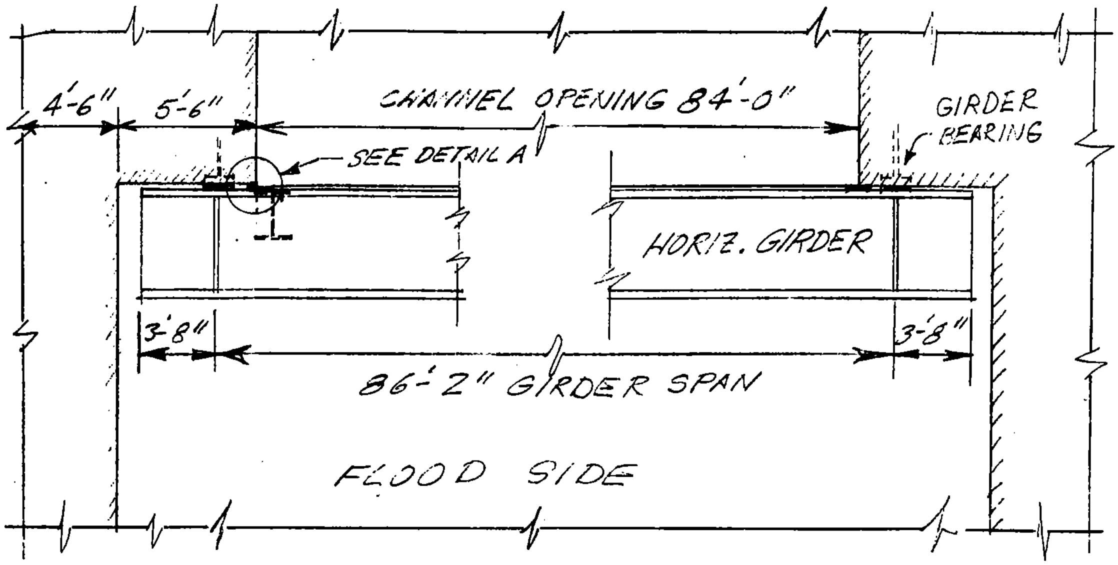
GATE MATERIAL – A36 STEEL

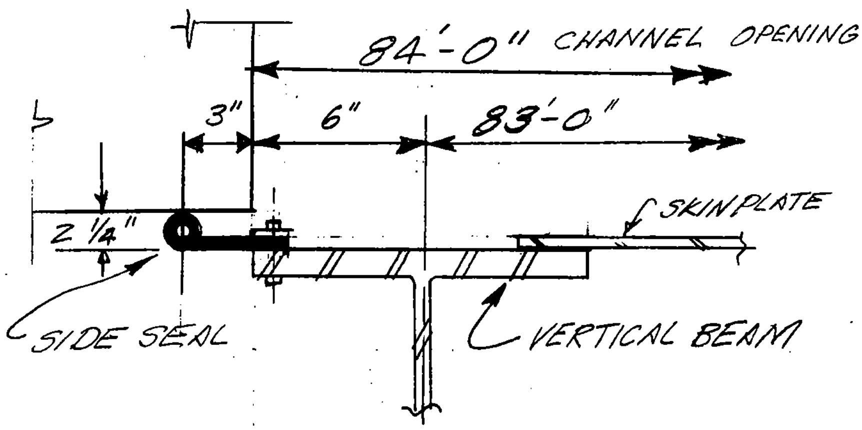
BASIC STRESS ALLOWABLE = 18000 PSI W/O WAVE

TEMPORY STRESS ALLOWABLE = 24000 PSI W/ WAVE

FIG IV-7

GATE DIMENSIONS





DETAIL "A"

TRY A VERTICAL BEAM SPACING OF:

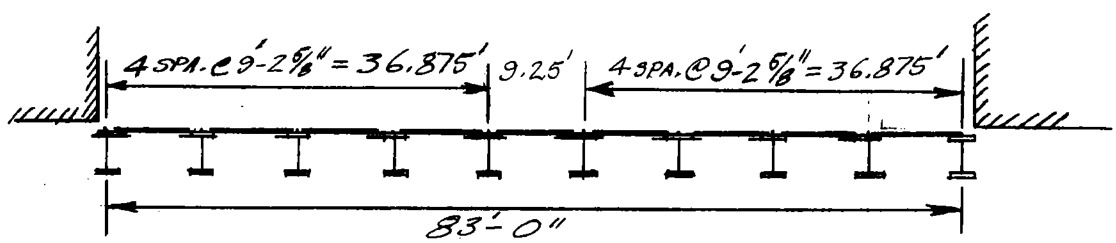
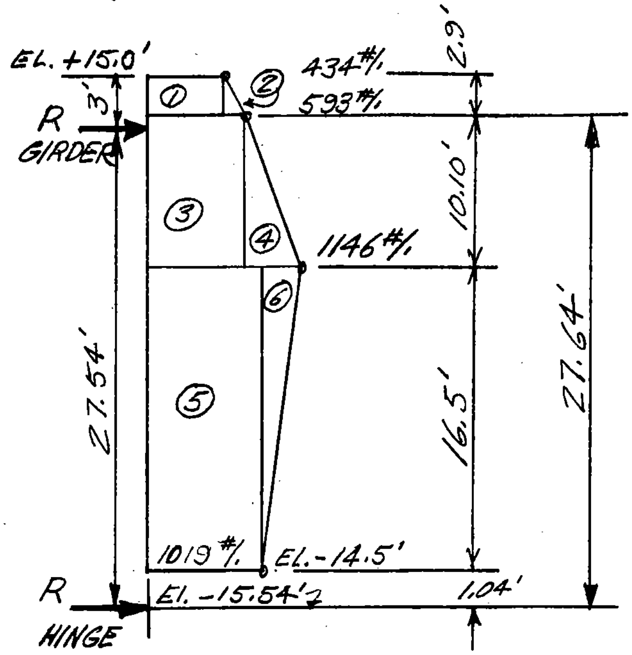


FIG IV-8

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA	Page 3 of 31	EJM	MAY 70
SUBJECT EMPIRE FLOOD GATE - GATE DESIGN		CHECKED BY HJ	DATE MAY 70

DESIGN OF VERTICAL BEAMS

USING CASE III WATER LOADS
(DIRECT HEAD PLUS WAVE LOAD)



FIND REACTIONS PER FOOT OF WIDTH

	p	d	= P	x $\bar{Y}$	M
①	434	x 2.90'	= 1258.60	x 29.09'	= 36,612.67#'
②	159	x 1.45'	= 230.55	x 28.61'	= 6,596.03
③	593	x 10.10'	= 5989.30	x 22.59'	= 135,298.29
④	553	x 5.05'	= 2792.65	x 20.91'	= 58,394.31
⑤	1019	x 16.50'	= 16813.50	x 9.29'	= 156,197.42
⑥	127	x 8.25'	= 1047.75	x 12.04'	= 12,614.91
			28132.35#	x	= 405,713.63

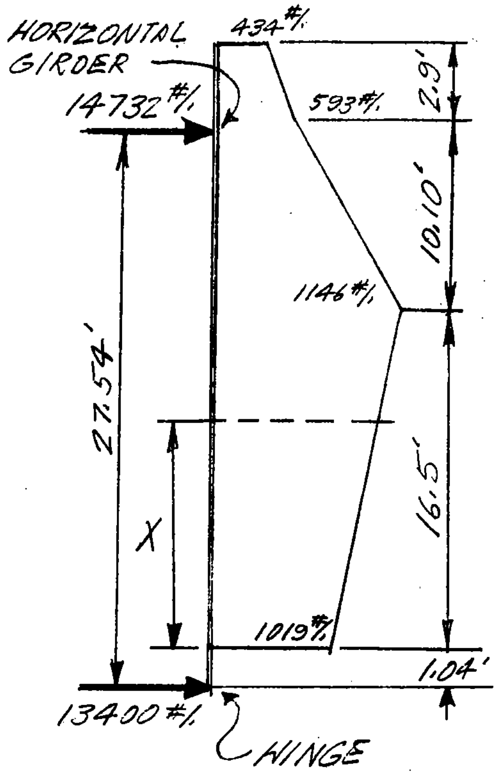
$R_{GIRDER}/_1 = 405713.63/27.54 = 14731.8\# \approx 14732\#/.$

$R_{HINGE}/_1 = 28132.35 - 14731.8 = 13400.55 \approx 13400\#/.$

FIG IV-9

MOMENT ON VERTICAL BEAM

BEAM SPACING = 9'-3" C.C.



LOAD PER FOOT ON VERTICAL BEAM

POINT OF ZERO SHEAR

$$13400 = 1019x + \frac{1146-1019}{16.5}\frac{x^2}{2}$$

$$13400 = 1019x + \frac{7.7}{2}x^2$$

$$x^2 + 264.68x - 3480.52 = 0$$

$$x = -132.34 + \sqrt{20994.4}$$

$$x = -132.34 + 144.89$$

$$x = 12.55'$$

FOR MAXIMUM MOMENT:

$$x' = x + 1.04' = 13.59'$$

MAXIMUM MOMENT:

$$13400\,(13.59) = 182106$$

$$-12788\,(6.275) = 80235$$

$$-606\,(4.183) = 2535$$

(MAX. MOM PER FT.) = 99336 '#/

VERTICAL BEAMS ARE SPACED @ 9'-3" c to c:

$$M_{MAX\ PER\ BEAM} = 99336\ '\#/\,(9.25') = 918860\ '\#$$

REQUIRED SECTION MODULUS:

$$S = \frac{M}{f} = \frac{918860\ '\#\,(12"/')}{24000\ \#/\square"} = 459.4\ IN.^3$$

TRY: 36 WF 150: (A36 STEEL w/ 24000 PSI ALLOWABLE)

FIG IV-10

36 WF 150: PROPERTIES:

$S = 502.9 \text{ IN}^3 > 459.4 \text{ IN}^3 \therefore OK$

MAXIMUM UNBRACED LENGTH OF COMPRESSION FLANGE:

$L_u = 14.2'$; BEAM LENGTH $\approx 28'$

BRACING WILL BE REQUIRED NEAR CENTER OF SPAN.
BRACE BEAMS WITH 36 WF 150 SECTION

CHECK WEB SHEAR:

$V = 13400^{\#} \times 9.25' = 123950\#$

UNIT WEB SHEAR:

$$v = \frac{V}{A_{WEB}} = \frac{123950\#}{.625(35.84 - 2(.94))} = 5839.8 \text{ PSI}$$

COMPUTE DEFLECTION OF 36 WF 150

BEAM SPAN = 27.54'

AVERAGE UNIFORM LOAD:

$$w = \frac{\frac{598+1146}{2}(10') + \frac{1146+1019}{2}(16.5)}{26.5}\left[9.25\right] \cong 9300^{\#}/'$$

APPROX $\Delta = \frac{5}{384}\frac{WL^3}{EI}$ AISC PG 2-12

$$\Delta = \frac{5(9300)(27.54)(27.54)^3(12''/')^3}{384(30\times10^6)(9012.1 \text{ IN}^4)} \cong 0.45''$$

$\therefore$ FOR VERTICAL BEAMS USE 36 WF 150-A36 ROLLED SECTIONS SPACED AS SHOWN ON PAGE 2 = 10 BEAMS.

WITH LATERAL BRACING OF COMPRESSION FLANGE NEAR CENTER OF SPAN. USE 36 WF 150 SECTIONS AS BRACING MEMBERS.

FIG IV-11

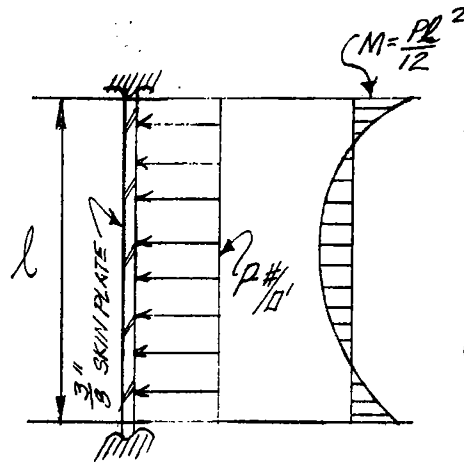
DESIGN OF SKINPLATE

TRY 3/8" TH. SKINPLATE:

SECTION MODULUS PER FOOT.

$$S = \frac{I}{c} = \frac{bh^3}{12}\left(\frac{2}{h}\right) = bh^2/6 = \frac{12(3/8)^2}{6} = 0.281\,IN.^3$$

LENGTH OF SKINPLATE TO DEVELOPE MAXIMUM MOMENT FROM LOAD.



ALLOWABLE STRESS:

$$f_b = 18000(1.33) = 24000\,PSI$$

MAXIMUM MOMENT:

$$M = f_b S$$

$$(12''/FT)\frac{P\ell^2}{12} = 24000(.281)$$

$$\ell^2 = \frac{6744}{P};\ \ell = \frac{82.12}{\sqrt{P}}$$

FROM LOAD CASE III:

ELEVATION	LOAD (#/□')	ℓ (FT.)
12.1 MSL	593	3.37'
+2.0 MSL	1146	2.42'
-8.25 MSL	1082	2.50
-14.5 MSL	1019	2.58

AVERAGE $\ell \cong 2.5'$

FIG IV - 12

DESIGN OF HORIZONTAL SKINPLATE STIFFENERS

SPACE STIFFENERS @ 2'-6" WITH A SPAN OF 9'-3" BETWEEN VERTICALS.

MAXIMUM LOAD ON STIFFENER (FOR CASE III LOAD)

$1146 \#/ft'(2.5') = 2865 \#/FT.$

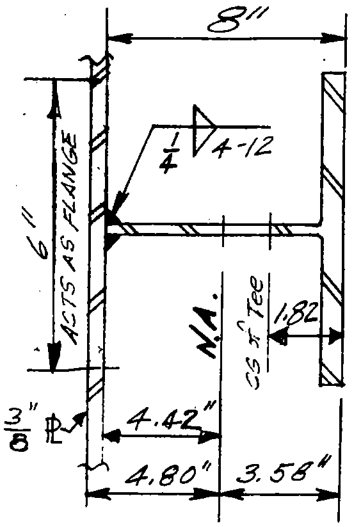
MAXIMUM MOMENT ON STIFFENER

$$M = \frac{w\ell^2}{8} = \frac{2865\,(9.25)^2}{8} = 30642 \ '\#$$

REQUIRED SECTION MODULUS

$$S = \frac{M}{f} = \frac{30642'\#(12''/')}{24000\#/\square''} = 15.32\ IN.^3$$

TRY AN ST 8 WF 20 ROLLED SECTION



CHECK SECTION MODULUS OF "T" AND PL

FIND MOMENT OF INERTIA

C.G. OF T + PL

$5.89\square'' \times 1.82'' = 10.72$	ST8
$6'' \times 3/8'' \times 8.19'' = 18.43$	SKIN PL
$8.14\square'' \times 3.58 = 29.15$	

I_0 ST 8 WF 20 $= 33.2\ IN^4$

$Ad^2(T)\ \ 5.89\,(1.76^2) = 18.2\ IN^4$

$Ad^2(PL)\ \ 2.25\,(4.61)^2 = 47.6\ IN^4$

$I = 99.0\ IN^4$

MINIMUM SECTION MODULUS

$S = I/c = 99/4.8 = 20.5\ IN^3 > 15.3$

CHECK FOR MAX SHEAR

$V = 13242\#$

$f_v = 13242/8(.307) = 5400\ psi$

$f_{va} = 14500\ psi \therefore OK$

CHECK WELD:

$$v = \frac{VQ}{I} = \frac{13242(2.25)(4.61)}{99} = 1429\#/''$$

SHEAR DUE TO WATER LOAD

$2860/12'' = 240\#/''$

$1669\#/''$

1/4 FILLET WELD ALLOWABLE

$500\,(4)(1.33) = 2660\#/''$

$$MAX\ f_b = \frac{30642\,(12)}{20} = 17936\ psi$$

MAX. UNBRACED LENGTH

$L_u = 136 f_2\ 13(7) = 91''$

$L_{ub} = 111 - 12 = 99''$

NO BRACING OF COMP. FL. REQ.

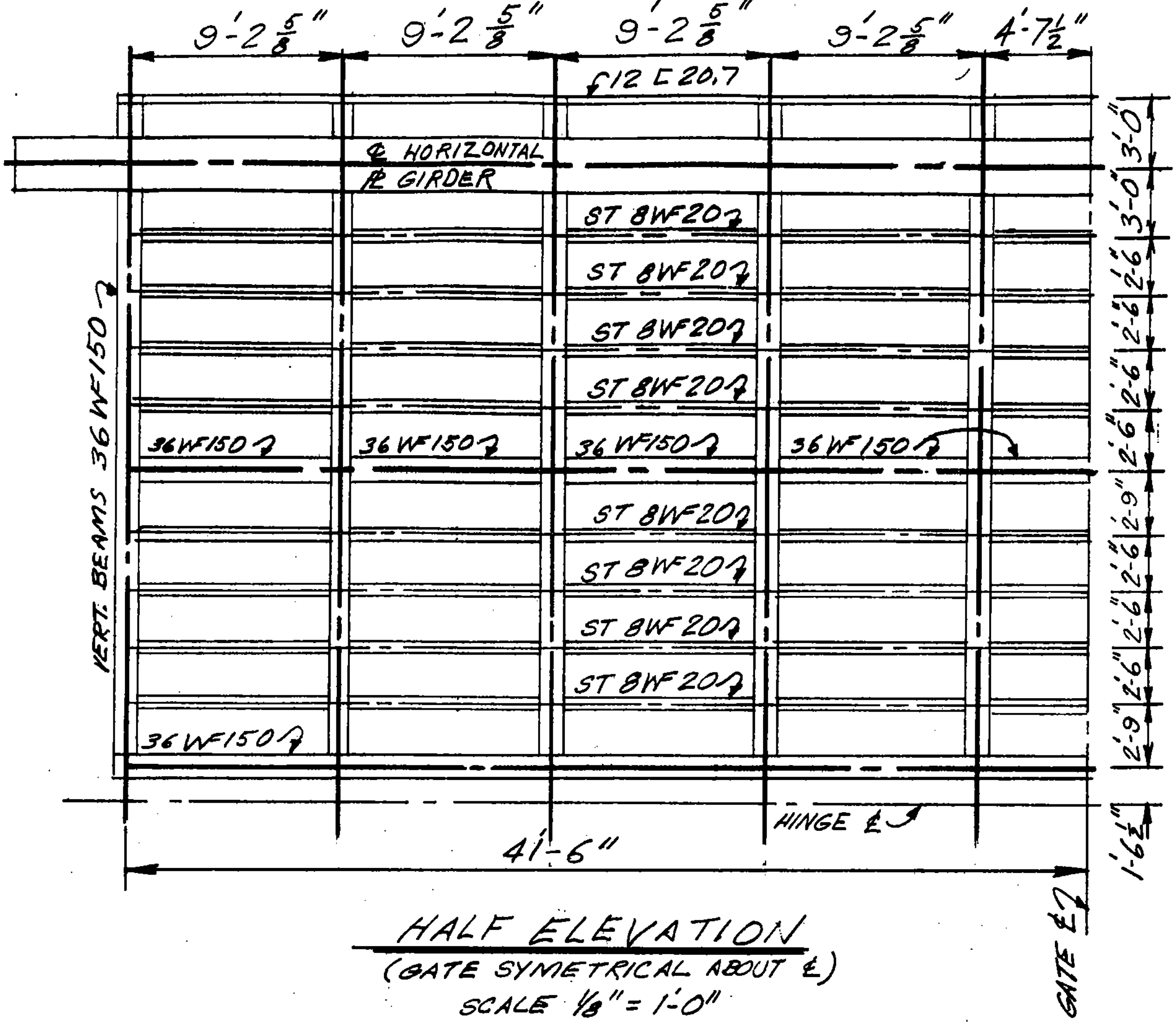
DEFLECTION OF STIFFENER

$$\delta_{MAX} = \frac{5}{384}\ \frac{2865 \times 9.25 \times 111^3}{30 \times 10^6 \times 99} = 0.16''$$

FIG IV-13

PROJECT NEW ORLEANS TO VENICE, LA.	Page 8 of 31	COMPUTED BY EJM	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE - GATE DESIGN		CHECKED BY HU	DATE

GATE LAYOUT:



HALF ELEVATION
(GATE SYMETRICAL ABOUT ℄)
SCALE 1/8" = 1'-0"

FIG IV- 14

LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT NEW ORLEANS TO VENICE, LA.	Page 9 of 31	COMPUTED BY EJM	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE – GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

DESIGN OF PLATE GIRDER

UPPER REACTION PER FOOT = 14 732 #/.

GIRDER SPAN = 86'-2" C. C. OF BEARINGS (SEE PAGE 2) (FIG IV-8)

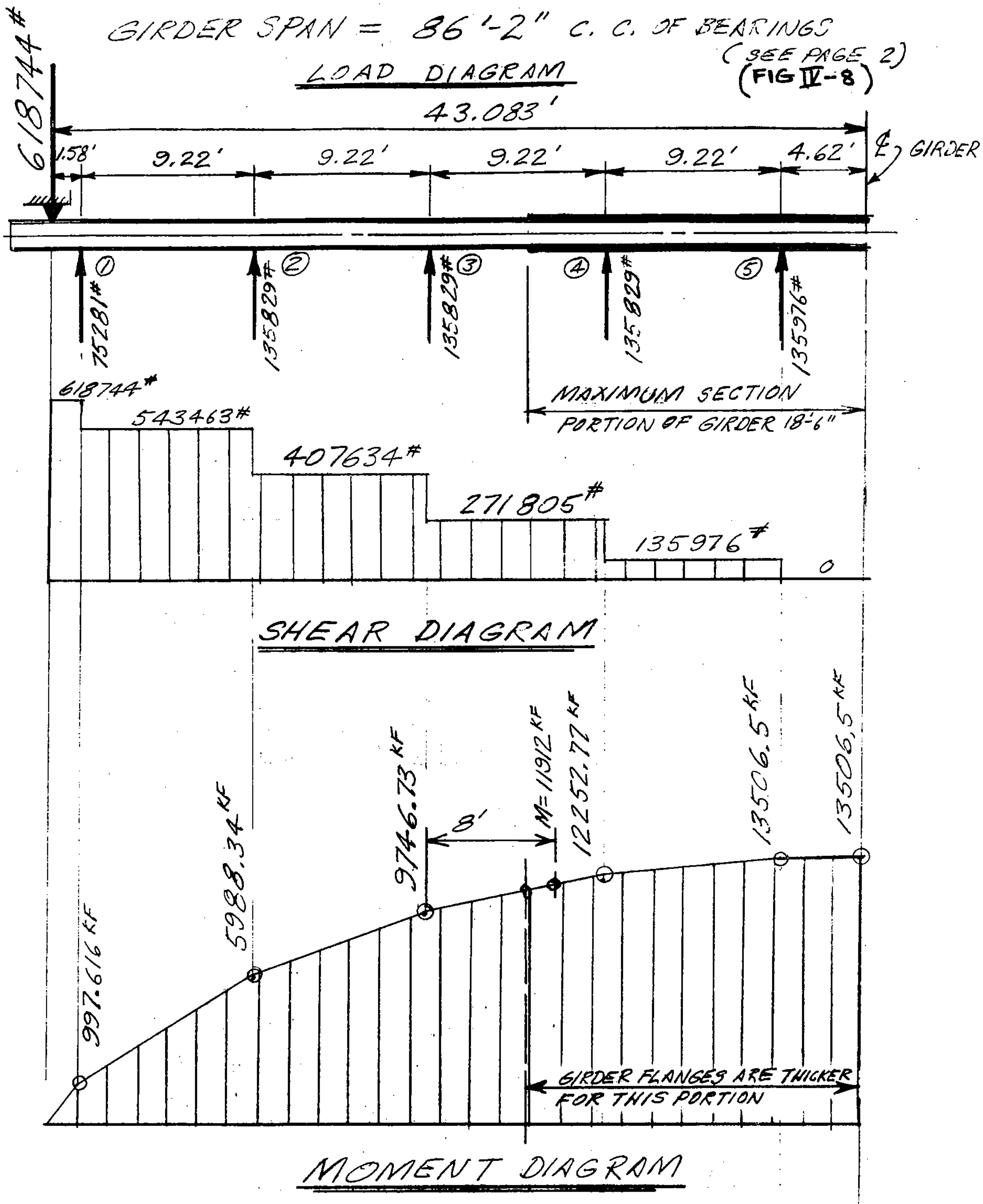


FIG IV-15

PROJECT			COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.		Page 10 of 31	EJM	MAY 70
SUBJECT	EMPIRE FLOOD GATE - GATE DESIGN		CHECKED BY H.O.	DATE MAY 70

I FOR THICK FLANGE PORTION OF GIRDER

A. PRELIMINARY WEB DESIGN:

REFERENCE AISC

1. ASSUME A WEB OF 65 INCHES ($h=72$)
   CHECK FOR NO REDUCTION IN FLANGE STRESS — 1.10.6

   $h/t \leq 162$ TRY 5/8" TH PL. FOR WEB — FOR A-36 STEEL
   CORRESPONDING WEB RATIO $= 65/.625 = 104$

2. MINIMUM WEB THICKNESS $= \frac{65}{320} \cong 1/4"$ — 1.10.2 PG. 5-72

   TRY WEB PLATE 65 X 5/8; $A_{WEB} = 40.62\square"$ $h/t = 104$

B. PRELIMINARY FLANGE DESIGN:

APPENDIX PG 5-67

1. REQUIRED FLANGE AREA:

   ASSUME FLANGES 3 1/2 IN. THICK W/$F_b=24000$ — CALCULATIONS PG. 1

   $$A_f \cong \frac{M}{hF_b} - \frac{A_w}{6} = \frac{13506.5(12)}{72(24)} - \frac{42.5}{6} = 86.7\square"$$

   TRY: 28" X 3 1/2" PL.; $A_f = 98\square" > 86.7\square"$ OK

   CHECK FOR LOCAL BUCKLING: — 1.9 5-72 PG.

   $b/t < 16 \Rightarrow 28/3.5 = 8 < 16$ ∴ OK

C. TRIAL GIRDER SECTION

1 WEB = 65" x 5/8" PL
2 FLANGES = 28" x 3 1/2" PL

1. CHECK BY MOMENT OF INERTIA METHOD — 1.10.1

SECTION	A IN²	Y IN	Ay² IN⁴	Io IN⁴	Igr
65 x 5/8 WEB	40.62			14303	14303
28 x 3 1/2 FLANGE	98	34.25	229908	200	230108
28 x 3 1/2 FL	98				
					244411

EFFECTIVE MOMENT OF INERTIA

FIG IV-16

PROJECT NEW ORLEANS TO VENICE, LA.	Page 11 of 31	COMPUTED BY E.V.M.	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE – GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

I CONTINUED: — REFERENCE

SECTION MODULUS FURNISHED.

$$S = \frac{I}{c} = \frac{244411}{36} = 6789 \text{ IN}^3$$

SECTION MODULUS REQUIRED

$$S = \frac{M}{f_b} = \frac{13506.5^{KF}(12''/')}{24 \text{ KSI}} = 6753.2 \text{ IN}^3$$

2. CHECK LATERAL BUCKLING:

MAXIMUM f_b AT MIDSPAN:

$$f_b = \frac{M}{S} = \frac{13506.5(12)}{6789} = 23.874 \text{ KSI}$$

MOMENT OF INERTIA OF FLANGE PLUS 1/6 WEB

$$I_{oy\text{-}y} = \frac{3.5(28)^3}{12} = 6402 \text{ IN}^4$$

$$A_f + \tfrac{1}{6}A_w = 98 + 40.62/6 = 104.77 \text{ IN}^2$$

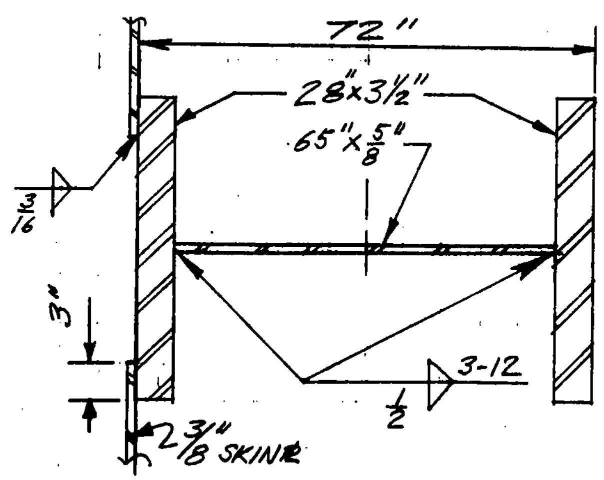
RADIUS OF GYRATION:

$$r = \sqrt{\frac{I_y}{A_f + \frac{1}{6}A_w}} = \sqrt{\frac{6402}{104.77}} = 7.8 \text{ IN.}$$

CHECK BENDING STRESS IN 9.25 FT. PANEL — EM 1110-1-2101

$$\text{COMPUTE: } \frac{l}{r} = \frac{9.25(12)}{7.8} = 14.23 < 40$$

∴ NO REDUCTION NECESSARY



DESIGN FLANGE WELDS

$V = 135976^{\#}$

$$N_1 = \frac{VQ}{I} = \frac{135976(98)(34.25)}{244411} = 1860^{\#}/''$$

$$N_2 = \frac{7.33'}{2}\left(\frac{593^{\#}/\square'}{12''/'}\right) = 180^{\#}/''$$

$2040^{\#}/''$

USE 1/2" WELDS (5300#/")

2 – 1/2" x 3 WELDS @ 12" (2650#/")

FIG IV-17

PROJECT NEW ORLEANS TO VENICE, LA,	Page 12 of 31	COMPUTED BY EJM	DATE MAY 73
SUBJECT EMPIRE FLOODGATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 73

II FOR THIN FLANGE PORTION OF GIRDER

TRIAL GIRDER SECTION

REFERENCE

ASSUME FLANGES 28" x 3" PL A = 84 □"

ASSUME WEB = 66 x 5/8" PL A = 41.25 □"

M MAXIMUM = 11912 KF @ 28.02 FROM BRG. PAGE 9 OF CALCS.

SECTION	A IN^2	Y IN	AY^2 IN^4	I_o	I_{GR}
66 x 5/8 WEB	41.25			14974	14974
28 x 3 FLANGE	84	34.5	200004	126	200130
28 x 3 FLANGE					

EFFECTIVE I_{TOTAL} = 215104

SECTION MODULUS FURNISHED:

$$S = 215104/36 = 5975 \text{ IN}^3$$

SECTION MODULUS REQUIRED:

$$S = 11912(12)/24 = 5956 \text{ IN}^3 \quad \therefore OK$$

CHECK LATERAL BUCKLING

MAXIMUM f_b @ 28.03' FROM BRG.

$$f_b = \frac{11912\,(12)}{5975} = 23.92 \text{ KSI}$$

MOMENT OF INERTIA OF FLANGE PLUS 1/6 WEB

$$I_{oY\text{-}Y} = \frac{3(28)^3}{12} = 5488 \text{ IN}^4$$

$$A_f + 1/6\,A_w = 84 + 11(5/8) = 90.875 \text{ □"}$$

$$r = \sqrt{\frac{5488}{90.875}} = 7.77 \text{ OR } 7.8 \text{ IN}$$

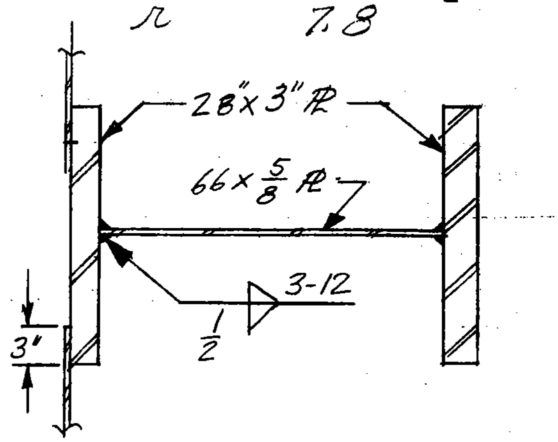
FIG IV-18

PROJECT	NEW ORLEANS TO VENICE, LA.	Page 13 of 31	COMPUTED BY EVM	DATE MAY 70
SUBJECT	EMPIRE FLOOD GATE — GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

II CONTINUED

CHECK ALLOWABLE BENDING STRESS IN 9.22' PANEL

$$\frac{\ell}{r} = \frac{9.22(12)}{7.8} = 14.18 < 40 \therefore \text{NO STRESS REDUCTION NECESSARY}$$



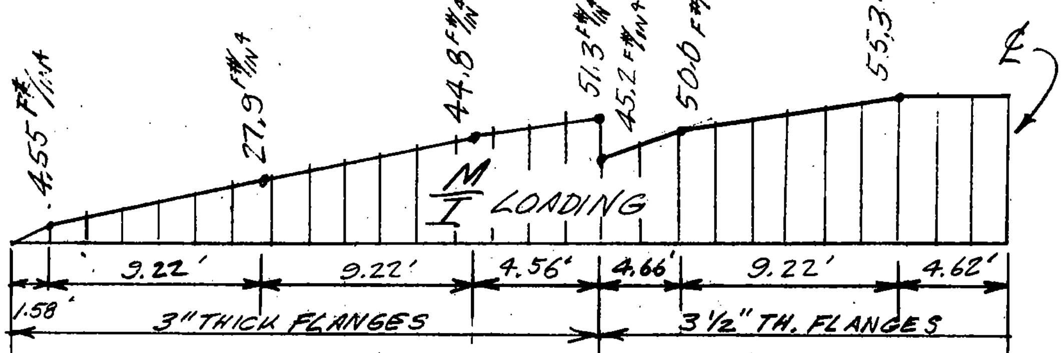
SEE PAGES 11, 17 AND 18 FOR REQUIRED WELDS. (FIG. IV-17, IV-23, & IV-24)

END REACTION ON PL GIRDER = 618744#

$$\text{END SHEAR} = \frac{618744}{5/8 \times 66} = 14{,}999 \text{ PSI}$$

ALLOWABLE SHEAR = 12000 (1.33) = 16000 PSI

DEFLECTION OF GIRDER



FORMULAS FOR A TRAPEZOID:

$$A_{TRAPEZOID} = \frac{d(b+b_1)}{2}$$

$$C_{TRAPEZOID} = \frac{d(2b+b_1)}{3(b+b_1)}$$

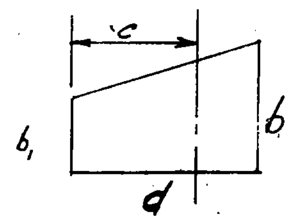


FIG IV - 19

PROJECT NEW ORLEANS TO VENICE, LA.	Page 14 of 31	COMPUTED BY E J M	DATE MAY 70
SUBJECT EMPIRE FLOODGATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

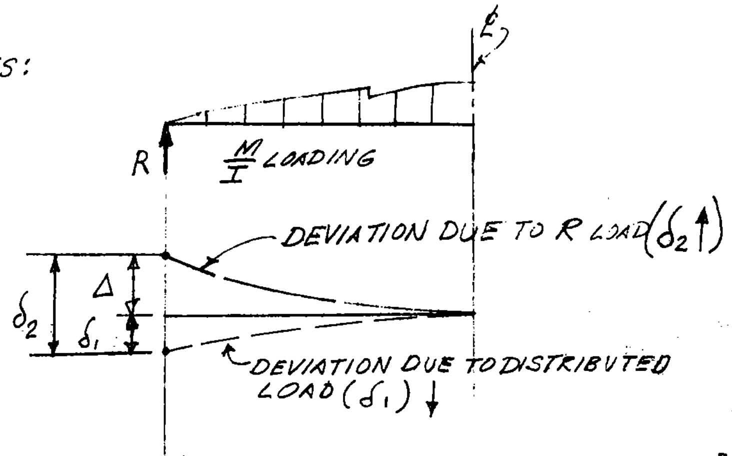
DEFLECTION OF GIRDER - CONTINUED.

FROM M/I DIAGRAM:

$\Delta_{MAXIMUM} = \delta_2 - \delta_1$

DISTRIBUTED LOAD MOMENTS:

3.6 x 42.02 =		151
150 x 35.78 =		5367
335 x 27.31 =		9700
219 x 20.73 =		4550
221 x 16.13 =		3570
486 x 9.14 =		4450
255 x 2.31 =		590
$1669.6 \frac{\#FT^2}{IN^4}$	=	$28378 \frac{\#FT^3}{IN^4}$



$E\delta_1 = 28378 \, \#FT^3/IN^4$

$E\delta_2 = 1669.6(43.08) = 71926.4 \, \#FT^3/IN^4$

$R = 1669.6$

$E\Delta = 71926.4 - 28378 = 43548.4^{\#}$

$$\Delta = \frac{43548.4 \, \#FT^3/IN^4 \, (12 \, IN/FT)^3}{30 \times 10^6 \, \#/IN^2} = 2.5''$$

$\Delta = 2.5 \, IN$ @ MIDSPAN. $\approx \frac{1}{400}$ of ℄ SPAN

FIG IV-20

GIRDER DESIGN CONTINUED:

STIFFENER REQUIREMENTS

REF.

STIFFENERS ARE REQUIRED AT EACH VERTICAL BEAM.

CHECK INTERMEDIATE STIFFENERS — AISC 1.10.5.3

SHEAR STRESS IN PANEL 4-5 — PG. 9 OF CALCS.

$$f_v = \frac{135976}{65 \times 5/8} = 3350 \text{ PSI}$$

$$\frac{h}{t} = \frac{65}{.625} = 104 < 260$$

$$C_v = \frac{45 \times 10^6 k}{F_y (h/t)^2} = \frac{45 \times 10^6 (6.71)}{36 \times 10^3 (104)^2} = .776 \qquad 1.10.5.2$$

$$k = 5.34 + \frac{4.00}{(a/h)^2} = 5.34 + \frac{4.00}{(\frac{111}{65})^2} = 6.71$$

$C_v < 1 \Rightarrow$ USE FORMULA 9 FOR WEB SHEAR

$$F_v = \frac{F_y}{2.89}(C_v) = \frac{36 \times 10^3 (.77)}{2.89} = 9.59 \times 10^3 \text{ PSI} \qquad 1.10.5.2$$

NO INTERMEDIATE STIFFENER REQUIRED — 1.10.5.3

CHECK WEB SHEAR IN PANEL 3-4

$$V = 271805^{\#} \quad f_v = \frac{271805}{.625(65)} = 6690 \text{ PSI}$$

NO INTERMEDIATE STIFFENERS NECESSARY — 1.10.5.3

CHECK WEB SHEAR IN PANEL 2-3

$$V = 407634^{\#} \quad f_v = \frac{407634}{66(.625)} = 9882 \text{ PSI}$$

ALLOWABLE WEB SHEAR — TABLE 3-36

$$a/h = 111/66 = 1.68 \qquad h/t = 66/.625 = 105.6$$

$$\text{FOR } a/h = 1.8 \qquad h/t = 110 \Rightarrow F_v = 10.3 \text{ KSI}$$

$\therefore$ NO STIFFENERS REQUIRED — 1.10.5.3

FIG IV - 21

PROJECT NEW ORLEANS TO VENICE, LA.	Page 16 of 31	COMPUTED BY EVM	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

GIRDER DESIGN - CONTINUED:

CHECK WEB SHEAR IN PANEL 1-2

REFERENCE

$V = 543463\,\#$

$f_v = \dfrac{543463\#}{66(.625)} = 13175 \text{ PSI}$

TRY AN ASPECT RATIO OF .84 — TABLE 3-36

$a = 66(.84) = 55.4'' \text{ SAY } 55\tfrac{1}{2}''$

$h/t = 66/.625 \cong 106$

CALCULATE FOLLOWING FOR FORMULA 9 — 1.10.5.2

$k = 4.00 + \dfrac{5.34}{(\frac{a}{h})^2} = 4.00 + \dfrac{5.34}{(.84)^2} = 4.00 + 7.56 = 11.57$

$C_v = \dfrac{6000}{h/t}\sqrt{\dfrac{k}{F_Y}} = \dfrac{6000}{106}\sqrt{\dfrac{11.57}{36000}} = 1.014 \approx 1.0$

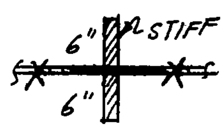
$F_v = \dfrac{F_Y}{2.89} C_v = = \dfrac{36 \times 10^3 (1.014)}{2.89} = 12631 \text{ PSI}$

$12631 \approx 13175 \therefore OK$

STIFFENER SIZE

TRY 2 PLS 3/8" x 6"

CHECK WIDTH-THICKNESS — 1.9



$6/3/8 = 16$ OK

CHECK REQUIRED MOMENT OF INERTIA — 1.10.5.4

$I_{REQ} = \left(\frac{h}{50}\right)^4 = \left(\frac{66}{50}\right)^4 = 3.04 \text{ IN}^4$

$I_{x\text{-}x} \text{ FURN.} = \frac{1}{12}(.375)(12.625)^3 = 62.9 \text{ IN}^4 \therefore OK$

REQUIRED STIFFENER LENGTH

USE 2 PLS 6 x 3/8 BEARING ON COMPRESSION FLANGE AND CUT SHORT 1" FROM TENSION FLANGE.

FIG IV- 22

LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT	Page 17 of 31	COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA		EVM	MAY 70
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE – GATE DESIGN		N.U.	MAY 70

INTERMEDIATE STIFFENER FOR PANEL 1-2-CON'T

REQUIRED WELD FOR STIFFENER

$$f_{vs} = h\sqrt{\left(\frac{F_y}{3400}\right)^3} = 66\left[\left(\frac{36000}{3400}\right)^3\right]^{1/2} = 2273 \ \#/\text{LIN. IN.}$$ REF. 1.10.5.4.

USE 1/4" FILLET WELDS: EM 1110-1-2101 PG 10

$500\#/\tfrac{1}{16}"\left(\tfrac{4}{16}"\right)(1.33) \Rightarrow 2660\ \#/''$

TRY 1/4 x 3" @ 6" WELDS BOTH SIDES
FOR 6" OF LENGTH:
SHEAR TRANSFER REQ. = 2273(6) = 13638#
SHEAR TRANSFER FURN. = 2660(3)(2) = 15960#

WELDS ON PLATE GIRDER

PT 4 TO ℄ PAGE 9 OF CALCS.

$V = 135976\#$ $\quad v_H = \frac{VQ}{I}$

HORIZ. SHEAR ON WELD: $\frac{98(34.25)(135976)}{244420}$

TENSION FROM SKINPLATE LOADING = 180# PG 11 OF CALCS.

SHEAR TRANSFER REQUIRED = 1860 + 180 = 2040#/''
SHEAR TRANSFER OF 2 - 1/2" x 3 WELDS @ 12" = 2650#/''

POINT 3 TO 4

$V = 271805\#$ $\quad$ HORIZ SHEAR = $\frac{271805(98)(34.25)}{244420} = 3700\#$

TENSION FROM SKINPLATE 180#
FOR 3" TH. FLANGE: 3880

HORIZ. SHEAR = $\frac{271805(84)(34.5)}{214437} = 3673\#/''$

TENSION FROM SKINPLATE = 180#/''
3830#/''

WELD SHEAR FURNISHED:
USE 1/2" WELDS BOTH SIDES (5300#/'')

USE 2 - 1/2 x 5" WELD @ 12" ⇒ 4417#/''

FIG IV-23

PROJECT NEW ORLEANS TO VENICE, LA.	Page 18 of 31	COMPUTED BY EJM	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

GIRDER WELDS - CONTINUED

REF.

POINT 2 TO 3 V = 407634#

HORIZ. SHEAR = $\frac{407634(84)(34.5)}{214437} = 5509$ #/"

TENSION DUE TO WATER ON SKINPLATE = 180 #/"

5689 #/"

USE 1/2" WELDS (5300 #/")

2 WELDS 1/2" x 7" @ 12" (6183 #/")

POINT 1 TO 2 V = 543463

HORIZ SHEAR = $\frac{543463(84)(34.5)}{214,437} = 7344.6$ #/"

TENSION DUE TO WATER ON SKINPLATE = 180 #/"

7524 #/"

USE 1/2 WELDS: (5300 #/")

2 WELDS 1/2" X 9" @ 12 ⇒ 7950 #/"

POINT 0 TO 1 V = 618744#

HORIZ. SHEAR = $\frac{618744(84)(34.5)}{214437} = 8362$ #/"

TENSION DUE TO WATER ON SKINPLATE = 180 #/"

8721 #/"

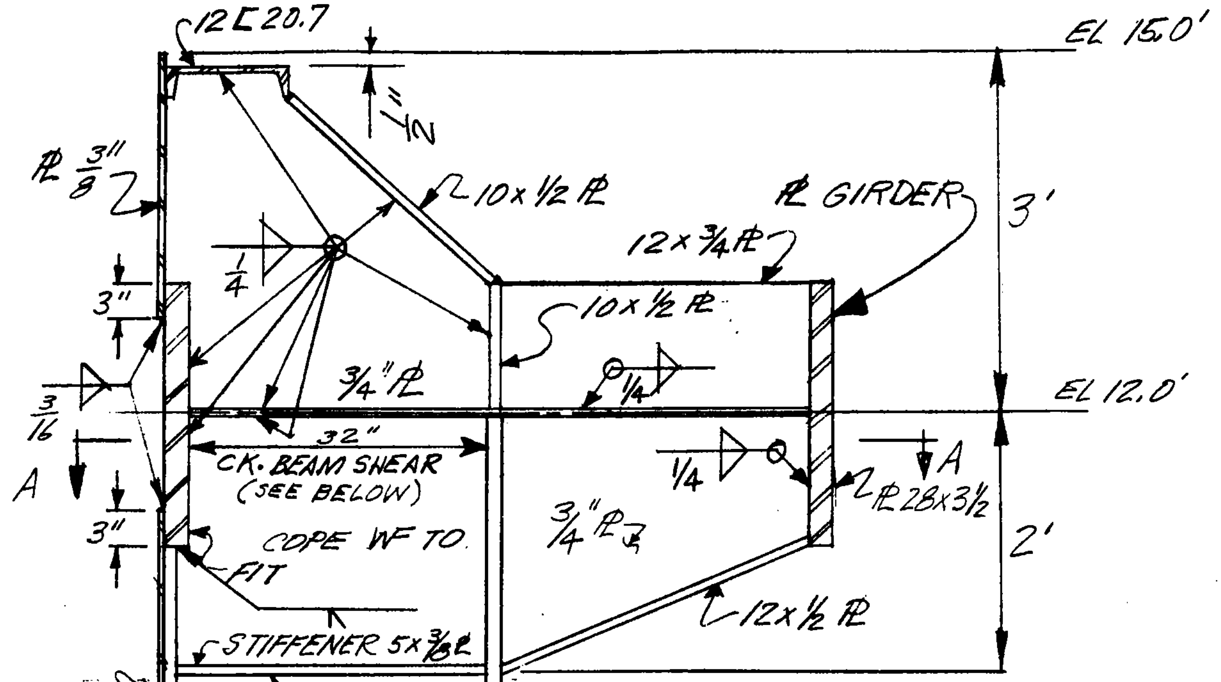
USE 1/2 WELDS (5300 #/")

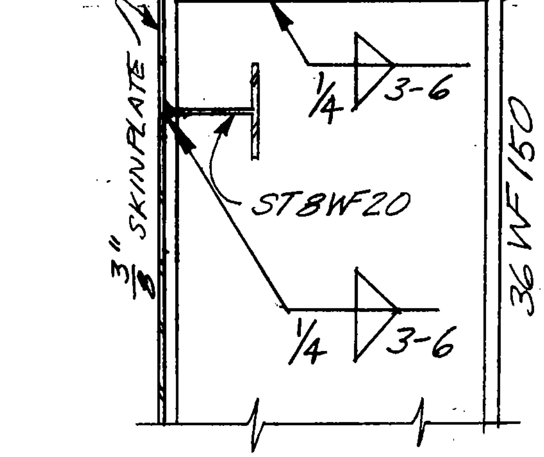
2 WELDS 1/2 x 10" @ 12" ⇒ 8833 #/"

FIG. IV-24

PROJECT	Page 19 of 31	COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.		EVM	MAY 70
SUBJECT		**CHECKED BY**	**DATE**
EMPIRE FLOODGATE - GATE DESIGN		H.U.	MAY 70

INTERIOR SECTION OF GATE AT VERTICAL BEAM





INTERIOR BEAM-GIRDER CONNECTION

MAXIMUM SHEAR ON 36WF150

V @ EL 12.0' (SEE PAGE 6)

$V = (14732 - 1549)(9.23) = 121681^{\#}$

USE WELDS ON BOTH SIDES:

REQUIRED WELD SHEAR:

$\frac{121681}{2(31.9)} \simeq 1907 \,\#/''$

USE 1/4" WELDS (2660 #/")

CHECK BENDING STRESS OF GIRDER FLANGE IN Y-Y AXIS

APPROX WT. OF GIRDER $\cong$ 800 #/'

$M_{MAXIMUM} = 800\left(\frac{9.25^2}{8}\right) = 8556'^{\#}$

$S_{YY} = 2\frac{bh^2}{6} = 3(28)^2/6\,(2) = 784 \text{ IN}^3$

$f_{b_{YY}} = 8556(12)/784 = 130 \text{ PSI}$

FIG III-25

LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA	Page 20 of 31	EVM	MAY 70
SUBJECT: EMPIRE FLOODGATE, GATE DESIGN		CHECKED BY: W.U.	DATE: MAY 70

PLATE GIRDER DESIGN - CONTINUED

DESIGN OF END BEARING STIFFENERS

REF

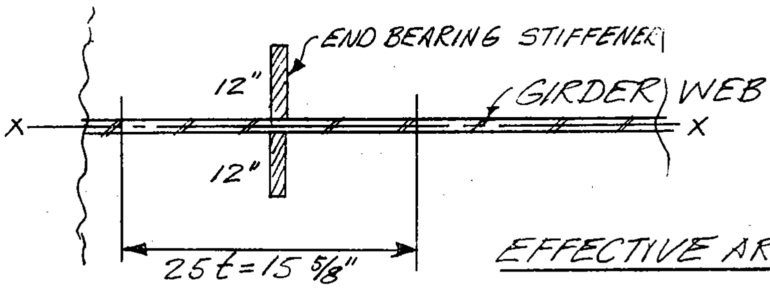
END REACTION = 618744#

TRY 2 PLS 12" X 3/4"

a. CHECK WIDTH-THICKNESS RATIO — AISC 1.9

$\frac{12}{.75} = 16$ OK

b. CHECK COMPRESSIVE STRESS (END BEARING) — 1.10.5.1



EFFECTIVE AREA

$I_{x\text{-}x} = \frac{.75(24.625)^3}{12} = 933\ IN^4$

$A_{EFF.} = 2(12)(.75) + 25(.625)^2 = 27.77\ \square''$

$r = \sqrt{I/A} = [933/27.77]^{1/2} = 5.79$ IN.

EFFECTIVE LENGTH = 3/4 (66) = 49.5 IN. — 1.10.5.1

$l/r = \frac{49.5}{5.79} = 8.55$; $F_a = 24000$ PSI * — TABLE 1-36

*(21210 (1.33) > 24000)

STRESS PRESENT IN STIFFENER

$f_a = 618744/27.77 = 22300$ PSI

BEARING ON STIFFENERS = 618744/2x12x3/4 = 34300

WELD ON BEARING STIFFENERS

SHEAR PRESENT = 618744/2(66) = 4680#/"

USE 1/4" WELDS (TWO) ⇒ (5320#/")

SEE SKETCH ON PAGE 24 (FIG. IV-30)

FIG IV-26

PROJECT	Page 21 of 31	COMPUTED BY	DATE
NEW ORLEANS TO VENICE		RES	1-12-70
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE - GATE DESIGN		EJM	JUN 70

CHAIN CONN TO GATE FOR LIFT CHAIN

USE $3\frac{1}{2}''\phi$ ROUND PIN ANCHOR SHACKLE WITH $3\frac{1}{2}''\phi$ PIN

CONN PLATE

$$\text{LOAD} = \frac{\text{CHAIN B.L.}^{*}}{\text{S.F.}} = \frac{548}{2} = 274^{k}$$

$$\text{MIN REQ'D NET AREA} = \frac{274}{13.5} = 20.3^{\square''}$$

$$\text{FOR } 3\tfrac{1}{2}''\phi \text{ PIN, MIN } t = \frac{274}{27 \times 3.5} = 2.90''$$

TRY PL $10'' \times 4''$

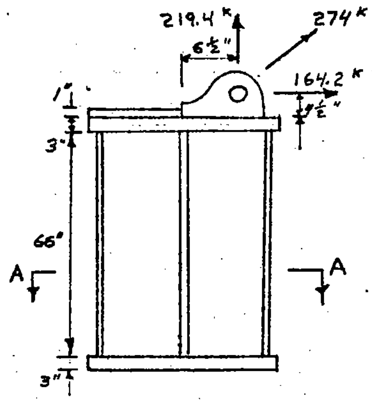
$$\text{TRANS. NET AREA} = 4(10-3.625) = 25.5^{\square''}$$

$$\text{BRG STRESS} = \frac{274}{3.5 \times 4} = 19.6^{ksi} < 27^{ksi} \quad \text{O.K.}$$

$$\text{NET SEC PARALLEL TO LOAD} = [5.5 - 1.81]\,4 = 14.76^{\square''}$$

$$\tfrac{2}{3} \text{ OF REQ'D NET AREA} = \tfrac{2}{3} \times 20.3 = 13.53 < 14.76 \quad \text{ok}$$

USE PL $10'' \times 4''$ [CUT TO $5\frac{1}{2}''$ RADIUS]



STIFFENER DESIGN

TRY $2 - 1'' \times 12\frac{1}{2}''$ PL $+ 2 - 1'' \times 3''$ PL (+WEB $\frac{5}{8}'' \times 15\frac{5}{8}''$)

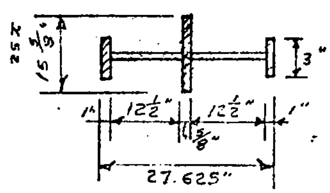
$$A = 2 \times 1 \times 12.5 + 2 \times 1 \times 3 + .625 \times 15.625 = 40.8^{\square''}$$

$$I_x = \tfrac{1}{12} \times 1 \times 25.625^3 + 2 \times 3 \times 13.81^2 = 2547 \qquad r_x = \sqrt{\frac{2547}{40.8}} = 8.1$$

$$\frac{\ell}{r} = \frac{66}{8.1} = 8.15$$

$$I_y = \tfrac{1}{12} \times \tfrac{5}{8} \times 15\tfrac{5}{8}^3 + 2 \times \tfrac{1}{12} \times 1 \times 3^3 = 201 \qquad r_y = \sqrt{\frac{201}{40.8}} = 2.2$$

$$\frac{\ell}{r} = \frac{66}{2.2} = 30$$



SECTION A-A

$$f_a = \frac{P}{A} = \frac{219.4}{40.8} = 5.38^{ksi} \qquad F_A = 18.0$$

$$f_b = \frac{Mc}{I} = \frac{[164.2 \times 4.5 + 219.4 \times 6.5]\,13.81}{2547} = 11.7^{ksi}$$

$$\text{Form (7b)} \quad \frac{f_a}{.5F_y} + \frac{f_b}{F_B} \le 1$$

$$\frac{5.38}{18} + \frac{11.7}{18} = \frac{17.08}{18} < 1 \quad \text{ok}$$

$$F_B = \frac{10{,}000}{\frac{\ell d}{A_F}} = \frac{10{,}000}{\frac{66 \times 27.625}{3}} = 16.5 > 11.7 \quad \text{ok}$$

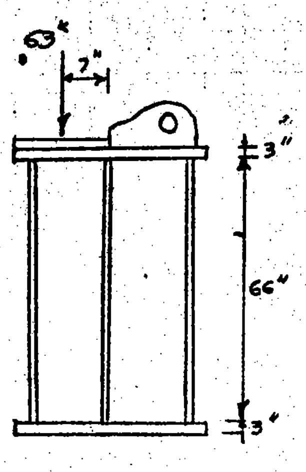
$$f_{SHEAR} = \frac{V}{A} = \frac{164.2}{40.8} = 4.0 < 12 \quad \text{ok}$$

USE $2 - 1'' \times 12\frac{1}{2}''$ PL $+ 2 - 1'' \times 3''$ PL

* CHAIN BREAKING LOAD FOR $2\frac{1}{8}''$ CHAIN. FROM "BALDT ANCHORS AND CHAIN" CATALOG NO 29D, PG 31

FIG II-27

SHOCK ABSORBER LOAD ON LIFT CHAI[illegible] [illegible]NN.



$$MOM = 7 \times 63 = 441^{\prime\prime\text{-}K}$$

$$f_a = \frac{P}{A} = \frac{63}{41.8} = 1.51$$

$$F_a = 16.6^{KSI}$$

$$\frac{f_a}{F_A} = \frac{1.51}{16.6} = .09 < .15 \therefore \text{USE FORM (6)}$$

$$f_B = \frac{Mc}{I} = \frac{441 \times 14.31}{2718} = 2.32^{KSI}$$

$$F_B = \frac{10{,}000}{\frac{66 \times 28.625}{3}} = 15.9$$

$$\frac{f_a}{F_A} + \frac{f_B}{F_B} \leq 1$$

$$\frac{1.51}{16.6} + \frac{2.32}{15.9} = .09 + .15 = .24 < 1 \quad OK$$

FIG IV-28

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA	Page 23 of 31	RCS	MAY 1970
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE - GATE DESIGN		EJM	MAY 70

CHAIN CONN. TO GATE FOR CWT. CHAIN

USE 2"Ø ROUND PIN ANCHOR SHACKLE w/ $2\frac{1}{4}$"Ø PIN PER FED. SPEC. RR-C-271a., TYPE IV CLASS 5

CONN. ℞ FOR 70^k LOAD $\left[\frac{\text{CHAIN } F_u}{F.S.} = \frac{198}{2.8} = 70^k\right]$

MIN REQ'D NET AREA $= \frac{70}{13.5} = 5.19^{\square''}$

FOR $2\frac{1}{4}$"Ø PIN, MIN REQD $T = \frac{70}{27 \times 2\frac{1}{4}} = 1.15^{\square''}$

TRY ℞ 8"x 2"

TRANS. NET AREA $= (8-2.375)2 = 11.25^{\square''} > 5.19^{\square''}$ OK

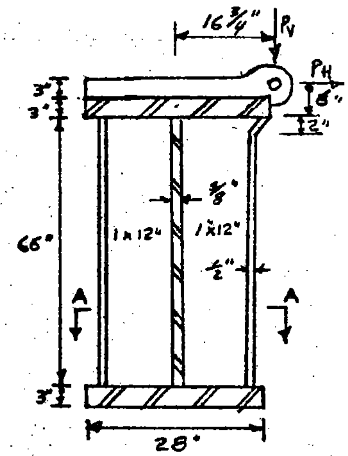
BRG. STRESS $= \frac{70}{2 \times 2\frac{1}{4}} = 15.6^{ksi} < 27^{ksi}$

NET SEC. PARALLEL TO LOAD $= [8-2.375]2 \times .5 = 5.625^{\square''}$

2/3 OF REQ'D MIN AREA $= \frac{2}{3} \times 5.19 = 3.46^{\square''} < 5.625^{\square''}$ OK

USE ℞ 8"x 2" (CUT ON 4" RADIUS)

STIFFINER DESIGN



$L = 66"$ $P_V = 69.95^k$ $P_H = 2.7^k$

$M = 16.75 \times 69.95 + 3 \times 2.7 = 1180^{''-k}$

TRY 2 ℞ 1"x12" + 2 ℞ 3"x½" + WEB ℞ 7¼"x⅝"

$A = 2 \times 1 \times 12 + 7.25 \times .625 + 1.5 \times 2 = 31.5^{\square''}$

$I_x = \frac{1}{12} \times 1 \times 24.625^3 + 2 \times 1.5 \times 12.56^2 = 1717^{in^4}$

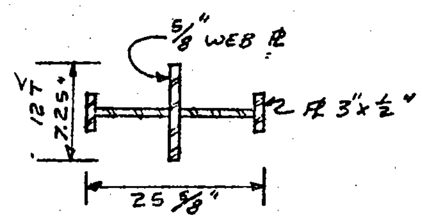
$r_x = \sqrt{\frac{1717}{31.5}} = 7.4$ $\quad \frac{\ell}{r} = \frac{66}{7.4} = 8.9$

$I_y = \frac{1}{12} \times \frac{5}{8} \times 7.25^3 + 2 \times \frac{1}{2} \times \frac{1}{12} \times 3^3 = 22$

$r_y = \sqrt{\frac{22}{31.5}} = .84$ $\quad \frac{\ell}{r_y} = 79$

$f_a = \frac{P}{A} = \frac{69.95}{31.5} = 2.22$ $\quad F_a = 12.7$

$\frac{f_a}{F_A} = \frac{2.22}{12.7} = .17 > .15$ (USE FORM 7a.)



SECTION A-A

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left[1 - \frac{f_a}{K_y F_e'}\right] F_b} \leq 1$$

$C_m = 0.85$
$K_y = 0.83$
$C_c = 126.1$

$f_b = \frac{Mc}{I} = \frac{1180 \times 12.81}{1717} = 8.8^{ksi}$

$F_e' = \frac{149,000}{(K\frac{\ell}{r})^2} = 188.1^{ksi}$

$A' = \frac{1}{6} \times 24 \times 1 + 3 \times \frac{1}{2} = 5\frac{1}{2}^{\square''}$ $\quad I' = \frac{1}{12} 4 \times 1^3 + \frac{1}{12} \times \frac{1}{2} \times 3^3 = 1.5$

$r' = \sqrt{\frac{1.5}{5.5}} = .53$ $\quad \frac{\ell}{r'} = 125$ $\quad K_2 = 1 - \frac{125^2}{2 \times 1.75 \times (126.1)^2} = .72$

FORM 4 $F_b = 18 \times .72 = 13.0^{ksi}$ $\quad$ FORM 5 $F_b = \frac{10000}{\frac{\ell d}{A_F}} = 8.9 < 13.0$

$$\frac{f_a}{F_A} + \frac{C_m f_b}{\left[1 - \frac{f_a}{K_y F_e'}\right] F_b} = \frac{2.22}{12.7} + \frac{.85 \times 8.8}{\left[1 - \frac{2.22}{.83 \times 188.1}\right] 13.0} =$$

$.17 + .58 = .75 < 1$ OK

USE 2 ℞ 1"x12" + 2 ℞ 3"x½"

FIG IV-29

LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.	Page 24 of 31	EJM	MAY 70
SUBJECT		**CHECKED BY**	**DATE**
EMPIRE FLOOD GATE - GATE DESIGN		H.U.	MAY 70

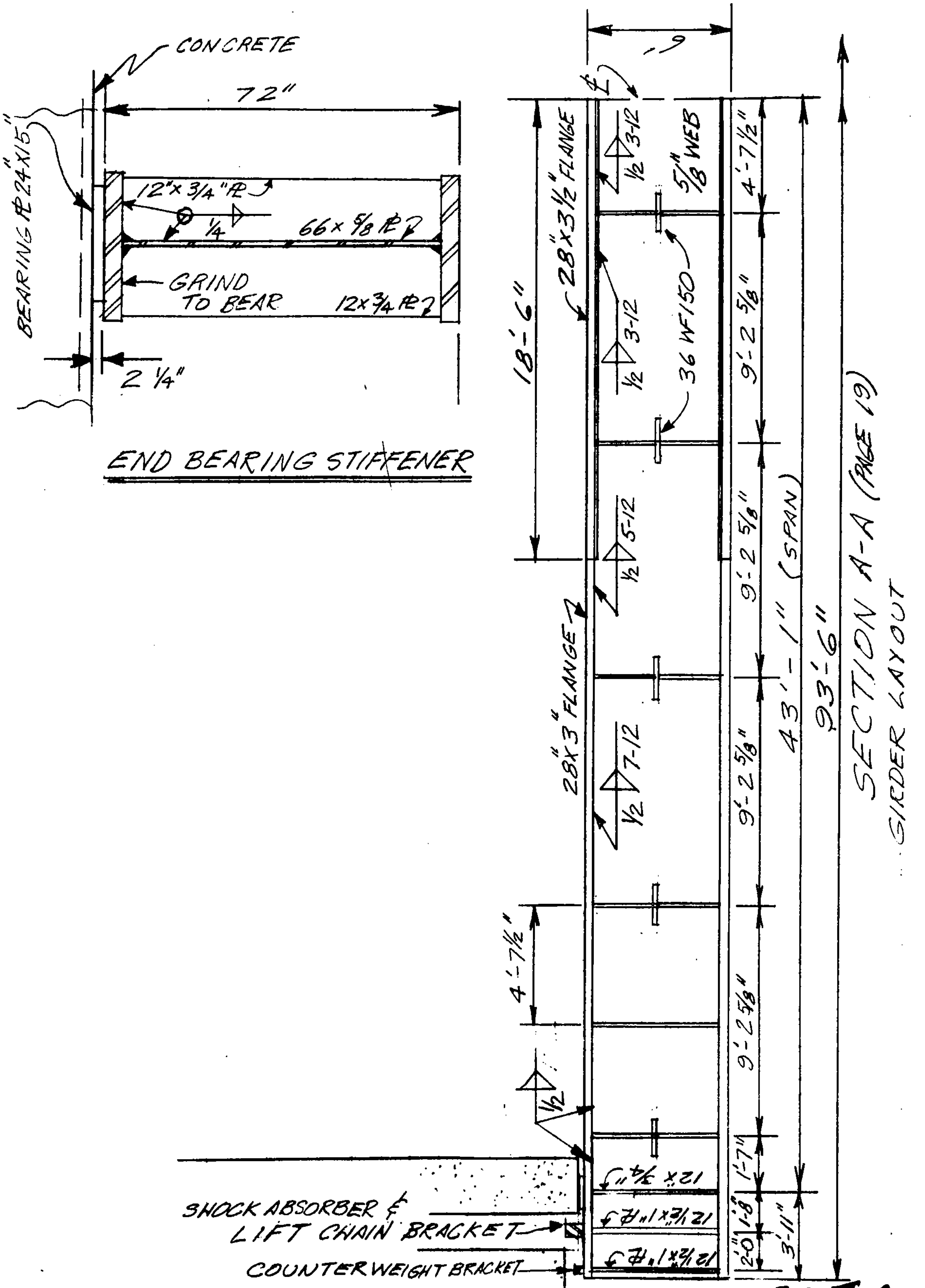


FIG II-30

PROJECT NEW ORLEANS TO VENICE	Page 25 of 31	COMPUTED BY EJM	DATE MAY 70
SUBJECT EMPIRE FLOOD GATE – GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

GIRDER WEIGHT:

2 PLS 3½ x 28" = 666 #/1
WEB 5/8" x 65" = 138 #/1

804 #/1

2 PLS 3" x 28" = 572 #/1
WEB 5/8 x 66" = 140 #/1

712 #/1

GATE WEIGHT & CENTROIDS

ITEM	WT.	$\bar{X}$ FR SKPL	Mx #"	$\bar{Y}$ FR. TOP	My #FT.
804 x 37 (GIRDER)	29748	36"	1,070 928	3.0	89244
712 x 56.5 "	40228	36"	1448 148	3.0	120 684
8 PLs 12 x ¾ x 5'-6"	1346	36"	48 400	3.0	4038
6 PLs 12 x ¾ x 2'-9"	505	52.5"	26,500	2.47	1250
4 PLs 12 x ¾ x 2'-8½"	331	52.25"	17,300	2.47	820
6 PLs 12 x ¾ x 2'-8½"	497	19.25"	9560	2.47	1230
4 PLs 12 x ¾ x 2'-8½"	325	19.50"	6350	2.47	800
10 PLs ¾ x 1'-0" x 1'-1"	586	6.00"	3520	1.00	586
5 PLs ¾ x 1'-8" x 1'-3½"	446	18.33"	8230	1.00	440
6 PLs 12 x ¾ x 2'-9"	505	52.50"	26510	3.53	1730
4 PLs 12 x ¾ x 2'-8½"	331	52.25"	17300	3.53	1160
3 PLs 14 x ¾ x 2'-9"	295	47.00"	13 900	4.42	1305
2 PLs 14 x ¾ x 2'-8½"	193	46.83"	9 100	4.42	855
10 PLs 10 x ½ x 2'-6"	425	21.0"	8 930	2.08	885
10 PLs 10 x ½ x 1'-0"	170	35.75"	6100	2.47	420
10 PLs 12 x ½ x 3'-0"	612	52.50"	32 000	4.58	2800
20 PLs 5 x ⅜ x 2'-10"	361	18.00"	6 500	5.33	1920
10 - 36 WF150 x 26'-0"	39 000	18.00"	702 000	16.00	624000
2 - 36 WF150 x 83'-0"	24 900	18.00"	448 200	22.5	560 250
8 - ST8WF20 x 83'-0"	13280	6.18"	82 070	16.125	214 140
1 - 12 [20.7 x 83'-0"	1718	6.00"	10 308	0.06'	103
SK PL ⅜ x 82'-6" x 27'-0"	34080	-0.188	- 6400	16.37	557890
BRG PL 2-2¼ x 15" x 24"	459	-1.125	- 516	3.00	1100
SEAL PL 2 x ⅜ x 142'-6"	360	-1.188	- 400	23.2	8350
TOTALS	190685 #		3993796 "#		2196327 '#

$\bar{X}$ = 20.94" $\bar{Y}$ = 11.518'

FIG IV-31

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.	Page 26 of 31	E J M	MAY 70
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE – GATE DESIGN		H.U.	MAY 70

TOTAL GATE WEIGHT BREAKDOWN

PLATE GIRDER	69976 #
STIFFENERS AND CHAIN SUPPORTS	6912 #
VERTICAL BEAMS + LATTERAL BRACING	63900 # (36 WF 150)
SKINPLATE SUPPORTS	14998 #
SKINPLATE	34080 #
GATE	189 866 #
BEARING PLATES	459 #
SEAL PLATES	369 #
TOTAL GATE WEIGHT	190 685 #

HINGES – DESIGN

MAX. SPACING OF HINGES = 9'-3"

MAX. VERTICAL LOAD 20 100 #
MAX. HORIZONTAL LOAD 124 000 #
MAX. HORIZONTAL /W SUBMERGED GATE WT = 18000 #

MAXIMUM LOAD ON PIN = 125.4 KIPS

BEARING ALLOWABLE ON BUSHING (NO GATE MOVEMENT)
$f_p = 6000$ PSI

REQUIRED BEARING AREA $= \frac{125400}{6000} = 21 \square''$

USE 7" ϕ PIN ON 3" WIDE BUSHING.

FIG II-32

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.	Page 27 of 31	EJM	MAY 70
SUBJECT EMPIRE FLOODGATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

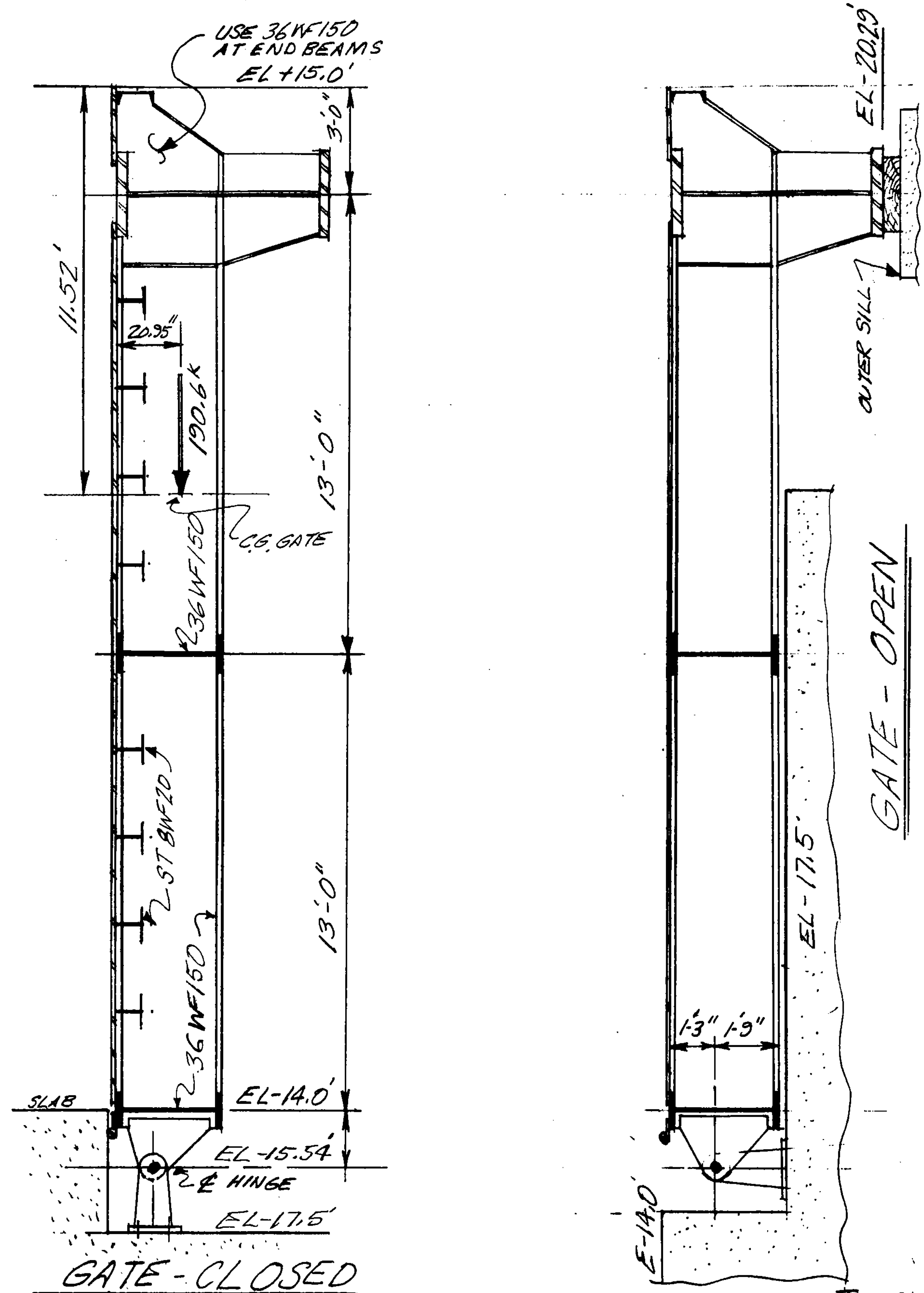
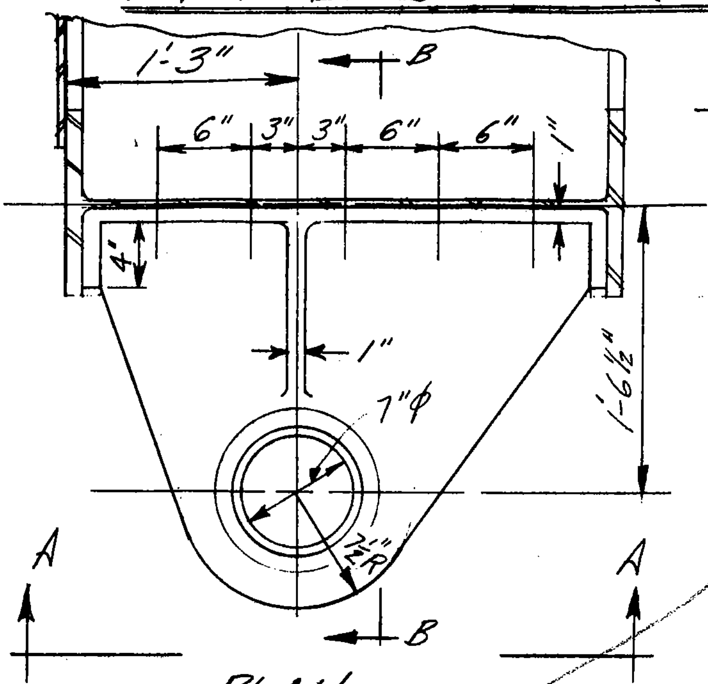


FIG IV-33

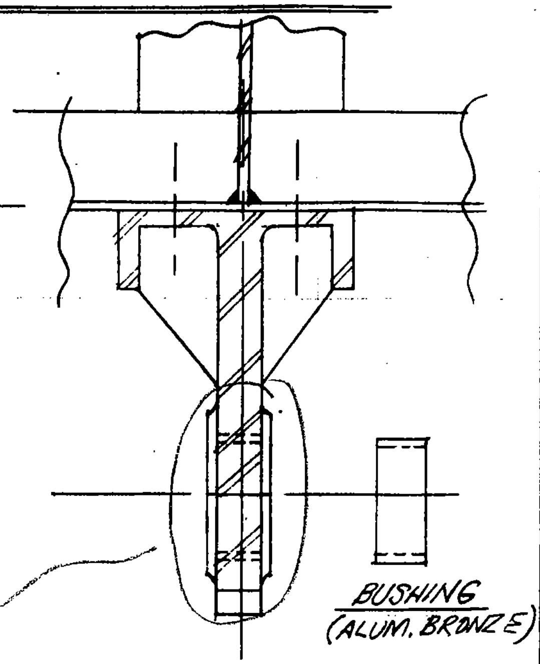
LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT	Page 28 of 31	COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA		EJM	MAY 70
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE - GATE DESIGN		N. U.	MAY 70

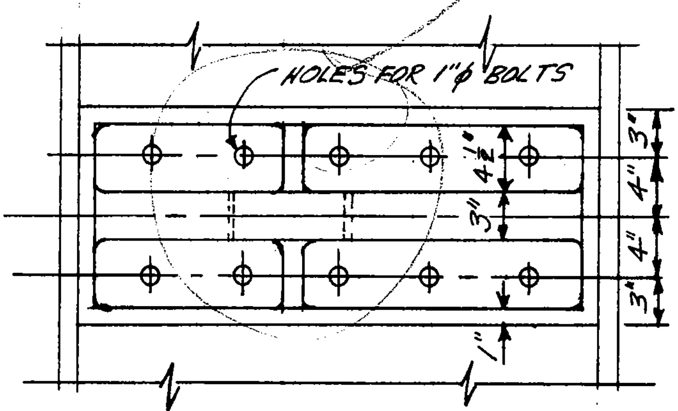
HINGE DESIGN-CONTINUED



PLAN
UPPER HINGE SUPPORT



SECTION
UPPER HINGE SUPPORT



SECTION A-A
UPPER HINGE SUPPORT

APPROX. WT. OF CASTING
700# ±

UPPER HINGE CASTING

SHEAR ON BOLTS

$$\frac{124000}{10\,(.7854)} = 15700\ PSI$$

A325 BOLTS

BEARING ON 5/8" WEB

$$\frac{124000}{5/8 \times 1 \times 10} = 20000\ PSI$$

LATERAL LOAD ON CASTING

$$M = 125.4^{k}(0.15')(1.33) = 24.9^{k'}$$

$$S = \tfrac{1}{6}(3^2)(32) = 48\ IN^3$$

$$f_b = \frac{24900(12)}{48} = 6225\ \#/\square''$$

FIG IV-34

PROJECT		COMPUTED BY	DATE
NEW ORLEANS TO VENICE, LA.	Page 29 of 31	EJM	MAY 70
SUBJECT		CHECKED BY	DATE
EMPIRE FLOODGATE – GATE DESIGN		N.U.	MAY 70

HINGE ASSEMBLY - SHOWING HINGE-BASE CASTING EMBEDED IN CONCRETE.

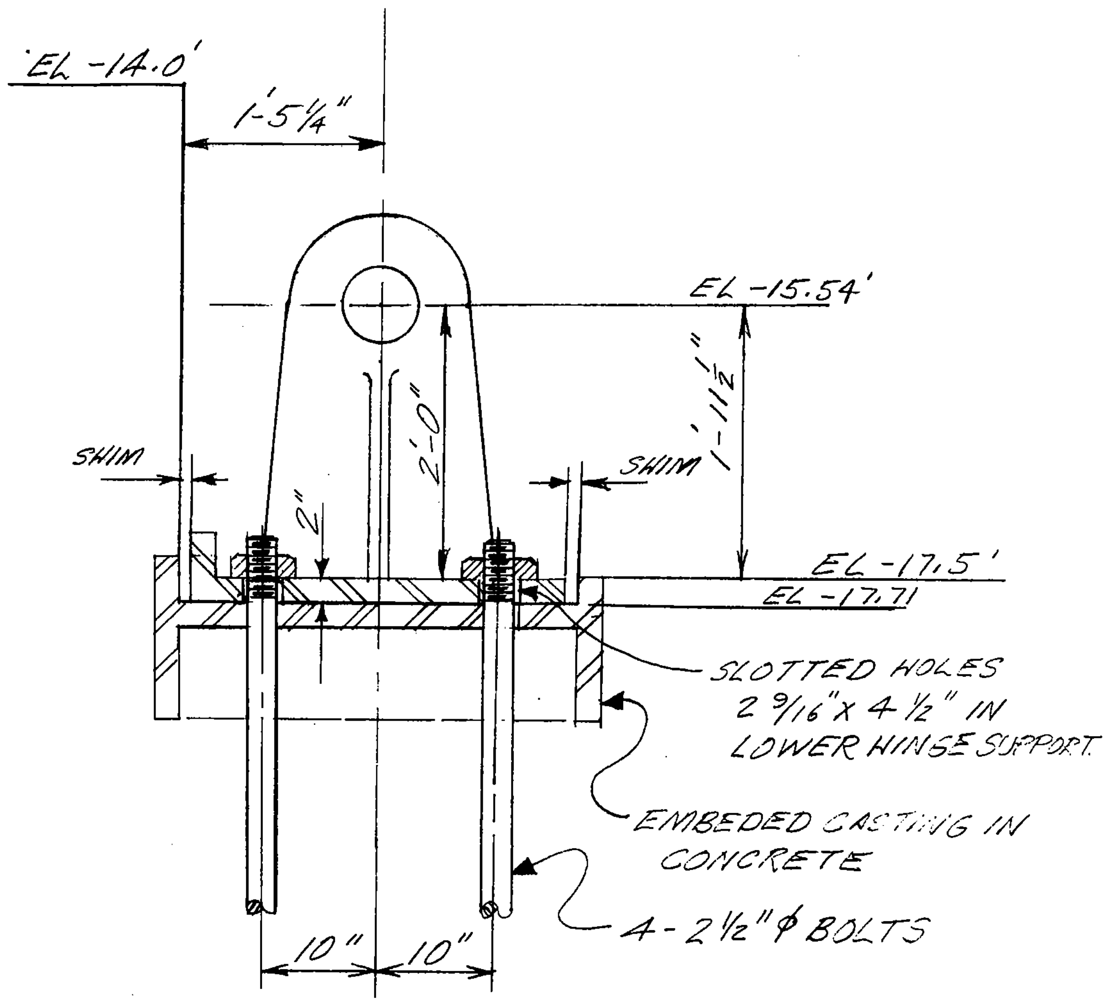
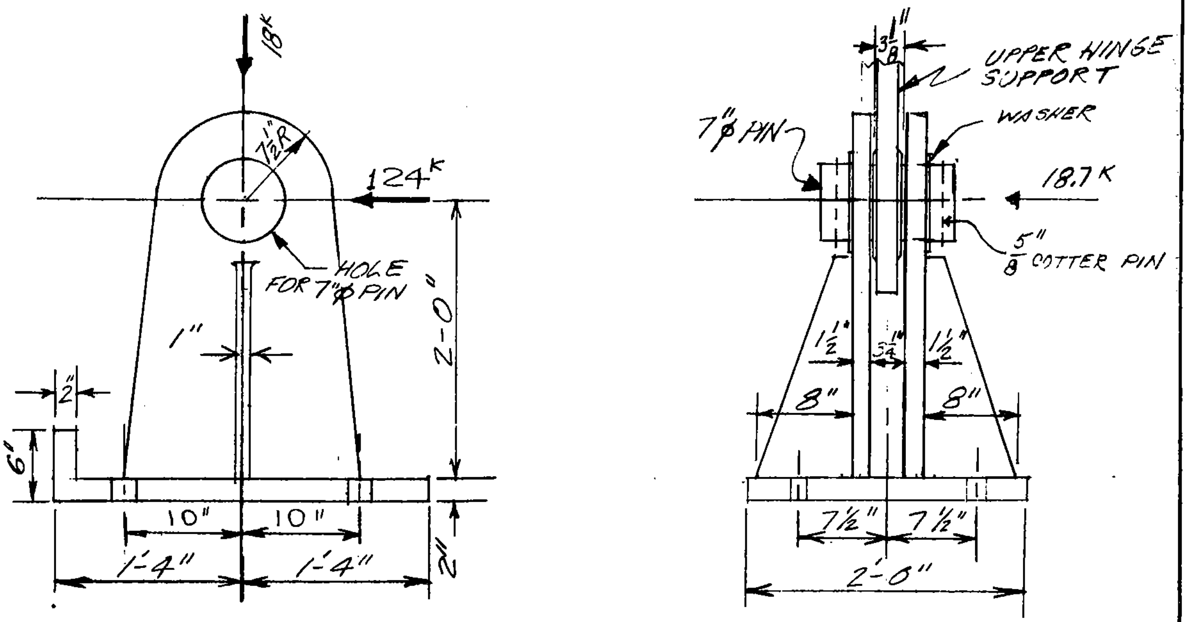


FIG IV-35

PROJECT NEW ORLEANS TO VENICE, LA.	Page 30 of 31	COMPUTED BY EVM	DATE MAY 70
SUBJECT EMPIRE FLOODGATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

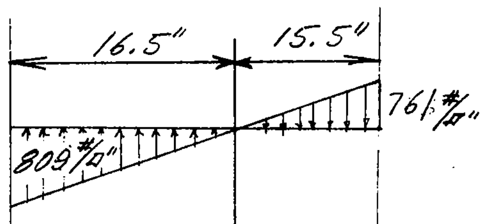


7"φ PIN - STAINLESS STEEL W/ALUM. BRONZE WELDED ON.

LOWER HINGE SUPPORT

3/4" = 1'-0"

HINGE SUPPORT DESIGN



$$P = \frac{18000}{32(24)} \pm \frac{124000(26)}{1/12\ 24(32)^3}$$

$$P = 24 \pm 785 = \begin{cases} 809\ \#/\square'' \\ 761\ \#/\square'' \end{cases}$$

BOLT LOAD:

$1/2\,(761)(15.5)(24) = 142000\#$

USE 2 BOLTS $2\frac{1}{2}''\phi$ ($A = 4\square''$)

$$f_T = \frac{142000}{2(4)} = 17{,}800\ PSI$$

MOMENT @ TOP OF BASE:

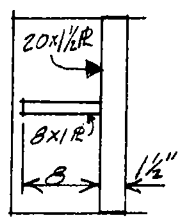
$M = 124000(2) = 248^{K'}$

$S = 2(1/6)(20)^2(1.5) = 200\ IN^3$

$$f_b = \frac{248000(12)}{200} = 14900\ PSI$$

DESIGN AGAINST 18.7^K LOAD

$M = 18700(2) = 37400\ \#'$



$\underline{I\ IS:}$

$.30(.75)$	$= 22.5$
$8(5.5)$	$= 44.0$
C.G. 38×1.75	$= 66.5$

$I = +1/12(8)^3(1) = 42.5\ IN^4$

$+ (8)(3.75)^2 = 113.0\ IN^4$

$+ 1/12(2)(1.5)^3 = 5.5\ IN^4$

$S = \frac{I}{c} = \frac{181}{7.75} = 23.35$ $\quad + 20(1)^2 = 20.0\ IN^4$

$181\ IN^4$

$$f_b = \frac{37400(12)}{23.35} = 19220\ PSI$$

FIG. III-36

PROJECT	NEW ORLEANS TO VENICE, LA	Page 31 of 31	COMPUTED BY E.V.M.	DATE MAY 70
SUBJECT	EMPIRE FLOODGATE - GATE DESIGN		CHECKED BY H.U.	DATE MAY 70

HINGE SUPPORT DESIGN - CON'T.

2" BASE PLATE:
BENDING MOMENT AT 6" FROM EDGE

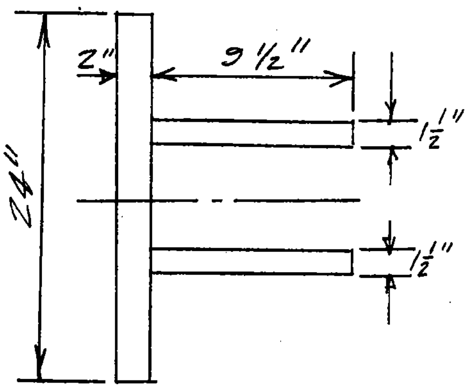
$M = 662(6)(24)(3.22) = 308000''\#$

$S = 1/6\,(24)(2^2) = 16\ in^3$

$f_b = \dfrac{308000}{16} = 19{,}300\ PSI$

MOMENT 15.5" FROM EDGE:

$M = 142\,000\,(15.5)(2/3) = 1{,}470{,}000''\#$



CENTER OF GRAVITY:

$24\times2 = 48.0 \times 1.0 =$		48.0
$9.5\times3 = 28.5 \times 6.75 =$		192.9
$76.5 \times 3.15 =$		240.9

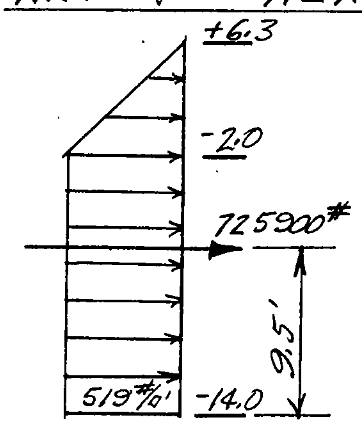
$I = +\ 1/12\,(24)(2^3) = 16.0\ in^4$
$+\ 48\,(2.15)^2 = 221.0\ in^4$
$+\ 1/12\,(3)(9.5)^3 = 213.0\ in^4$
$+\ 28.5\,(3.6)^2 = 369.0\ in^4$

$319.0\ in^4$

$S_{MIN} = \dfrac{819}{8.35} = 97\ in^3$

$f_b = \dfrac{1470000(12)}{97} = 15200\ \#/\square''$

REVERSE HEAD LOAD

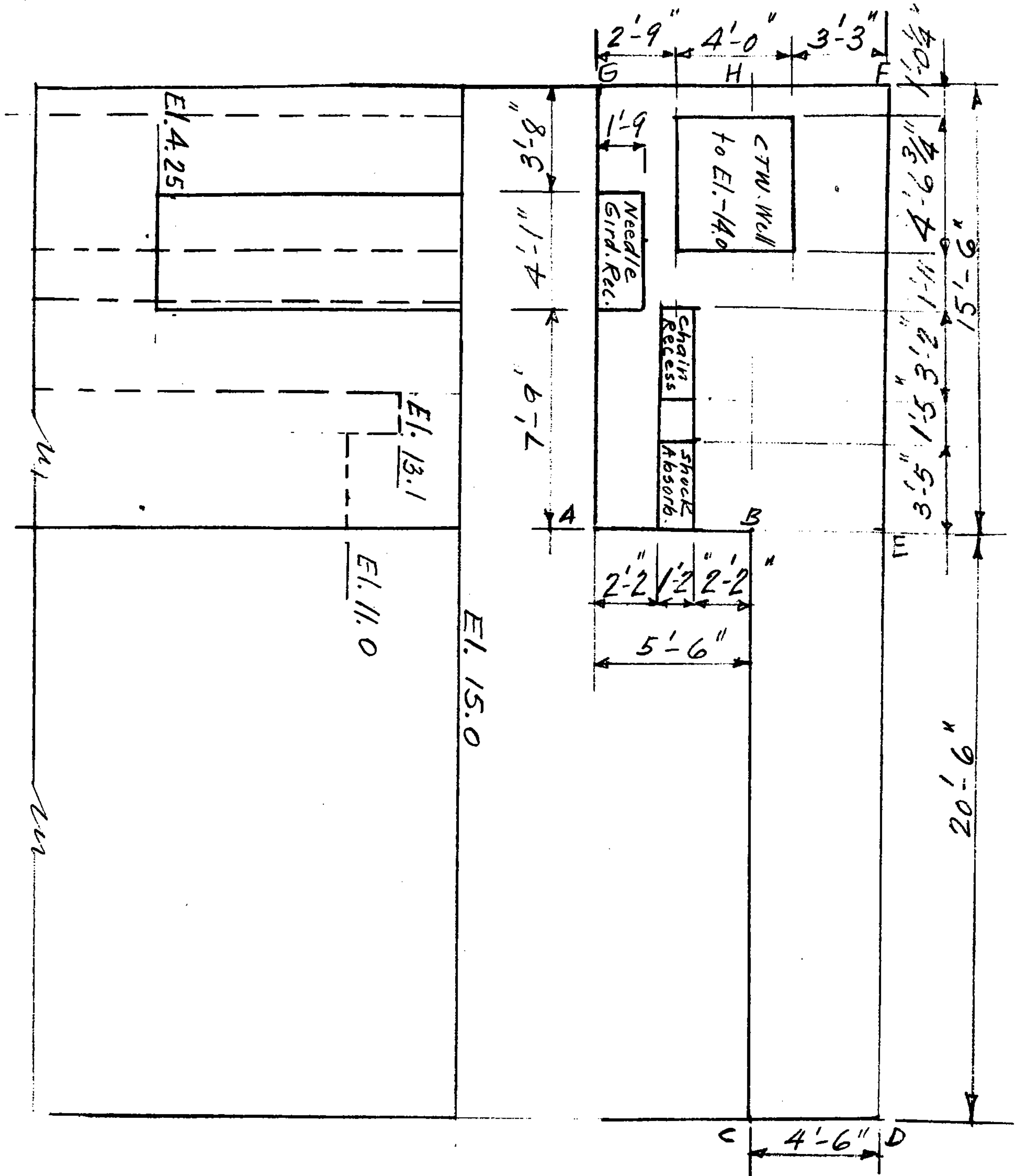


HORIZONTAL HINGE LOAD = $48^k \pm$

NOTE:
PROVISIONS TO BE MADE TO HOLD GATE AT PLATE GIRDER FOR A LOAD OF 250^k FOR REVERSE HEAD.

FIG IV-37

PROJECT New Orleans to Venice, La.	Page 1 of 13	COMPUTED BY H.V.	DATE June 70
SUBJECT Empire Floodgate- Concrete Wall		CHECKED BY DG.M	DATE June 70

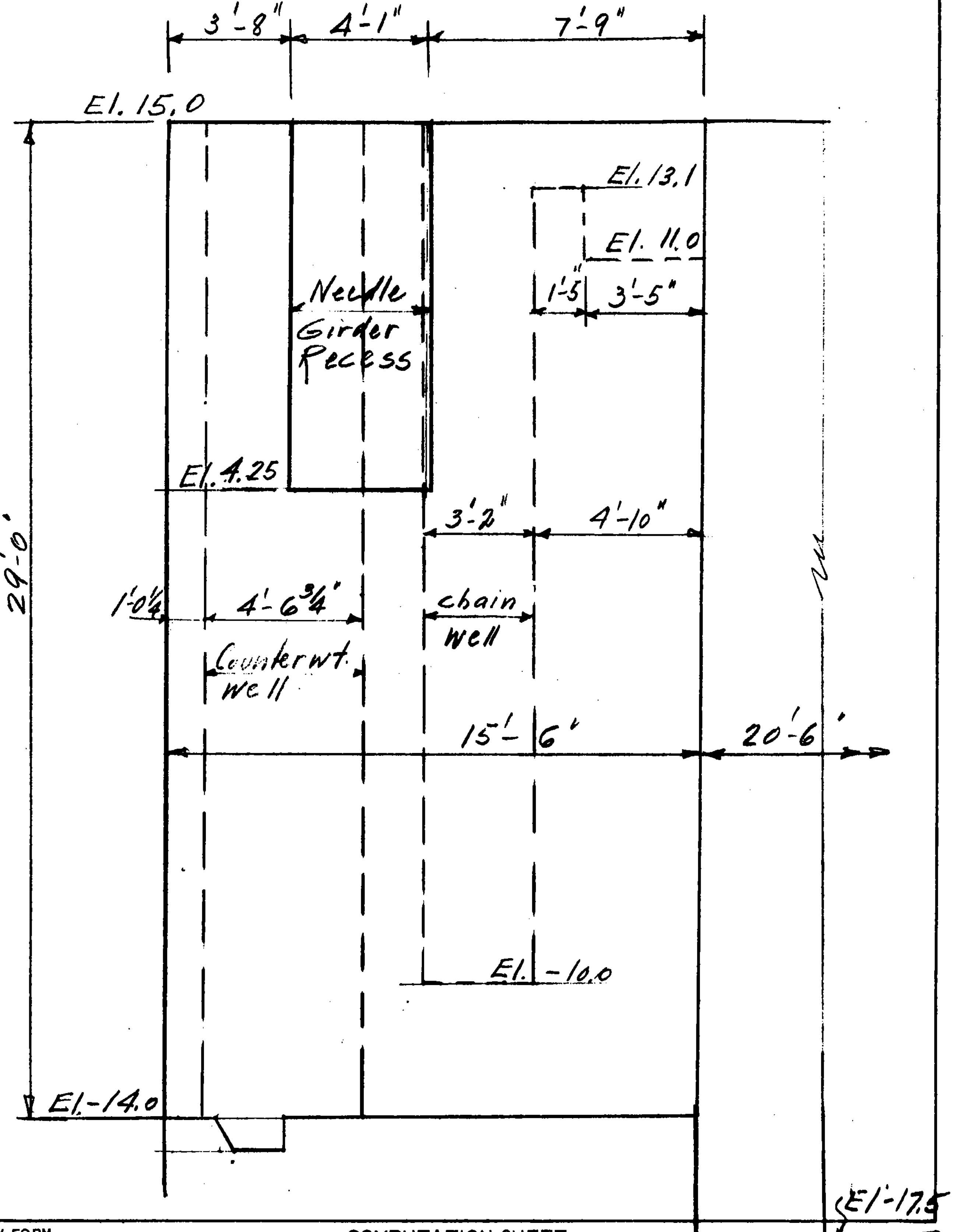


DESIGN OF STEEL GRILLAGE
FOR GATE REACTION

CASES III and IV critical
Group 2 Loading design stresses

FIG IV-38

PROJECT New Orleans to Venice, La	Page 2 of 13	COMPUTED BY H.V.	DATE June 70
SUBJECT Empire Floodgate - Conc. Wall		CHECKED BY [signature]	DATE June 70



PROJECT New Orleans to Venice, La.	Page 3 of 13	COMPUTED BY H.U.	DATE Jun. 70
SUBJECT Empire Floodgate, Concrete Wall		CHECKED BY D.G.M.	DATE June 70

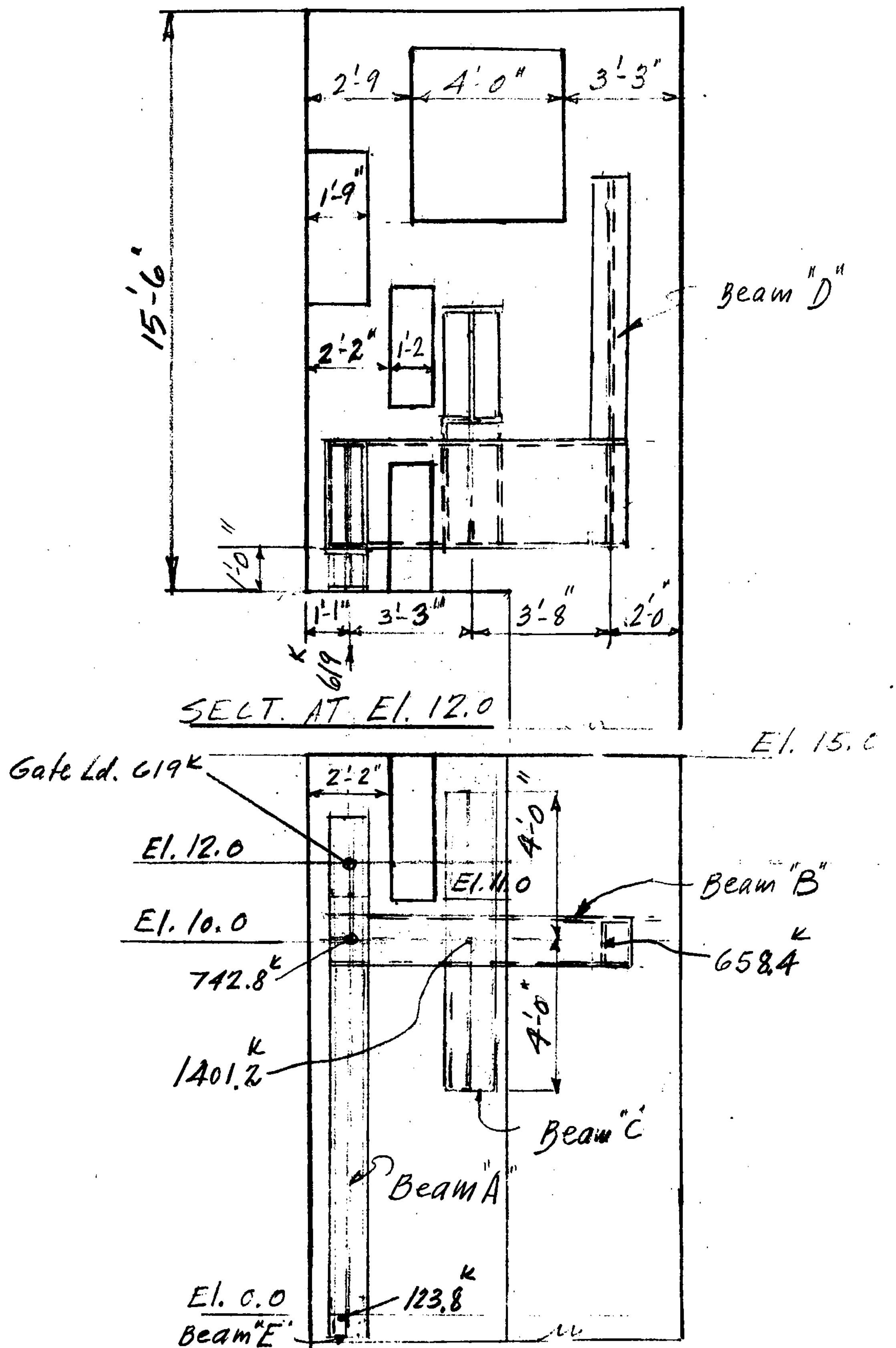
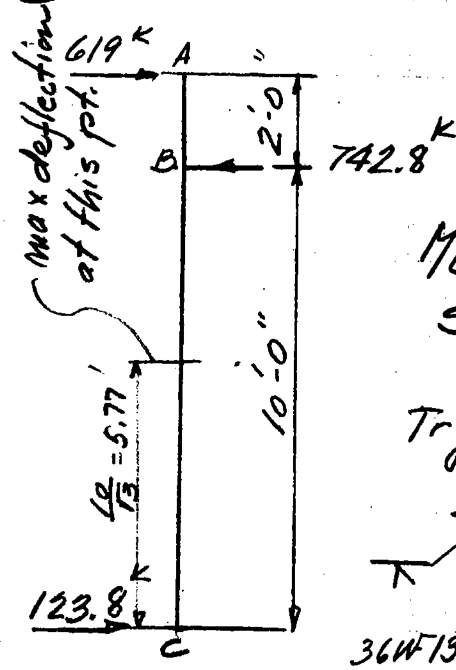


FIG III-40

PROJECT New Orleans to Venice, La. Page 4 of 13 COMPUTED BY H.V. DATE June 70

SUBJECT Empire Floodgate - Concrete Wall CHECKED BY [illegible] DATE June 70

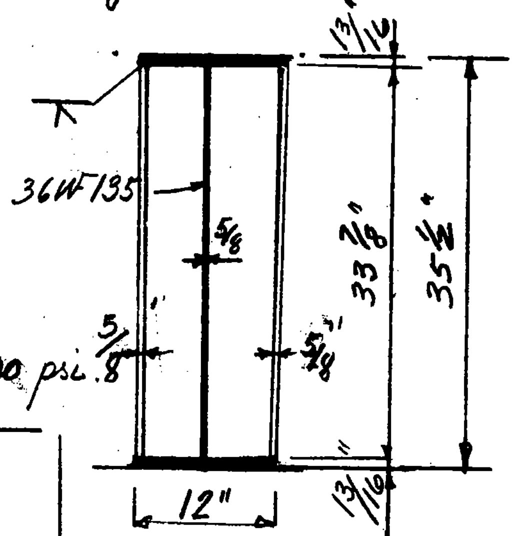
Beam "A":



$M_B = 619^K \times 2.0 = 1238'^K$

$S\ reqd = \frac{1238000 \times 12}{24000} = 619\ in^3$

Try 36 WF @ 135



$V_{max} = 619{,}000^{\#}$

$v = \frac{619000}{1\frac{7}{8} \times 33\frac{7}{8}} = 9{,}700\ psi.$

Deflection of BC:

$\Delta_{max} = \frac{619000 \times 24 \times 120^2}{9\sqrt{3} \times 30 \times 10^6 \times 11846}$

$= 0.041''$ at $\frac{10}{13}$ from C.

5/8" outside webs:

$I = \frac{1}{12} \times 1.0 \times \left(\frac{5}{8}\right)^3$

$A = 5/8 \times 1.0$

$r = \sqrt{\frac{(5/8)^2}{12}} = \frac{0.625}{.346} = 0.18$

$\frac{\ell}{r} = \frac{0.7 \times 33\frac{7}{8}}{0.18} = 130$

allow $8{,}840 \times 1.33 = 11800\ ^{\#}/\square''$

$I = 7796.1$

$+\ 2 \times \frac{1}{12} \times \frac{5}{8} \times 33.875^3 = 4050$

$I = 11846\ in^4$

$S = 670\ in^3$

Wt. 280 #/'

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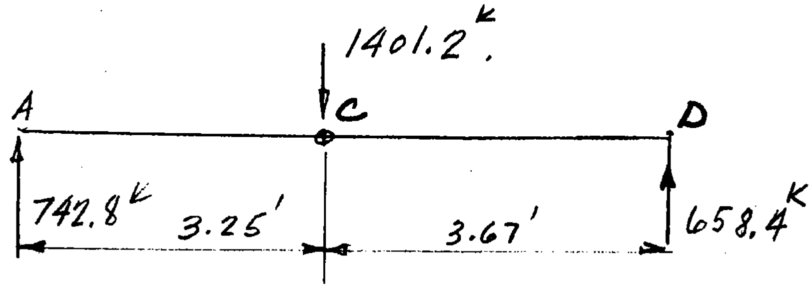
FIG IV-41

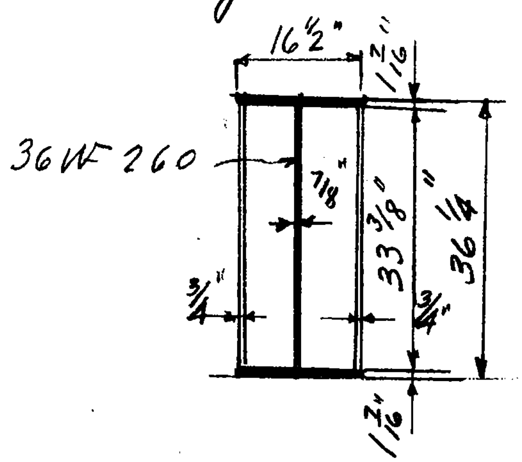
Beam "B":



$$M_{max} = 742.8 \times 3.25 = 2414^{1k}$$

$$S\ reqd. = \frac{2414000 \times 12}{24,000} = 1207\ in^3$$

Try 36 WF 260.



Wt. 430 #/'

$$I = 17233.8$$
$$+ 2 \times \frac{1}{12} \times \frac{3}{4} \times 33.375^3 = 4620.0$$
$$21,853.8\ in^4$$

$$S = 1210\ in^3$$

$$V = 742800^{\#}$$

$$v = \frac{742800}{33\,3/8 \times 2\,3/8} = 9500\ psi$$

Deflection at A

$$\Delta_A = \frac{742800 \times 39^3}{3 \times 30 \times 10^6 \times 21853.8} = 0.023''$$

¾" Web: $I = \frac{1}{12} \times 1.0 \times (\frac{3}{4})^3 = \frac{1}{12} \times 0.422$

$A = 0.75\ \square''$, $r = \sqrt{\frac{.422}{9}} = 0.22''$

$$\frac{\ell}{r} = \frac{33.375 \times 0.7}{0.22} = 106,\ \text{allow } 12,200\ psi. \times 1.33$$

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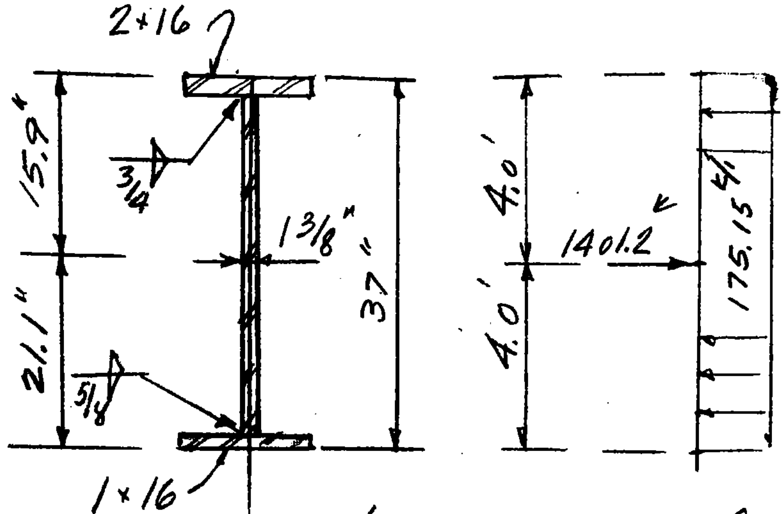
SUBJECT Empire Floodgate, Concrete Wall		CHECKED BY N.G.M.	DATE June 70

Beam "C"



Built-Up Sect

Brg. on Conc. = $\frac{175150}{16 \times 12} = 913$ #/□"

C.g. $32 \times 1.0 = 32.00$
$46.75 \times 19 = 888.25$
$16.00 \times 36.5 = 584.00$
$94.75 \times 15.9" = 1504.25$

Deflection:

$\Delta = \frac{175150 \times 4 \times 48^3}{8 \times 30 \times 10^6 \times 18{,}859}$
$= 0.017"$

$I = \frac{1}{12} \times 2^3 \times 16 = 10.6$
$\frac{1}{12} \times 1^3 \times 16 = 1.4$
$32 \times 14.9^2 = 7104.3$
$\frac{1}{12} \times 1.375 \times 34^3 = 4503.6$
$46.75 \times 3.1^2 = 449.3$
$16 \times 20.6^2 = 6789.8$

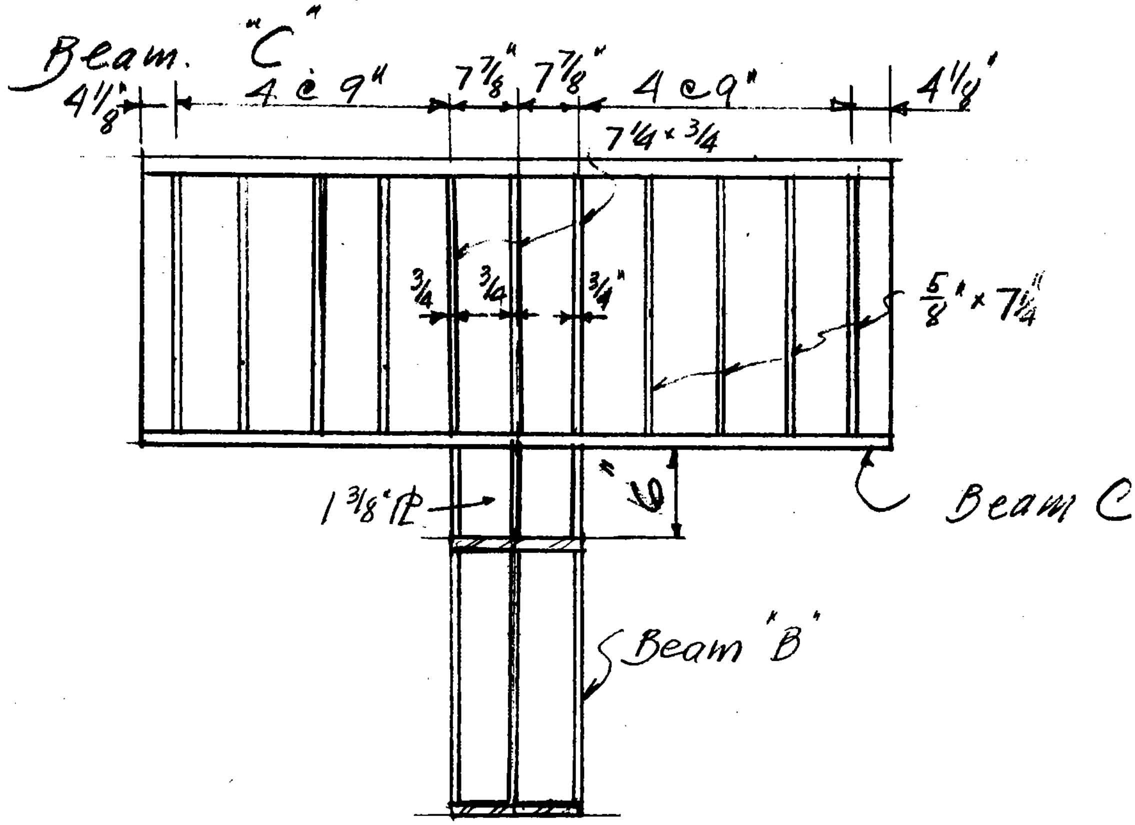
$I = 18{,}859.0$

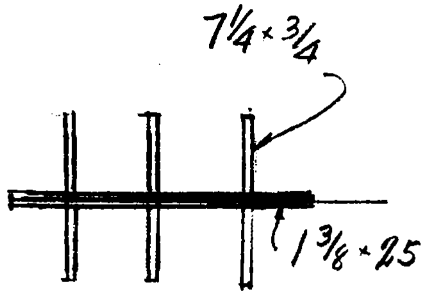
$S_{min} = 893.8 \text{ in}^3$
$S_{max} = 1186.1 \text{ in}^3$.

Max compr. stress = $1{,}401{,}200 \times 12 \div 893.8 = 18{,}800$ #/□"
Max tensile stress = $1401200 \times 12 \div 1186.1 = 14{,}200$ #/□"

Bending on flange: allowed stress $24000 - 14200 = 9{,}800$ #/□"
$M = 913 \times X^2 \div 12$ where X is stiffener spacing
$f_s = 9{,}800 = \frac{913 \times X^2}{12 \times 1/6 \times 2^2 \times 1.0}$; $X^2 = 86$, $X = 9.3"$

PROJECT	Page	COMPUTED BY	DATE
New Orleans to Venice, La.	Page 7 of 13	H.V.	Jun. 70
SUBJECT		CHECKED BY	DATE
Empire Floodgate, Concrete Wall		D.G.M.	June 70





End Reaction = 1,402,000#

$$I = \frac{15.875^3 \times \frac{3}{4} \times 3}{12} = 750, \quad A = 67.0 \ \square''$$

$$r = \sqrt{11.2} = 3.34 \qquad \frac{\ell}{r} = 0.7 \times \frac{34}{3.34} = 7.1$$

$$f_a = \frac{1402000}{67} = 21,000 \text{ psi.}$$

PROJECT New Orleans to Venice, La	Page 8 of 13	COMPUTED BY H.V.	DATE Jun. 70
SUBJECT Empire Floodgate - Conc. Wall		CHECKED BY Da.M	DATE June 70

Beam "D"

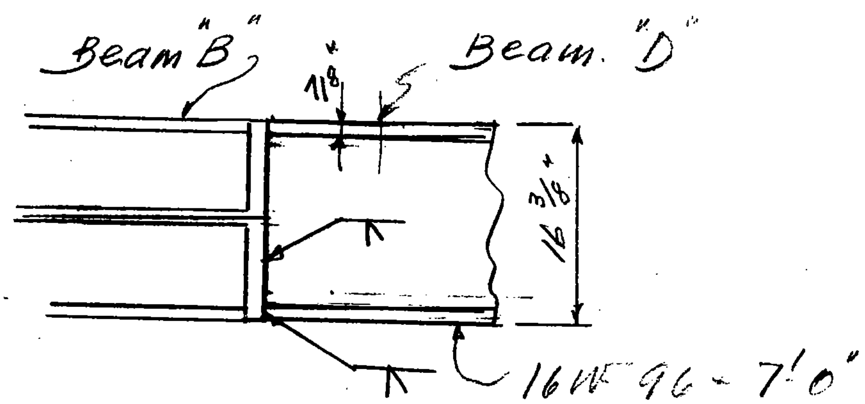
Tension = 658,400#.

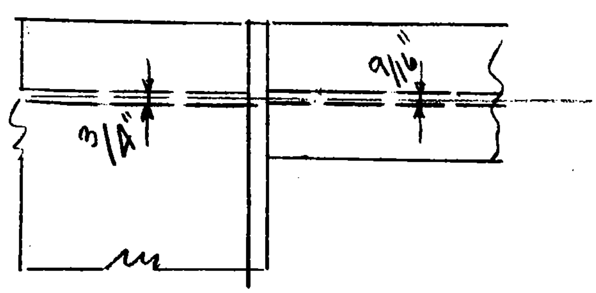
allow bond stress 150 psi.

$$\text{Beam Area Reqd.} = \frac{658400}{24,000} = 27.5^{\square''}$$

Use 16 WF 96 (A = 28.22)

$$\text{Length Reqd} = \frac{658400}{2 \times (16\frac{3}{8} + 11\frac{1}{2}) \times 150} = 78.5'' \sim 6'\text{-}6''$$





PROJECT New Orleans to Venice, La.	Page 9 of 13	COMPUTED BY H.V.	DATE June 70
SUBJECT Empire Floodgate - Concrete Wall		CHECKED BY D.Q.M.	DATE June 70

Beam "E".

Tension = 123,800#.

Reqd Area $\frac{123,800}{24,000} = 5.17 \square''$

Use 12WF 65. (A = 19.11).

Reqd Length = $\frac{123800}{2(12+12\frac{1}{8})+100} = 27''$

Use 5'-0"

PROJECT New Orleans to Venice, La	Page 10 of 13	COMPUTED BY H.V.	DATE Jun 70
SUBJECT Empire Floodgate Concrete Wall		CHECKED BY D.G.M.	DATE June 70

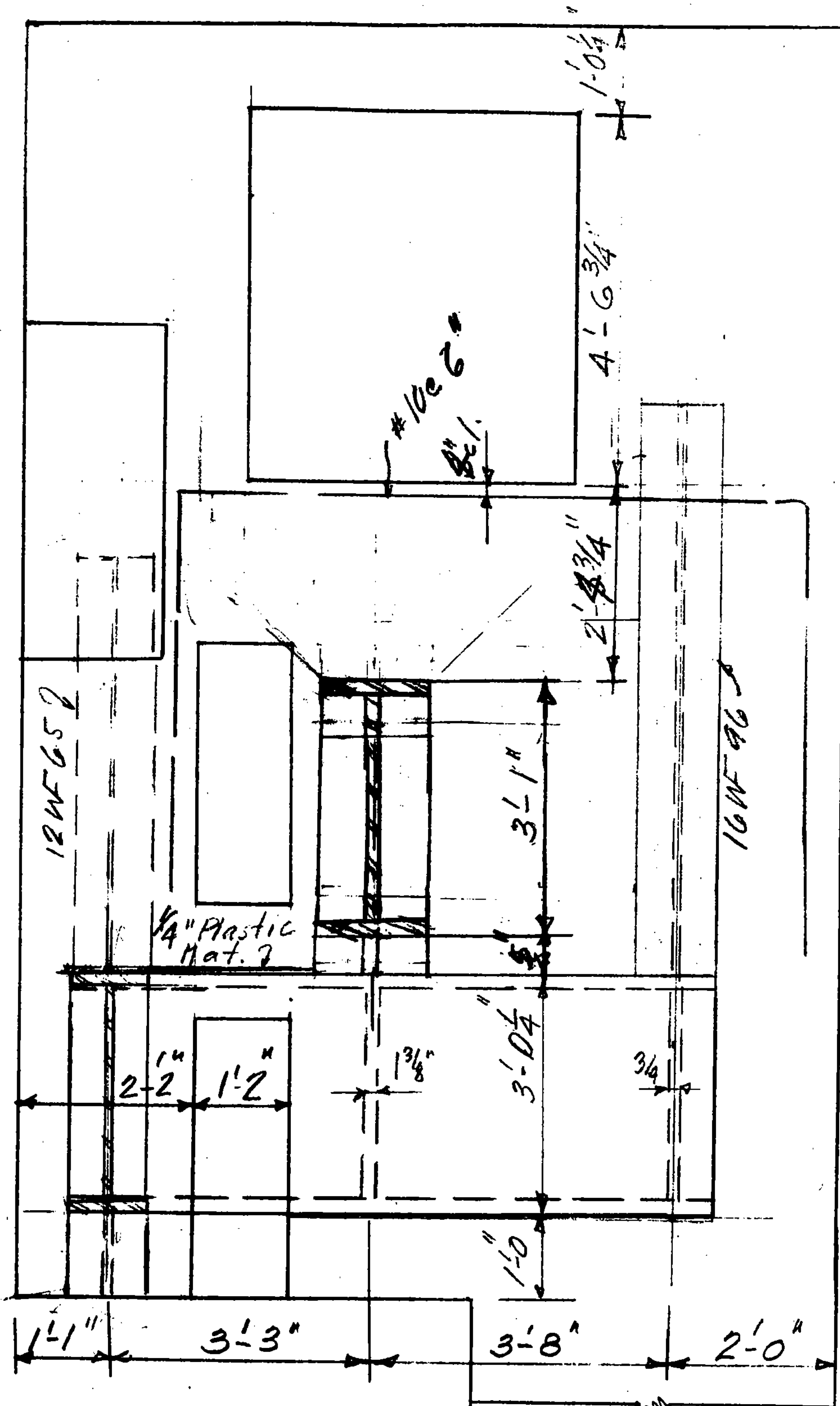


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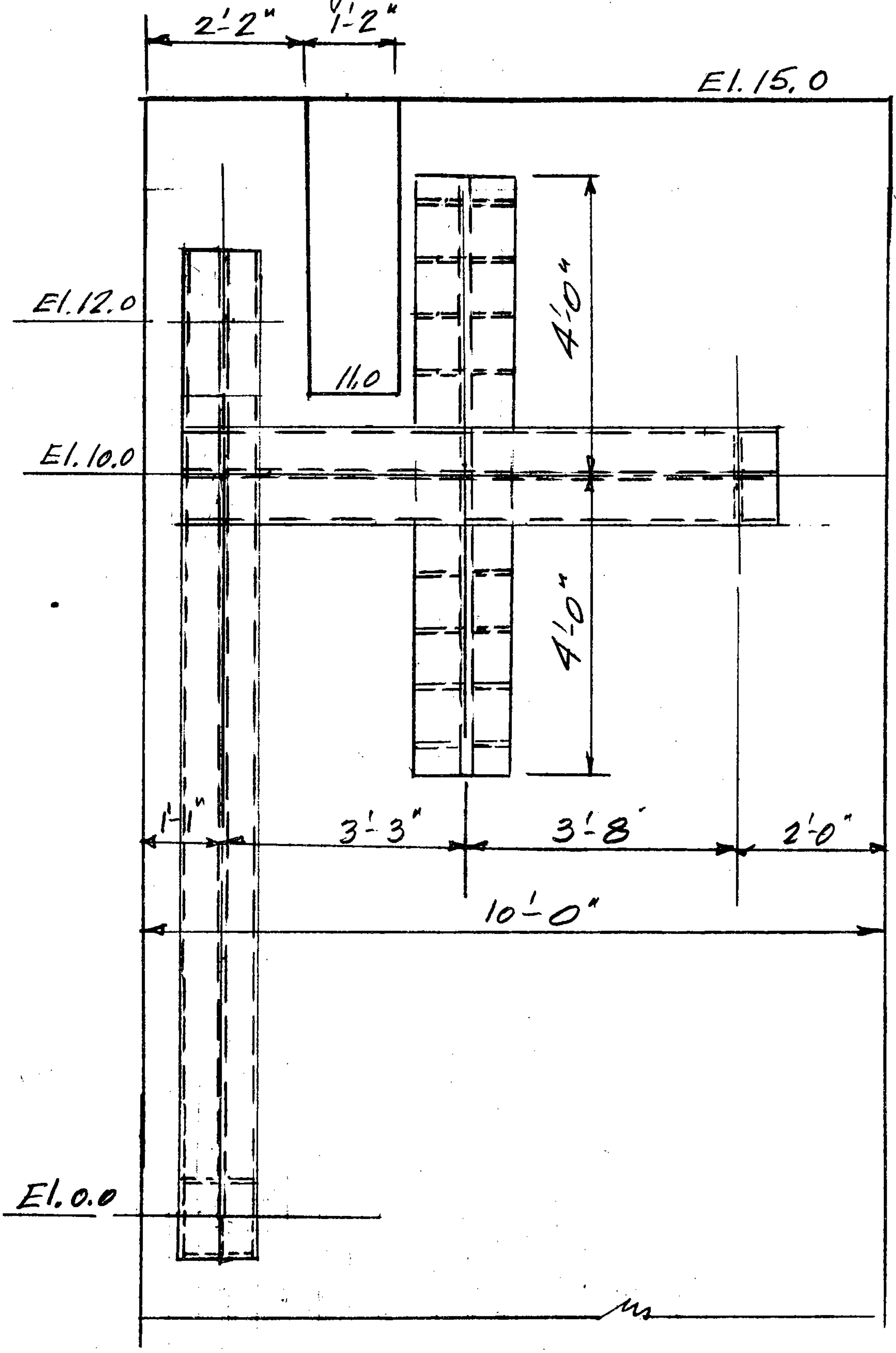
FIG IV-47

PROJECT New Orleans to Venice, La	Page 11 of 13	COMPUTED BY H.V.	DATE Jun. 70
SUBJECT Empire Floodgate - Conc. Wall		CHECKED BY W.a.M.	DATE June 70



PROJECT	Page	COMPUTED BY	DATE
New Orleans to Venice, La	12 of 13	H.U.	Jun. 70
SUBJECT		**CHECKED BY**	**DATE**
Empire Floodgate - Concrete Wall		D.G.M	June 70

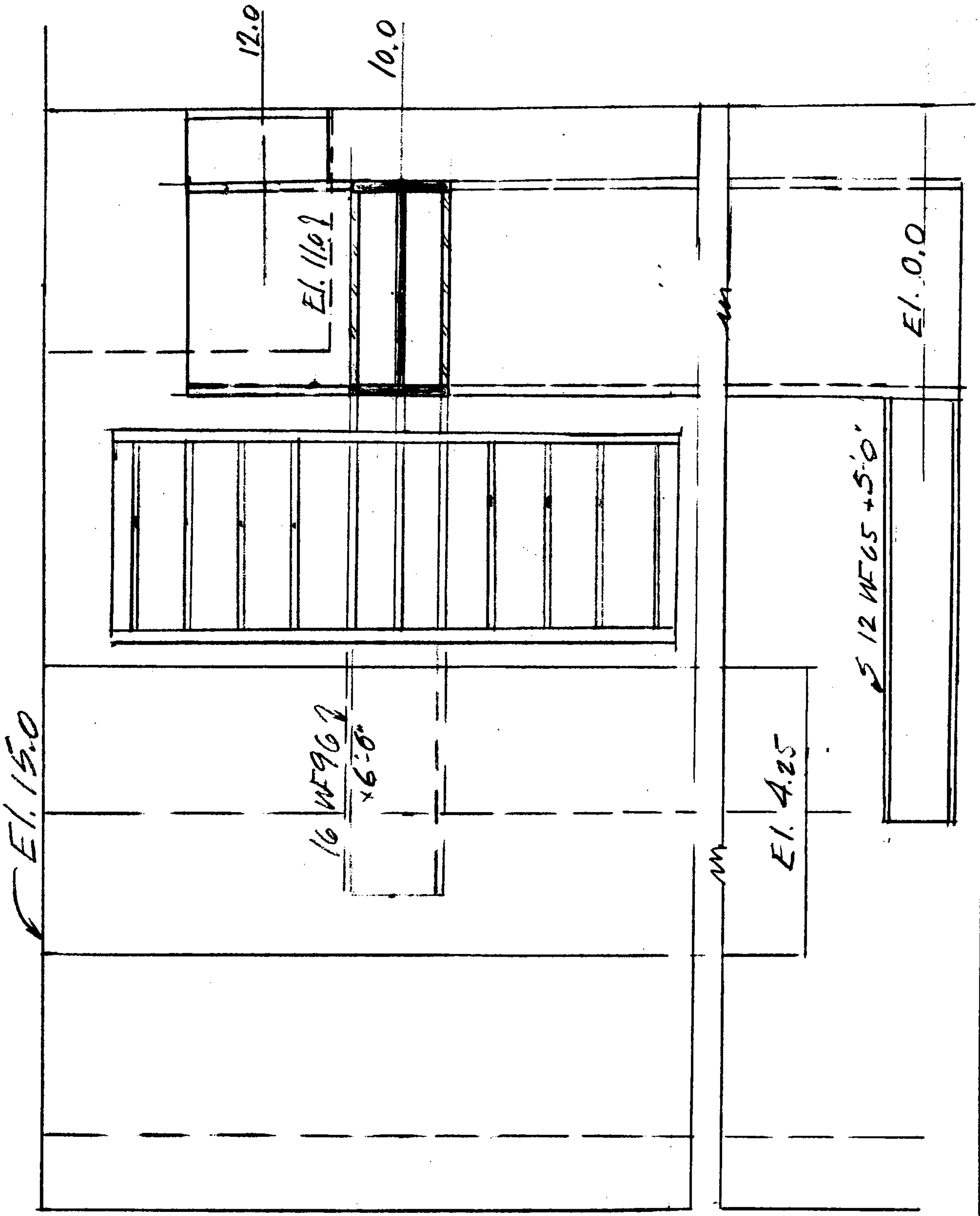
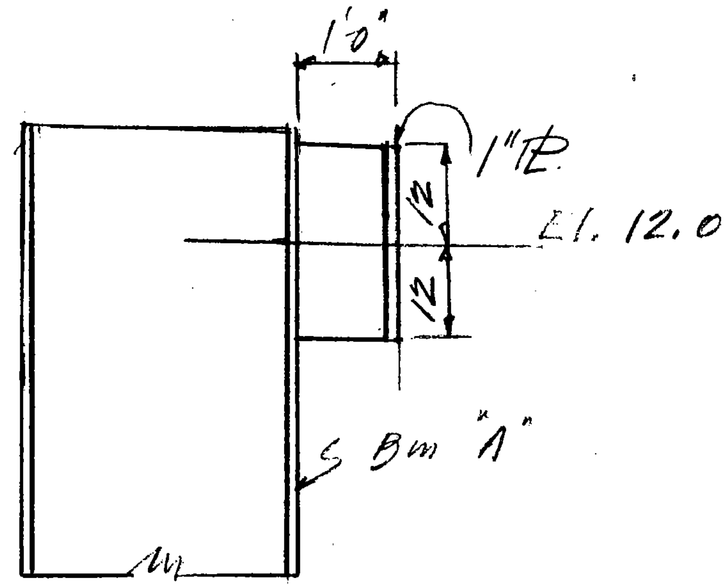


FIG IV-49

PROJECT New Orleans to Venice, La.	Page 13 of 13	COMPUTED BY H.U.	DATE Jun. 70
SUBJECT Empire Floodgate - Conc. Wall		CHECKED BY D.A.M.	DATE June 70



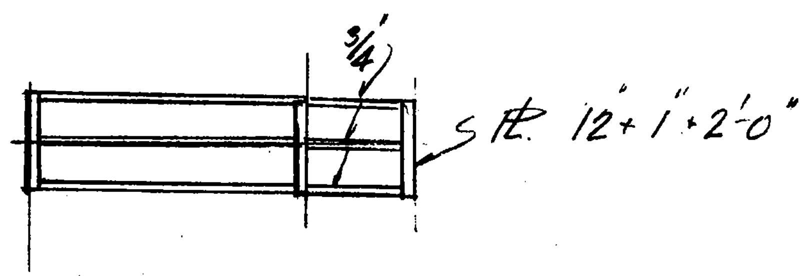


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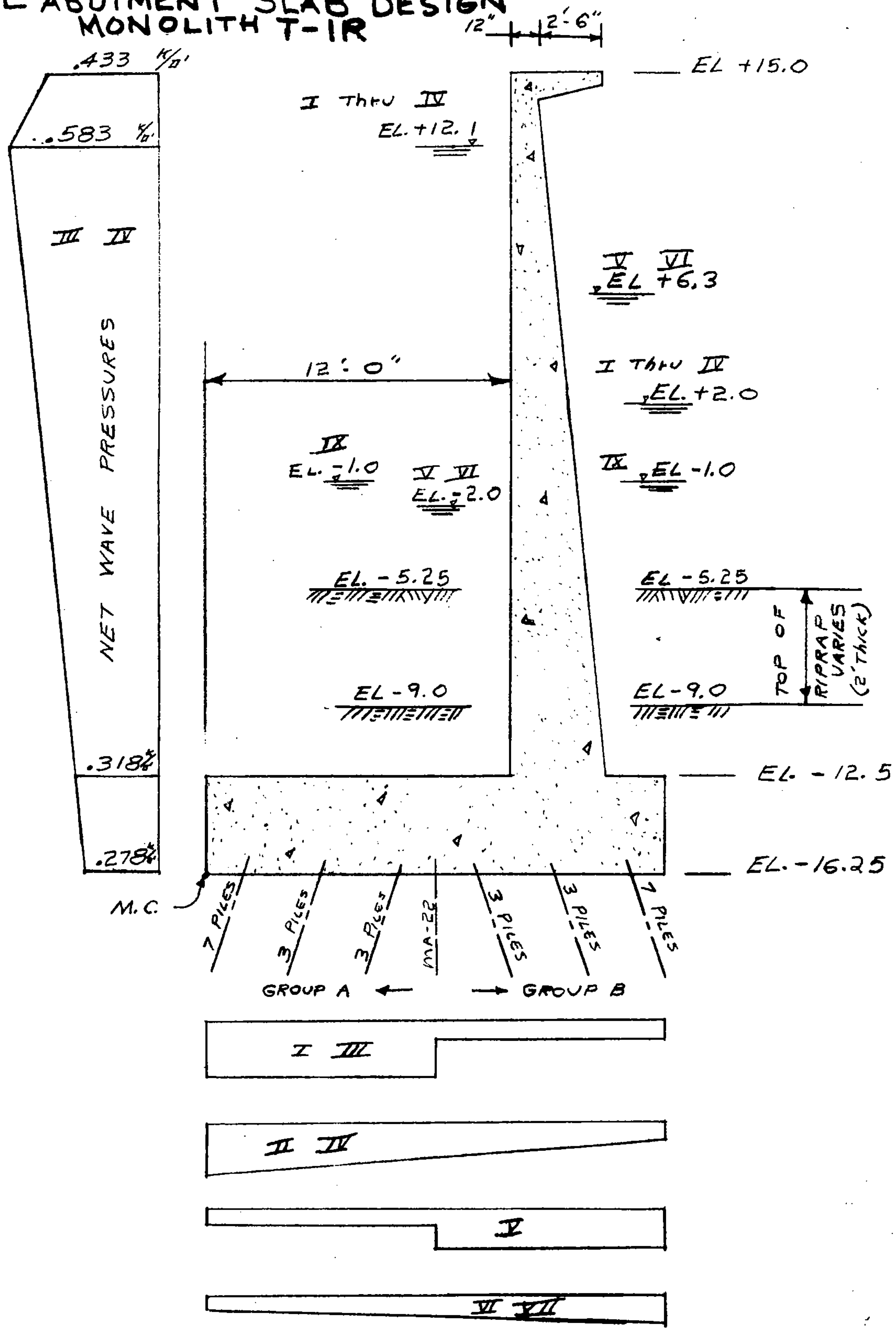
FIG III-50

PROJECT EMPIRE FLOODGATE	Page 1 of 14	COMPUTED BY CWR	DATE 10 June 70
SUBJECT CONCRETE DESIGN		CHECKED BY DM	DATE 7 July 70

TYPICAL ABUTMENT SLAB DESIGN
MONOLITH T-1R



UPLIFT PRESSURE DIAGRAMS

FIG. III-51

MOMENTS AND FORCES FOR ABUTMENT WALL - MONOLITH T-1R

CASE	MOMENT	VERTICAL	HORIZONTAL
I	9190.057	452.433	367.703
II	8977.010	452.433	367.703
III *	11539.245	363.934	531.109
IV*	11309.473	363.934	531.109
V	1518.279	352.175	-238.625
VI	1693.357	352.175	-238.625
IX	4494.027	416.574	0.0

PILE LOADS WERE COMPUTED BY THE HRENNIKOFF METHOD Ⓐ OF ANALYSIS OF PILE FOUNDATIONS WITH BATTER PILES, UTILIZING A GE 400 DATA PROCESSING SYSTEM. A SUMMARY OF CRITICAL PILE LOADS IS TABULATED BELOW.

* TO ALLOW FOR A $33\frac{1}{3}$% INCREASE IN ALLOWABLE STRESSES WHEN GROUP II LOADING WAS INVESTIGATED, ACTUAL GROUP II LOADS WERE REDUCED BY 25% AND THE SAME ALLOWABLE STRESSES WERE USED IN ALL CASES.

COMPUTED PILE LOADS Ⓑ

ROW AND GROUP	CASE GOVERNING DESIGN	TENSION			COMPRESSION		
		AXIAL FORCE	TRANSVERSE		AXIAL FORCE	TRANSVERSE	
			FORCE	DEFL.		FORCE	DEFL.
1-A	III	48.51^k	-	.1431 IN	-	-	-
	II	-	1.410^k	.0534 IN	-	-	-
2-A	IV	45.76	-	.1302 IN	-	-	-
	VI	-	-	-	-	1.865^k	.0609 IN
3-A	VI	-	-	-	78.03^k	-	.0609 IN
	VI	-	-	-	-	1.788^k	.0609 IN
1-B	VI	49.50^k		.0609 IN	-	-	-
	IV	-	1.325^k	.1302 IN	-	-	-
2-B	IV	-	-	-	75.25^k	-	.1302 IN
	VI	-	-	-	-	1.937^k	.0609 IN
3-B	III	-	-	-	78.00^k	-	.1431 IN
	IV	-	1.325^k	2.860 IN	-	-	-

Ⓐ PAPER NO. 2401 OF ASCE TRANSACTIONS - "ANALYSIS OF PILE FOUNDATIONS WITH BATTER PILES" BY A. HRENNIKOFF.

Ⓑ A PILE LOADING OF 51^k IN TENSION AND 80^k IN COMPRESSION IS ALLOWABLE.

Fig IV-52

PROJECT Empire Floodgate	Page 3 of 14	COMPUTED BY DM	DATE 7 July 70
SUBJECT Concrete Desigin Monolith T-1R		CHECKED BY [illegible]	DATE 7-July-70

Design of Base Slab:

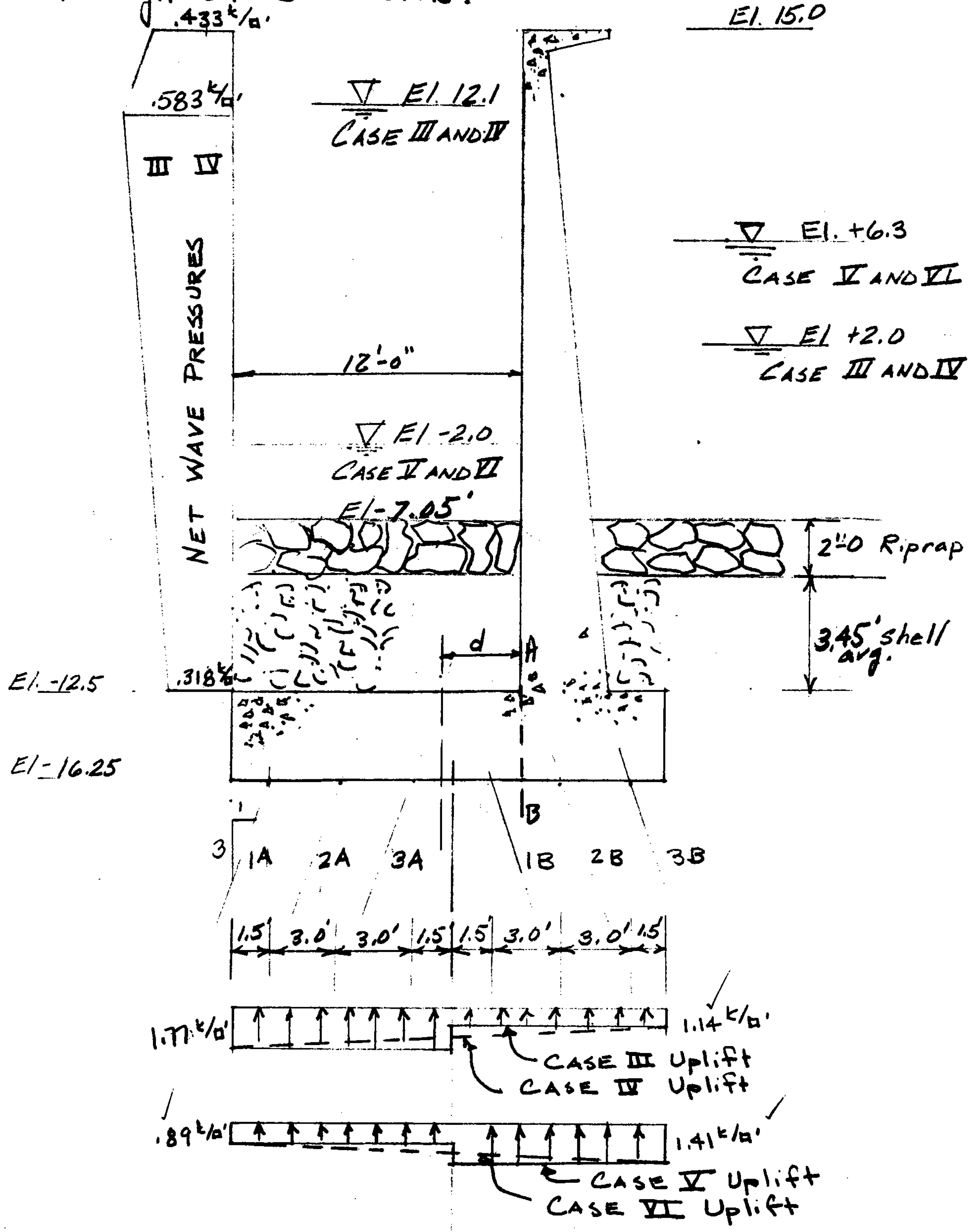
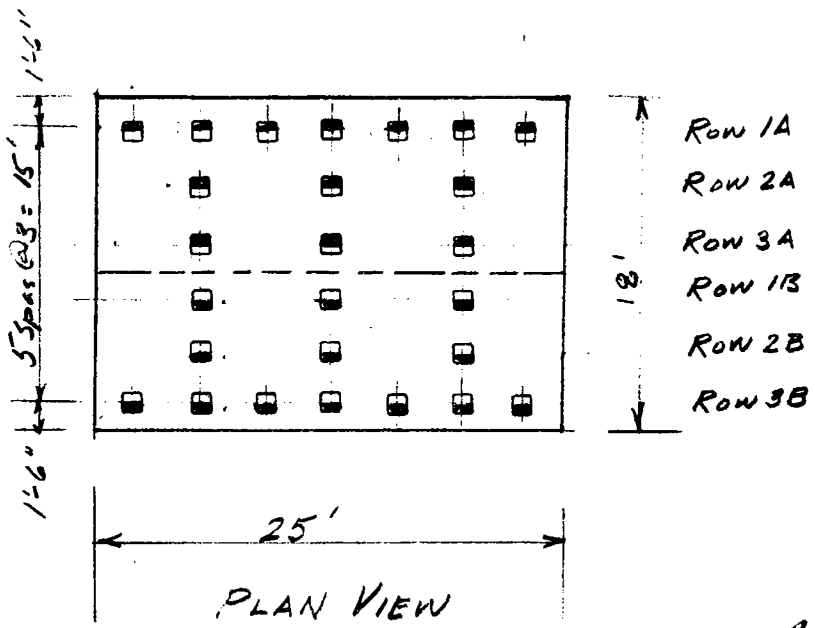


Fig. IV-53

PROJECT		COMPUTED BY	DATE
EMPIRE FLOODGATE	Page 4 of 14	DM	
SUBJECT: CONCRETE DESIGN MONOLITH T-1R		CHECKED BY: [illegible]	DATE: 7-July-70



PLAN VIEW

Comp. = +
Tens = −

Pile Row	CASE III		CASE IV		CASE V		CASE VI	
	Axial	Transverse	Axial	Transverse	Axial	Transverse	Axial	Transv.
1A	-48.5	-1.25	-45.8	-1.40	+18.1	+1.83	+16.1	+1.94
2A	-45.2	-1.26	-45.8	-1.40	+46.6	+1.76	+47.0	+1.87
3A	-41.8	-1.27	-45.8	-1.40	+75.0	+1.69	+78.0	+1.79
1B	+71.30	-1.19	+75.2	-1.33	-46.5	+1.76	-49.5	+1.86
2B	+74.6	-1.19	+75.2	-1.33	-18.1	+1.83	-18.5	+1.94
3B	+78.0	-1.18	+75.3	-1.33	+10.4	+1.90	+12.5	+2.01

Note: Cases III and IV pile loads have been reduced 25% because these cases are Group 2 Loadings.

Fig. IV-54

DESIGN OF BASE SLAB:

CASE III VERTICAL COMPONENTS OF PILE LOADS

	ROW	AXIAL	TRANSVERSE	COMPUTATION	FY (KIPS)
7 PILES	1A	-48.5	-1.25	$-48.5(3/\sqrt{10}) = -46.0$ $-1.25(1/\sqrt{10}) = +0.4$	-45.6
3 PILES	2A	-45.2	-1.26	$-45.2(3/\sqrt{10}) = -42.9$ $-1.26(1/\sqrt{10}) = +0.4$	-42.5
3 PILES	3A	-41.8	-1.27	$-41.8(3/\sqrt{10}) = -39.7$ $-1.27(1/\sqrt{10}) = +0.4$	-39.3
3 PILES	1B	+71.30	-1.19	$+71.30(3/\sqrt{10}) = +67.7$ $-1.19(1/\sqrt{10}) = -0.4$	+67.3

CASE IV

	ROW	AXIAL	TRANSVERSE	COMPUTATION	FY (KIPS)
7 PILES	1A	-45.8	-1.40	$-45.8(3/\sqrt{10}) = -43.5$ $-1.40(1/\sqrt{10}) = +0.4$	-43.1
3 PILES	2A	-45.8	-1.40	$-45.8(3/\sqrt{10}) = -43.5$ $-1.40(1/\sqrt{10}) = +0.4$	-43.1
3 PILES	3A	-45.8	-1.40	$-45.8(3/\sqrt{10}) = -43.5$ $-1.40(1/\sqrt{10}) = +0.4$	-43.1
3 PILES	1B	+75.2	-1.33	$+75.2(3/\sqrt{10}) = +71.4$ $-1.33(1/\sqrt{10}) = -0.4$	+71.0

NOTE: CASES III & IV ARE GROUP II LOADINGS, THEREFORE THE COMPUTER PROGRAM HAS REDUCED THE ABOVE PILE LOADS BY 25%. ACTUAL LOADS = 1.33 X THE ABOVE LOADS.

Fig IV-55

BASE SLAB DESIGN (CASE III) TOP BARS

MOMENT AT FACE OF WALL (SECTION A-B)

NOTE: PILE LOADS WILL BE INCREASED BY 1/3 TO CONVERT TO ACTUAL

ITEM	COMPUTATION	FORCE	$\bar{X}$	MOM.
CONCRETE	3.75(0.15)(12.0)	6.75	6.0	40.50
VERT. WATER	24.6(0.0625)(12.0)	18.45	6.0	110.70
VERT. WAVE	0.318(12.0)	3.82	6.0	22.92
RIPRAP	2.0(0.063)(12.0)	1.51	6.0	9.06
SHELL	3.45(0.030)(12.0)	1.24	6.0	7.44
	SUB TOTAL	31.77		190.62
UPLIFT	-1.77(9.0)	-15.93	7.5	-119.48
	-1.14(3.0)	-3.42	1.5	-5.13
PILE LOADS	x 1/3 (Converted to kips/ft)			
ROW 1A	45.6(1.33)(7)/25	16.98	10.5	178.29
ROW 2A	42.5(1.33)(3)/25	6.78	7.5	50.85
ROW 3A	39.3(1.33)(3)/25	6.27	4.5	28.22
ROW 1B	-67.3(1.33)(3)/25	-10.74	1.5	-16.11

TOTAL CASE III = $+31.71^{k}$ $\quad +307.26^{1\text{-}k}$

REDUCE MOMENT: $3/4(307.26) = 230.45^{1\text{-}k}$

$M_{AB} = 230.45^{1\text{-}k}$

Fig IV-56

Check Required "d"

$K = 152$
$a = 1.44$
$d = 40''$

$$F = \frac{M}{k} = \frac{230.45}{152} = 1.51$$

$d_{req.} = 38.87$ say $39''$ O.K.

$$A_s = \frac{M}{ad} = \frac{230.45}{1.44(40)} = 4.00^{\square''/1}$$

Use #11 @ $4\frac{1}{2}''$ Gives $4.16^{\square''/1}$, 3" Clear

Check Shear: (At "d" distance from face of wall)

Item	Computation	Shear
Concrete	3.75(0.15)(8.67)	4.88
Vert. Water	24.6(0.0625)(8.67)	13.33
Vert. Wave	0.318(8.67)	2.78
Riprap	2.0(0.063)(8.67)	1.09
Shell	3.45(0.030)(8.67)	0.90
	Subtotal	22.98
Uplift	-1.77(8.67)	-15.35
Pile Loads	Same as Sh. #6	
Row 1A		16.98
Row 2A		6.78
Row 3A		6.27

Total Shear Case III = 37.66^k

Fig IV-57

$V = 37.66(0.75) = 28.25^k$

$v = \frac{V}{bd} = \frac{28.25^k}{12(40)} = 59$ psi < 60 O.K.

CHECK BOND STRESS:

$u = \frac{V}{\Sigma_o jd} = \frac{28250}{11.77(0.891)(40)} = $ 67 psi < 148 O.K.

CASE IV

MOMENT AT AB

ITEM	COMPUTATION	FORCE	$\bar{X}$	MOM.
SUB-TOTAL FROM SH. #6				190.62^{1-k}
UPLIFT	$-1.35(12)$	-16.20	6.0	-97.20
	$-0.42(12)(1/2)$	-2.52	8.0	-20.16
PILE LOADS				
ROW 1A	43.1(1.33)(7)/25	16.04	10.5	168.42
ROW 2A	43.1(1.33)(3)/25	6.87	7.5	51.53
ROW 3A	43.1(1.33)(3)/25	6.87	4.5	30.92
ROW 1B	$-71.0(1.33)(3)/25$	-11.33	1.5	-17.00

TOTAL MOMENT, CASE IV $+307.13^{1-k}$

REDUCE MOM., 0.75(307.13) = 230.35

230.35 < 230.45 O.K.

Fig. IV-58

PROJECT EMPIRE FLOOD GATE	Page 9 of 14	COMPUTED BY	DATE 7-JULY-70
SUBJECT CONCRETE DESIGN - MONOLITH T-1R		CHECKED BY	DATE

CASE IV

CHECK SHEAR AT "d" DISTANCE FROM FACE OF WALL:

ITEM	COMPUTATION	SHEAR
	SUB-TOTAL FROM SH. # 7	22.98^k
UPLIFT	$-1.56(8.67)$ $-0.21(8.67)(\frac{1}{2})$	-13.53 -0.91
Piles	FROM SH. # 8	
ROW 1A		16.04
ROW 2A		6.87
ROW 3A		6.87

TOTAL SHEAR, CASE IV 38.32^k

$V = 38.32(0.75) = 28.74^k$

$v = \dfrac{28.74}{12(40)} = 59.87$ Say 60 psi O.K.

Bond O.K. by Inspection

Fig. IV-59

PROJECT EMPIRE FLOOD GATE	Page 10 of 14	COMPUTED BY [illegible]	DATE 7-JULY-70
SUBJECT CONCRETE DESIGN - MONOLITH T-1R		CHECKED BY	DATE

BASE SLAB DESIGN:

VERTICAL COMPONENTS OF PILE LOADS

CASE V

	ROW	AXIAL	TRANSVERSE	COMPUTATION	Fy (Kips)
7 PILES	1A	+18.1	+1.83	$+18.1(3/\sqrt{10}) = +17.18$ $+1.83(1/\sqrt{10}) = -0.58$	+16.60
3 PILES	2A	+46.6	+1.76	$+46.6(3/\sqrt{10}) = +44.24$ $+1.76(1/\sqrt{10}) = -0.56$	+43.68
3 PILES	3A	+75.0	+1.69	$+75.0(3/\sqrt{10}) = +71.20$ $+1.69(1/\sqrt{10}) = -0.53$	+70.67
3 PILES	1B	-46.5	+1.76	$-46.5(3/\sqrt{10}) = -44.15$ $+1.76(1/\sqrt{10}) = +0.56$	-44.71

CASE VI

ROW	AXIAL	TRANSVERSE	COMPUTATION	Fy (kips)
1A	+16.1	+1.94	$+16.1(3/\sqrt{10}) = +15.28$ $+1.94(1/\sqrt{10}) = -0.61$	+14.67
2A	+47.0	+1.87	$+47.0(3/\sqrt{10}) = +44.62$ $+1.87(1/\sqrt{10}) = -0.59$	+44.03
3A	+78.0	+1.79	$+78.0(3/\sqrt{10}) = +74.05$ $+1.79(1/\sqrt{10}) = -0.56$	+73.49
1B	-49.5	+1.86	$-49.5(3/\sqrt{10}) = -47.00$ $+1.86(1/\sqrt{10}) = +0.58$	-46.42

Fig. IV-60

BASE SLAB DESIGN (CASE V) BOTTOM BARS

MOMENT AT FACE OF WALL

ITEM	COMPUTATION	FORCE	$\bar{X}$	MOM.
CONCRETE	3.75(0.15)(12.0)	6.75	6.0	40.50
VERT. WATER	14.5(0.0625)(12.0)	10.86	6.0	65.16
RIPRAP	2.0(0.063)(12.0)	1.51	6.0	9.06
SHELL	3.45(0.030)(12.0)	1.24	6.0	7.44
	SUBTOTAL	20.36		122.16
UPLIFT	-0.89(9.0) -1.41(3.0)	-8.01 -4.23	7.5 1.5	-60.08 -6.35
PILE LOADS				
ROW 1A	-16.60(7)/25	-4.64	10.5	-48.72
ROW 2A	-43.68(3)/25	-5.24	7.5	-39.30
ROW 3A	-70.67(3)/25	-8.48	4.5	-38.16
ROW 1B	+44.71(3)/25	+5.36	1.5	+8.04

TOTAL MOMENT CASE V = $-62.41^{1\text{-}k}$

FOR BOTTOM BARS USE "d" = 39"
k = 152
a = 1.44

$$F = \frac{M}{k} = \frac{62.41}{152} = 0.41$$

$d_{req.} = 20" < 39"$ O.K. $\quad A_s = \frac{M}{ad} = \frac{62.41}{1.44(39)} = 1.11^{\square''/1}$

Fig IV-61

CHECK SHEAR AT "d" DISTANCE FROM WALL:

CASE V

ITEM	COMPUTATION	SHEAR
CONCRETE	3.75(0.15)(8.67)	4.88
VERT. WATER	14.5(0.0625)(8.67)	7.84
RIPRAP	2.0(0.063)(8.67)	1.09
SHELL	3.45(0.030)(8.67)	0.89
	SUBTOTAL	14.70
UPLIFT	- 0.89(8.67)	- 7.72
PILE LOADS	SAME AS SH. # 11	
ROW 1A	↓	-4.64
ROW 2A	↓	-5.24
ROW 3A	↓	-8.48

TOTAL SHEAR CASE V = -11.38^{k}

$V = 11{,}380^{\#}$ $\quad v = \frac{V}{bd} = \frac{11{,}380}{12(39)} = 24.31$ psi O.K.

PROJECT EMPIRE FLOOD GATE	Page 13 of 14	COMPUTED BY MH	DATE 7-JULY-70
SUBJECT CONCRETE DESIGN-MONOLITH T-1R		CHECKED BY R	DATE 7-13

BASE SLAB DESIGN (CASE VI) BOTTOM BARS
MOMENT AT FACE OF WALL

ITEM	COMPUTATION	FORCE	$\bar{X}$	MOM.
	SUBTOTAL FROM SH. #11			122.16'-k
UPLIFT	$-0.89(12)$ $-0.34(12)(\frac{1}{2})$	-10.86 -2.04	6.0 8.0	-65.16 -16.32
PILE LOADS				
ROW 1A	$-14.67(7)/(25)$	-4.10	10.5	-43.05
ROW 2A	$-44.03(3)/25$	-5.28	7.5	-39.60
ROW 3A	$-73.49(3)/25$	-8.81	4.5	-39.65
ROW 1B	$+46.42(3)/25$	+5.57	1.5	+8.36

TOTAL MOMENT CASE VI = -73.26'-k

$$F = \frac{M}{K} = \frac{73.26}{152} = 0.48$$

$d_{req.} = 22'' < 39''$ O.K.

$$A_s = \frac{M}{ad} = \frac{73.26}{1.44(39)} = 1.30^{\square''/'}\ \text{(CRITICAL)}$$

NOTE: PLACE REINFORCEMENT BETWEEN PILES: SPACE BETWEEN PILES = 25'-7' = 18'

INCREASE A_s by $\frac{25}{18}(1.30) = 1.79^{\square''/'}$

USE #8 @ 5"

Fig IV-63

PROJECT EMPIRE FLOOD GATE	Page 14 of 14	COMPUTED BY [initials]	DATE 7-JULY-70
SUBJECT CONCRETE DESIGN - MONOLITH T-1R		CHECKED BY RES	DATE 7-70

CHECK SHEAR AT "d" DISTANCE FROM WALL:

CASE VI

ITEM	COMPUTATION	SHEAR
	SUB-TOTAL SH.# 12	14.70
UPLIFT	$-.89(8.67)$ $-.25(8.67)(\frac{1}{2})$	-7.72 -1.08
PILE LOADS	SAME AS SH.# 13	
ROW 1A	↓	-4.10
ROW 2A		-5.28
ROW 3A		-8.81

TOTAL SHEAR CASE VI -12.29^k

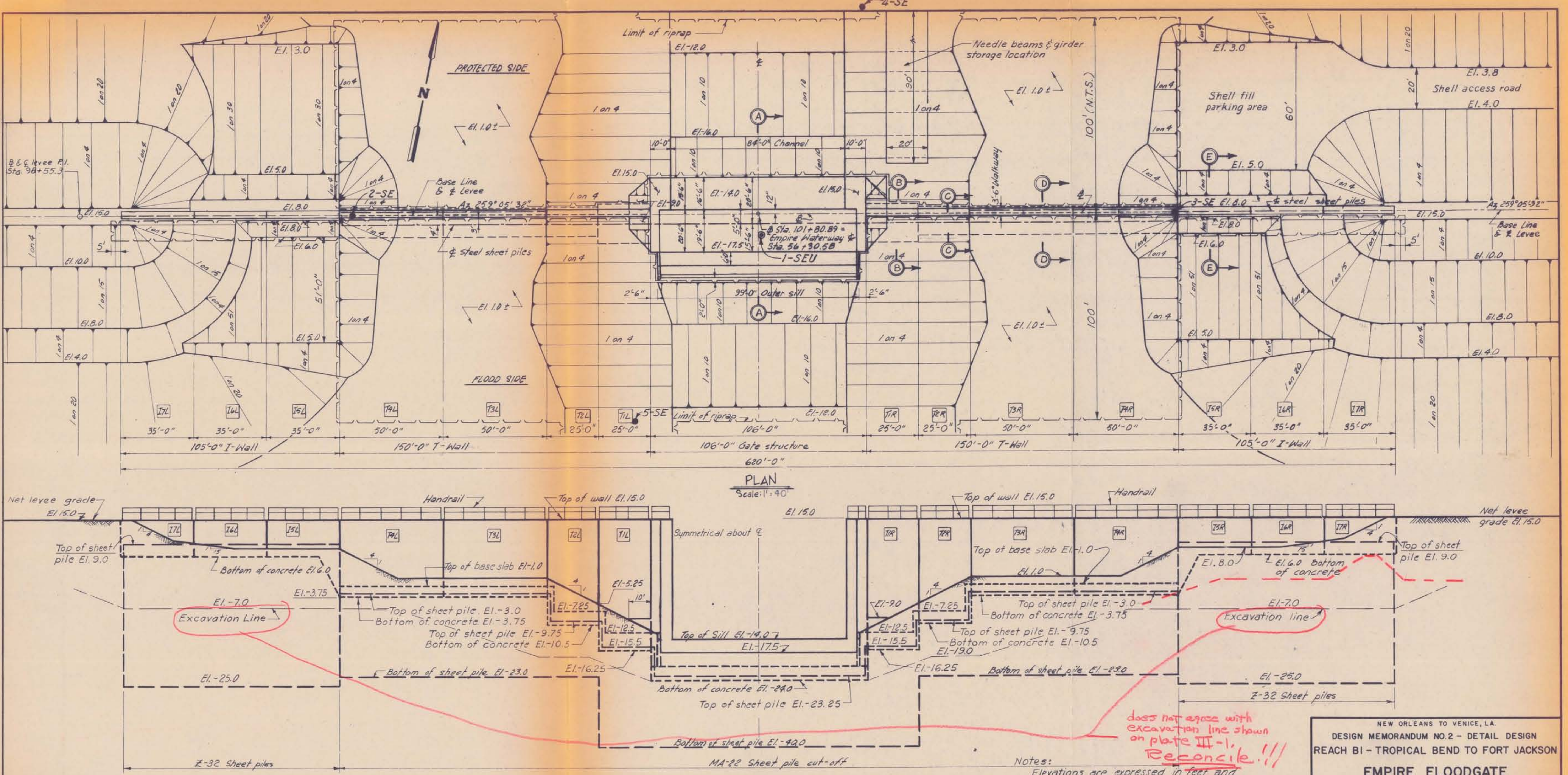
$V = 12{,}290 \qquad v = \frac{V}{bd} = \frac{12{,}290}{12(39)} = 26.26$ psi O.K.

BOND O.K.

SUMMARY OF FLEXURAL STEEL

TOP BARS - #11 @ $4\frac{1}{2}$"

BOTTOM BARS - #8 @ 5" (Place Between Piles)



does not agree with excavation line shown on plate III-1, Reconcile!!!

FLOODSIDE ELEVATION

Scale: 1" = 40' Horiz.
1" = 20' Vert.

SOIL BORING LEGEND

- ● - General type Soil Boring
- ◉ - 5" dia. Undisturbed Soil Boring

Notes:
Elevations are expressed in feet and refer to mean sea level.
For excavation and cofferdam, see plate III-1
For tie-in to existing levee, see plate I-3.
For sections, see plate IV-2
For Soil Boring logs see plates III-23 and III-24.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

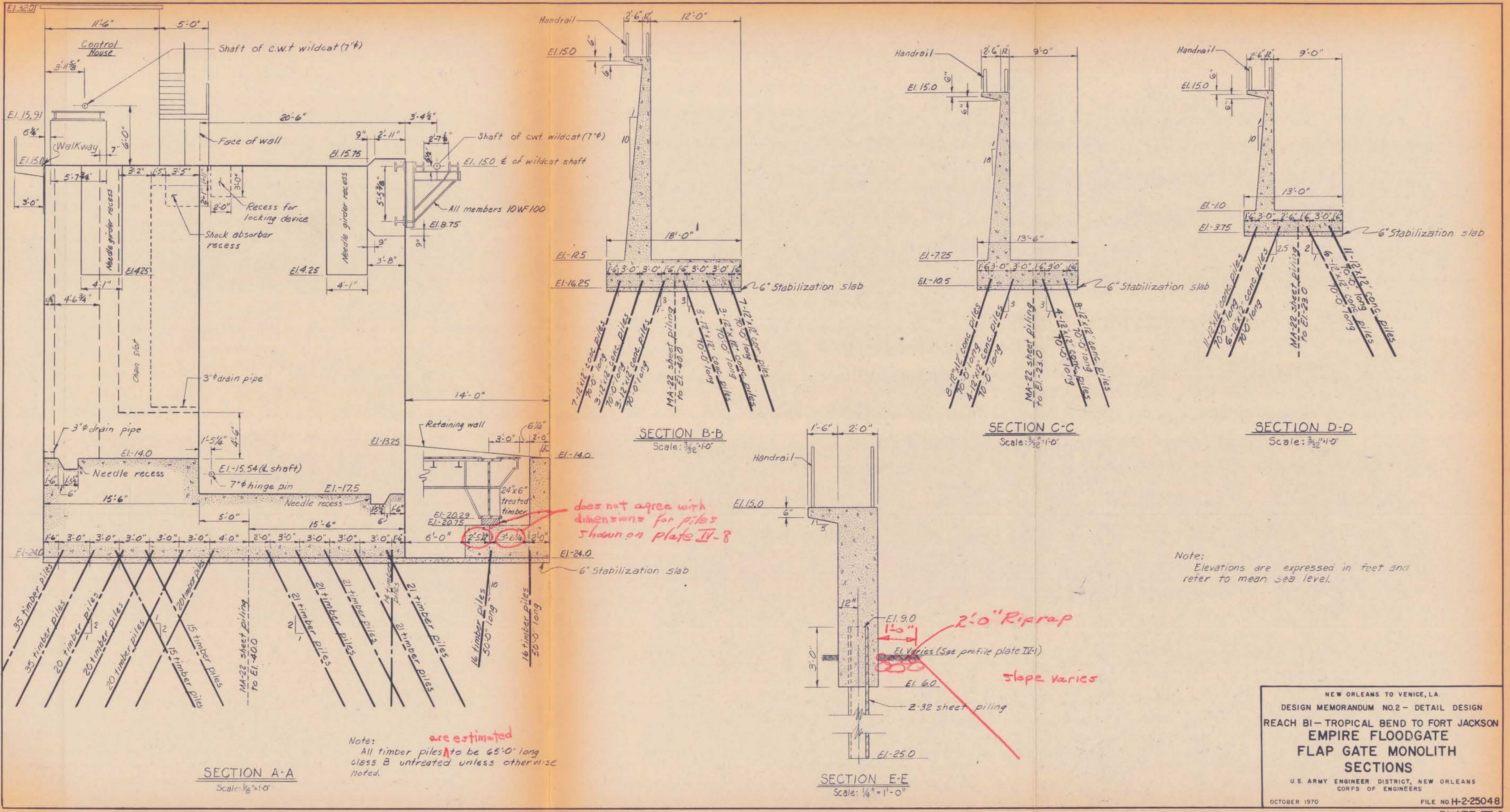
EMPIRE FLOODGATE
PLAN AND ELEVATION

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

OCTOBER 1970

FILE NO. H-2-25048

PLATE IV-1



NEW ORLEANS TO VENICE, LA.

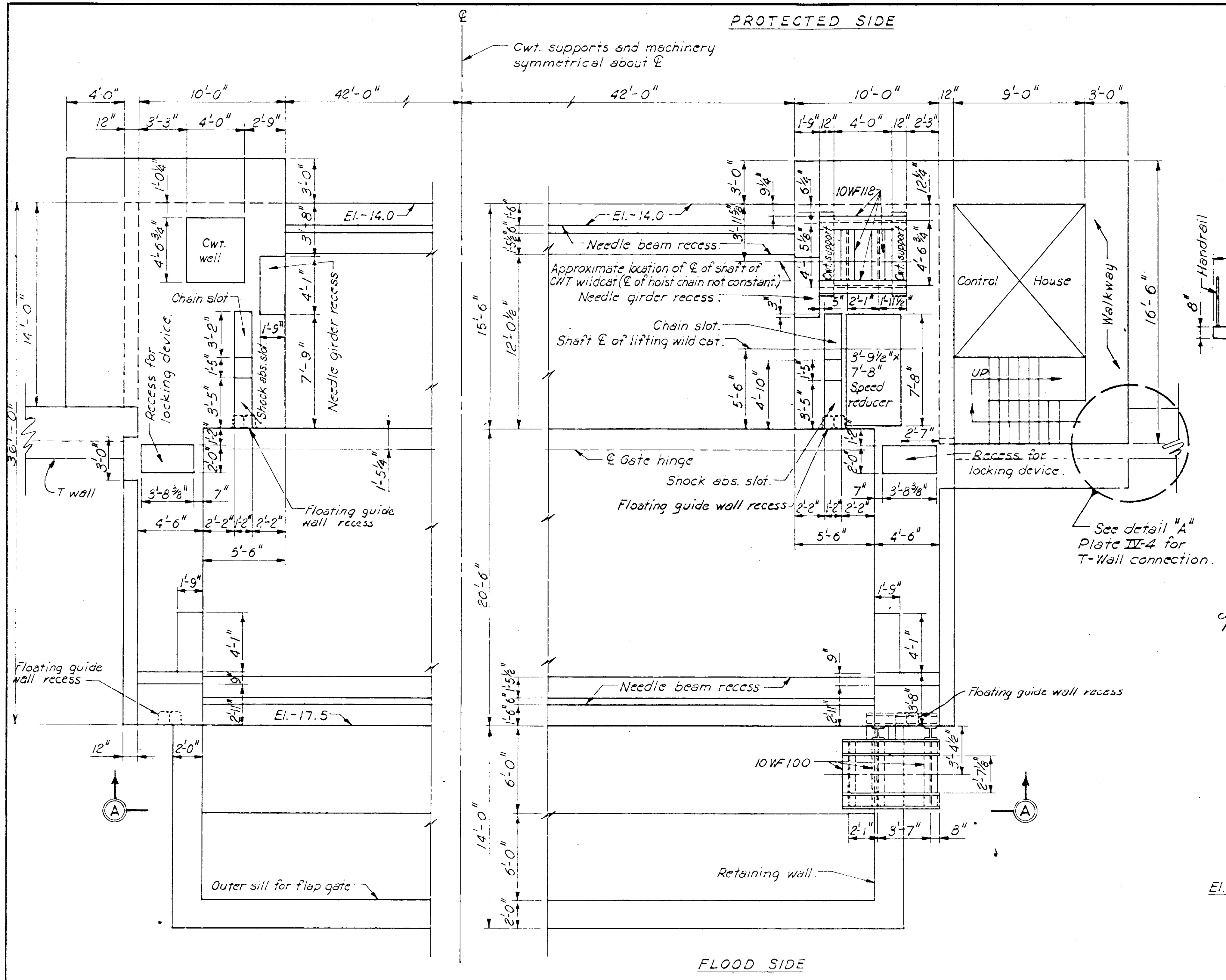
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN

REACH BI – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
FLAP GATE MONOLITH
SECTIONS

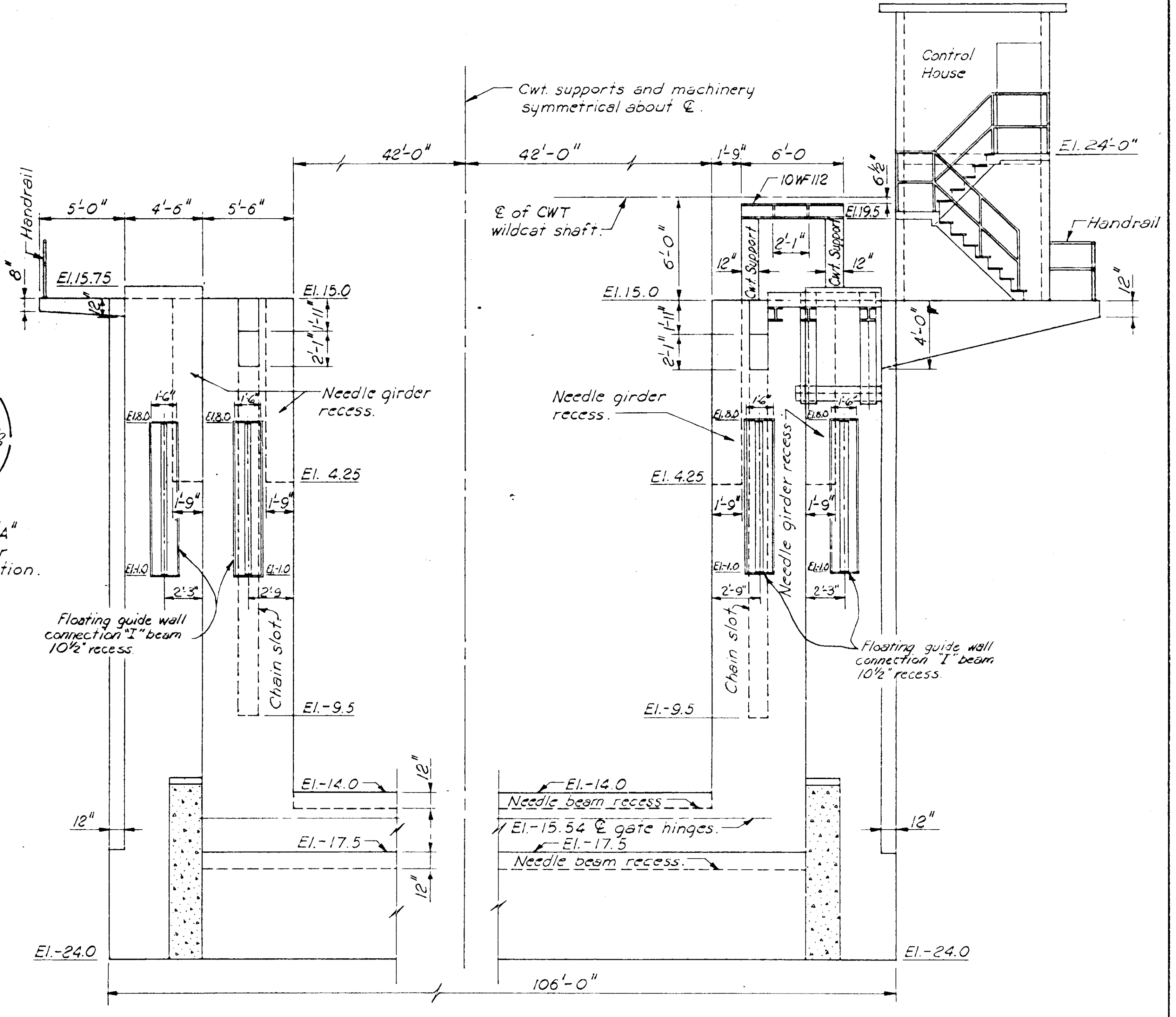
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 — FILE NO. H-2-25048



PLAN

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



SECTION A-A

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

Note:
Elevations are expressed in feet and refer to mean sea level.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
FLAP GATE MONOLITH
PLAN AND SECTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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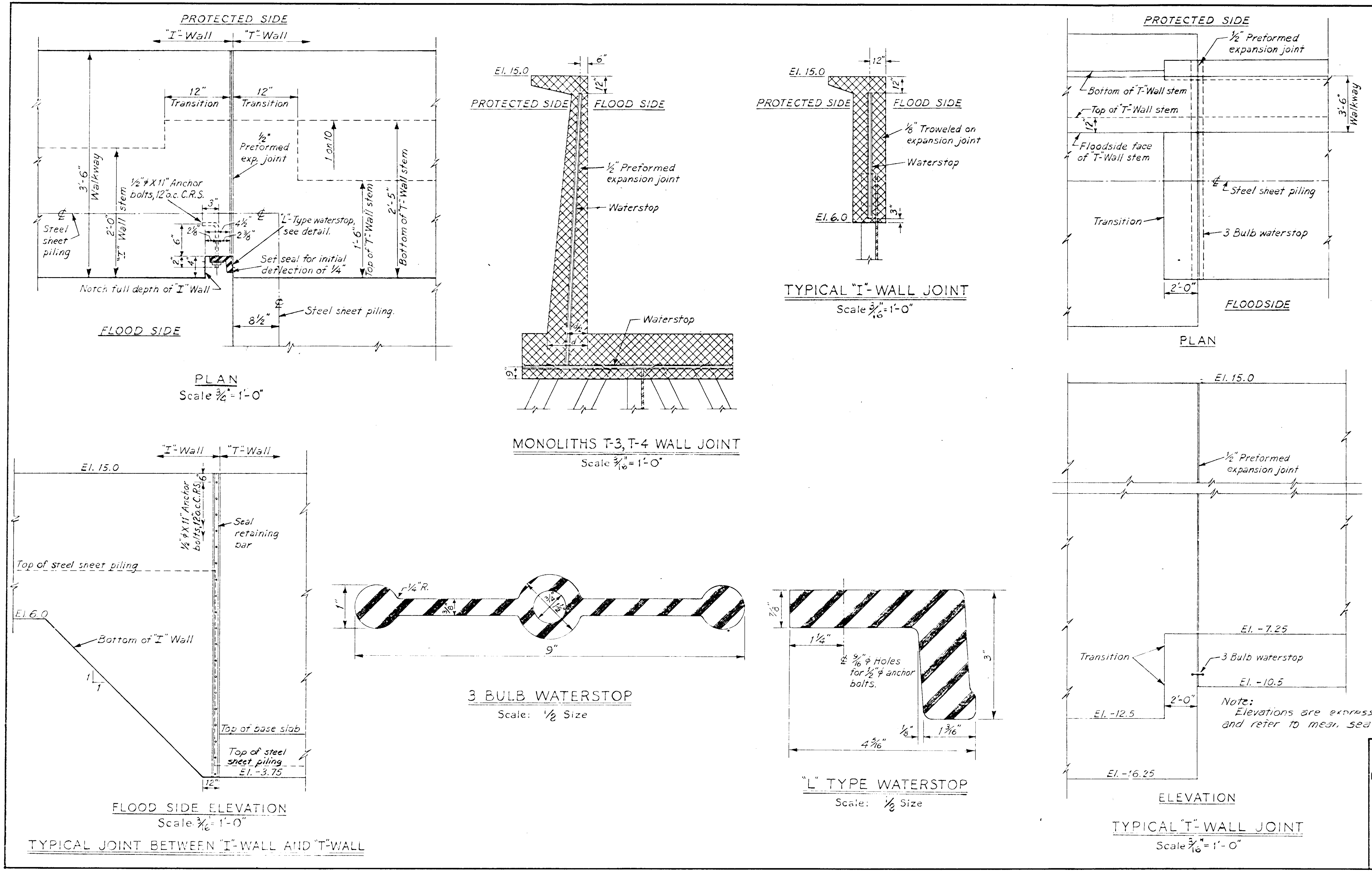
PLATE IV-3



NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

TYPICAL WALL JOINTS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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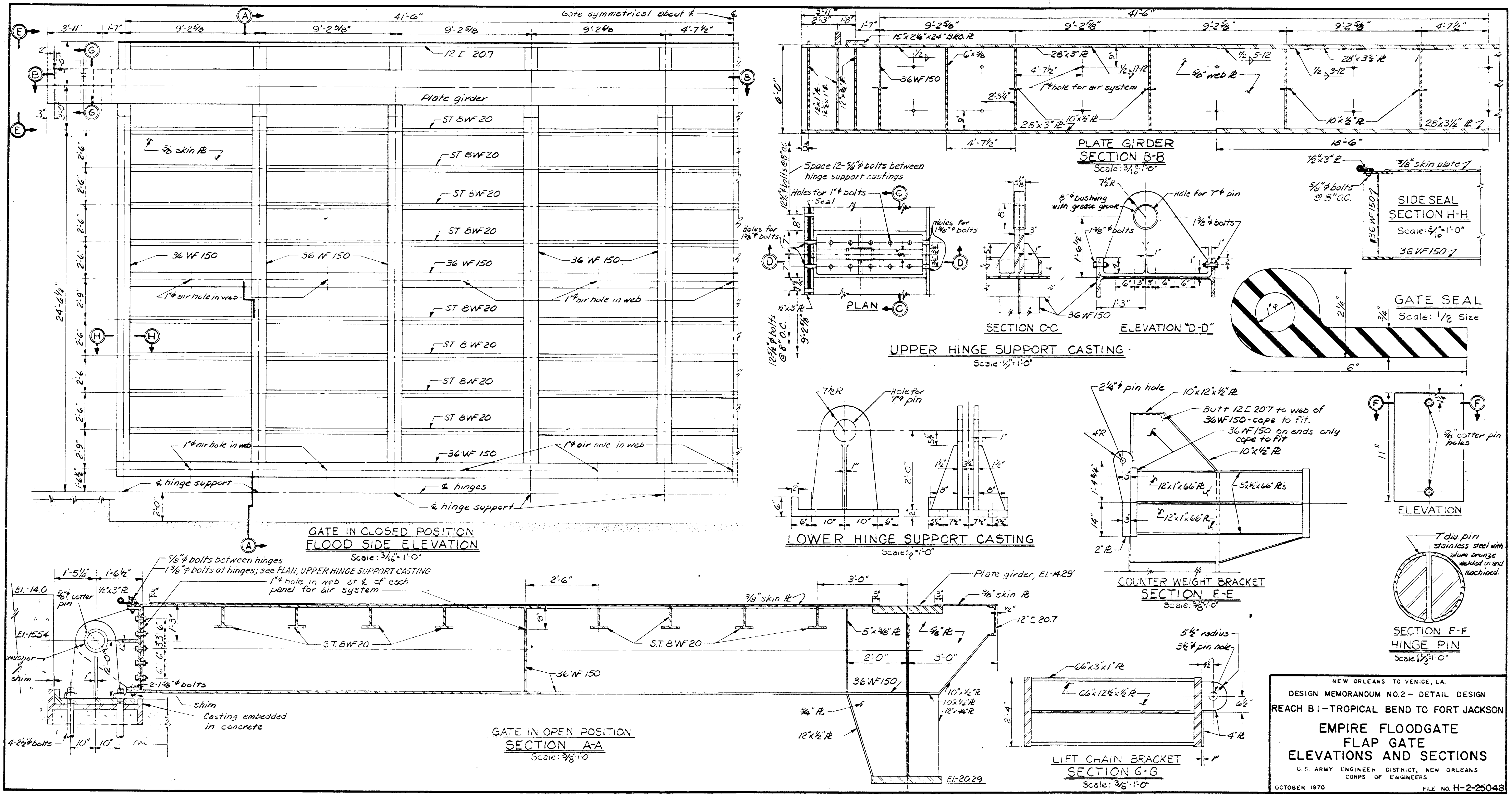
PLATE IV-4



NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
FLAP GATE
ELEVATIONS AND SECTIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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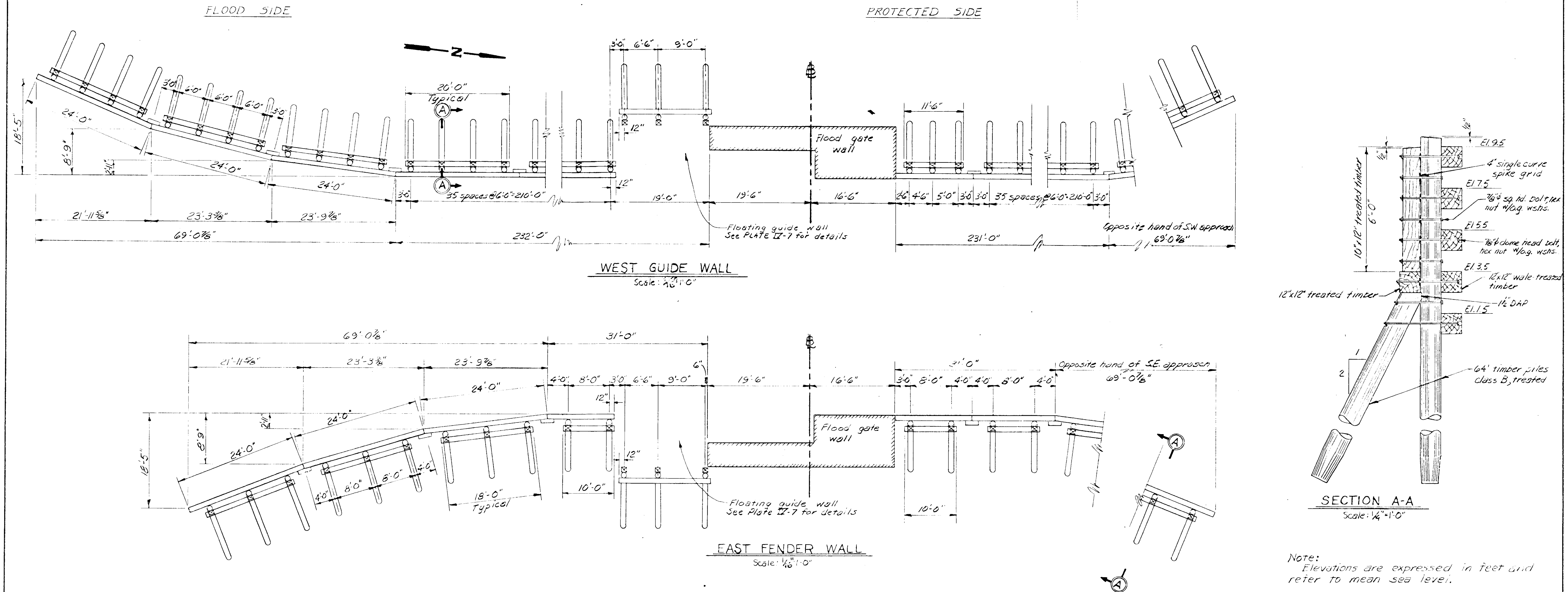
PLATE IV-5



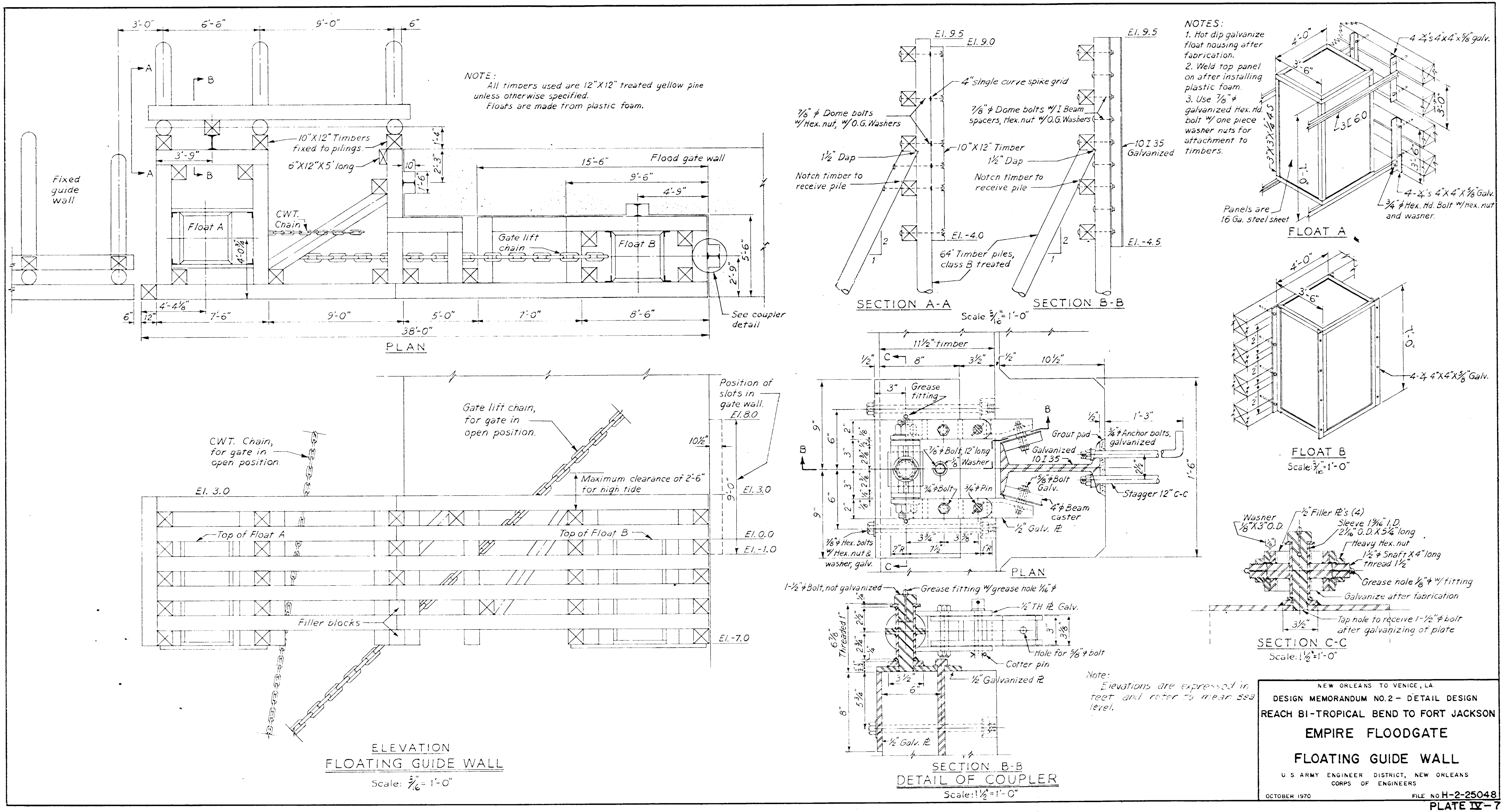
Note:
Elevations are expressed in feet and refer to mean sea level.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
GUIDE WALL AND FENDER WALL

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

OCTOBER 1970 FILE NO. H-2-25048



NEW ORLEANS TO VENICE, LA.

DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN

REACH B1-TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

FLOATING GUIDE WALL

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

OCTOBER 1970 — FILE NO. H-2-25048

PLATE IV-7

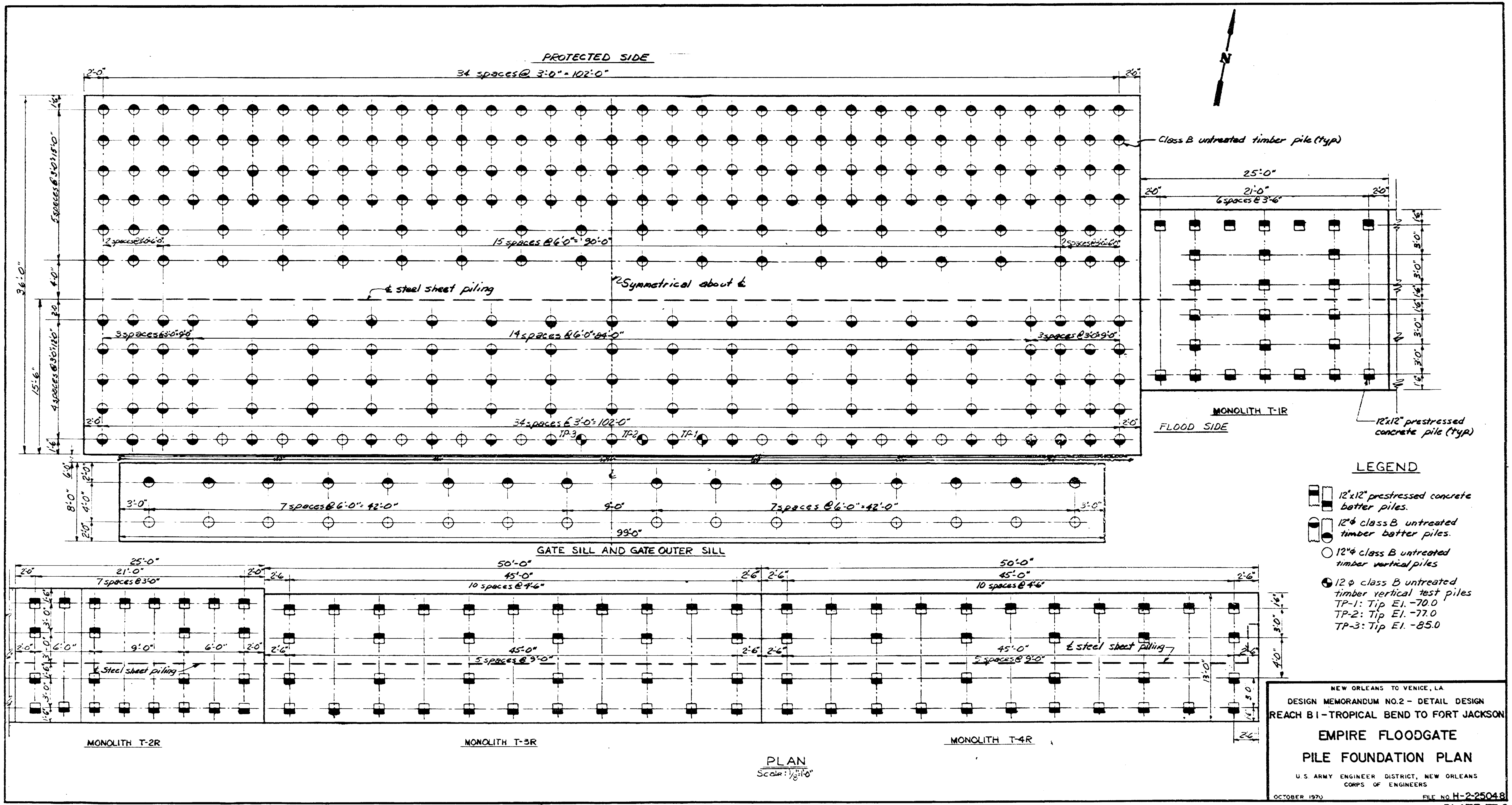
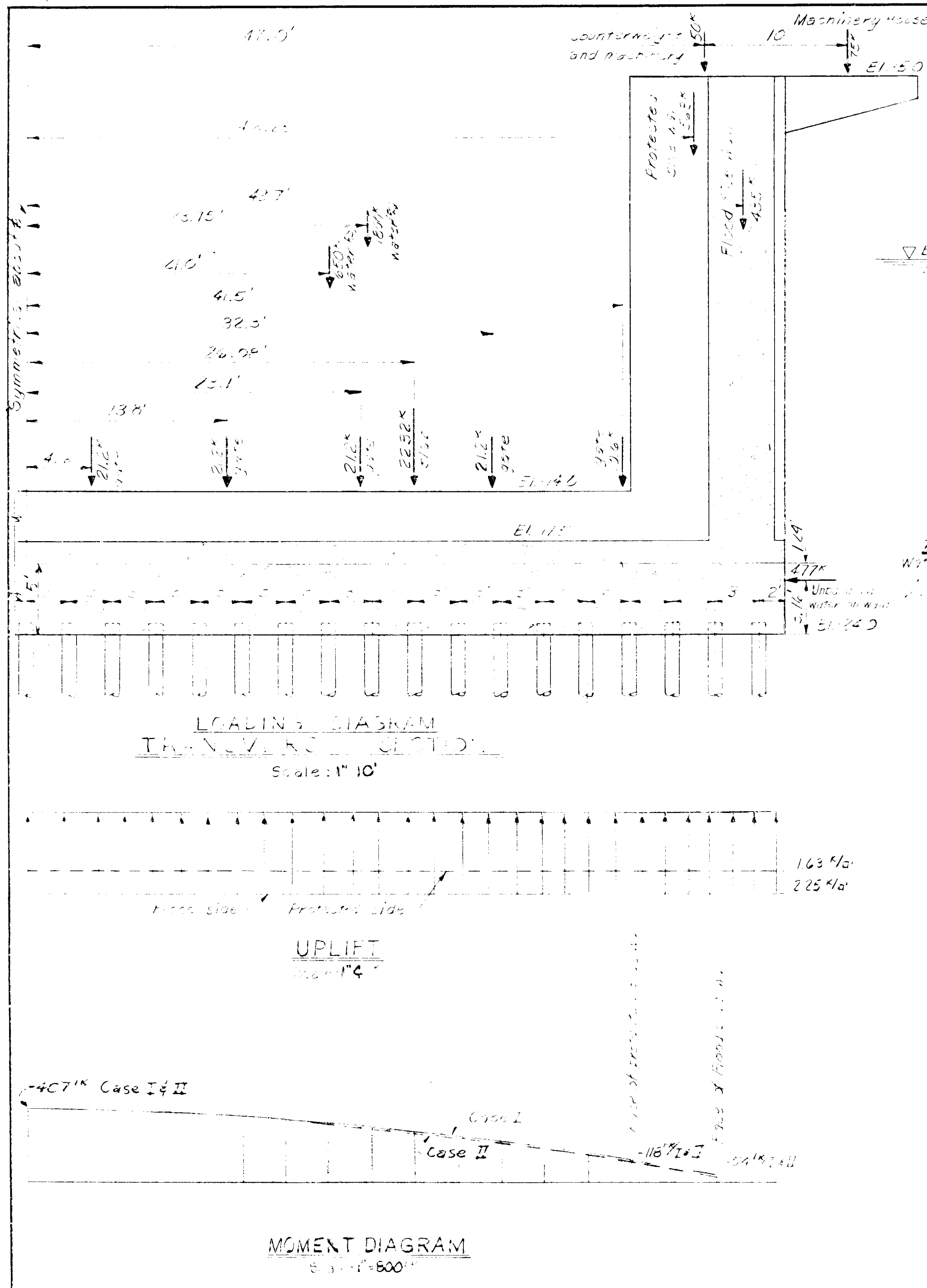

PROTECTED SIDE
34 spaces @ 3'-0" = 102'-0"
Class B untreated timber pile (typ)
5 spaces @ 3'-0" = 15'-0"
2 spaces @ 3'-0" = 6'-0"
15 spaces @ 6'-0" = 90'-0"
2 spaces @ 3'-0" = 6'-0"
℄ steel sheet piling
Symmetrical about ℄
3 spaces @ 3'-0" = 9'-0"
14 spaces @ 6'-0" = 84'-0"
4 spaces @ 3'-0" = 12'-0"
36'-0"
15'-6"
34 spaces @ 3'-0" = 102'-0"
TP-3
TP-2
TP-1
25'-0"
21'-0"
6 spaces @ 3'-6"
MONOLITH T-1R
FLOOD SIDE
12"x12" prestressed concrete pile (typ)
7 spaces @ 6'-0" = 42'-0"
7 spaces @ 6'-0" = 42'-0"
99'-0"
GATE SILL AND GATE OUTER SILL
LEGEND
12"x12" prestressed concrete batter piles.
12"φ class B untreated timber batter piles.
12"φ class B untreated timber vertical piles
12 φ class B untreated timber vertical test piles
TP-1: Tip El. -70.0
TP-2: Tip El. -77.0
TP-3: Tip El. -85.0
25'-0"
21'-0"
7 spaces @ 3'-0"
Steel sheet piling
50'-0"
45'-0"
10 spaces @ 4'-6"
5 spaces @ 9'-0"
50'-0"
45'-0"
10 spaces @ 4'-6"
℄ steel sheet piling
5 spaces @ 9'-0"
MONOLITH T-2R
MONOLITH T-3R
MONOLITH T-4R
PLAN
Scale: 1/8" = 1'-0"
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
PILE FOUNDATION PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970
FILE NO. H-2-25048

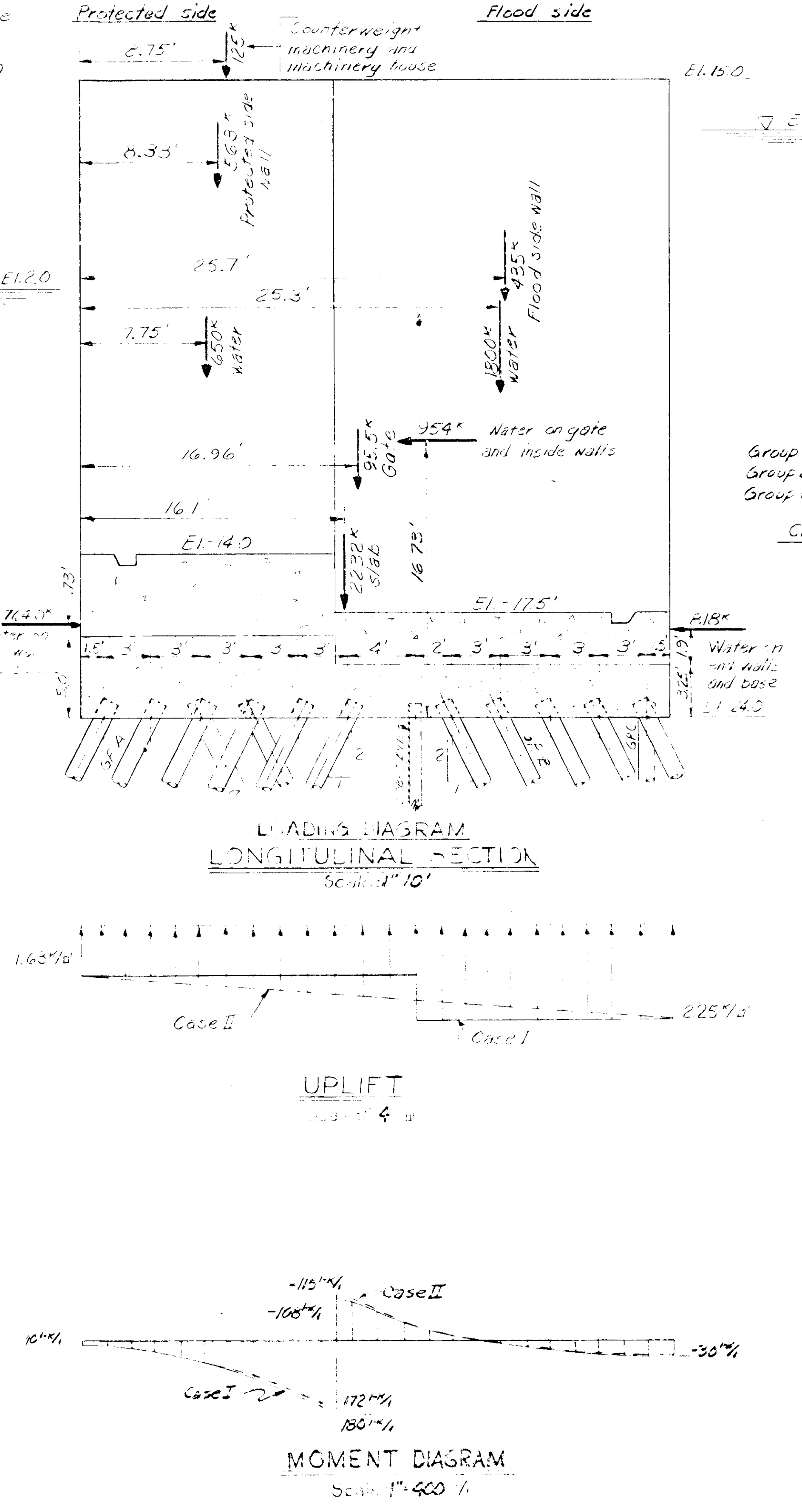
PLATE IV-8



LOADING DIAGRAM
TRANSVERSE SECTION
Scale: 1" = 10'

UPLIFT

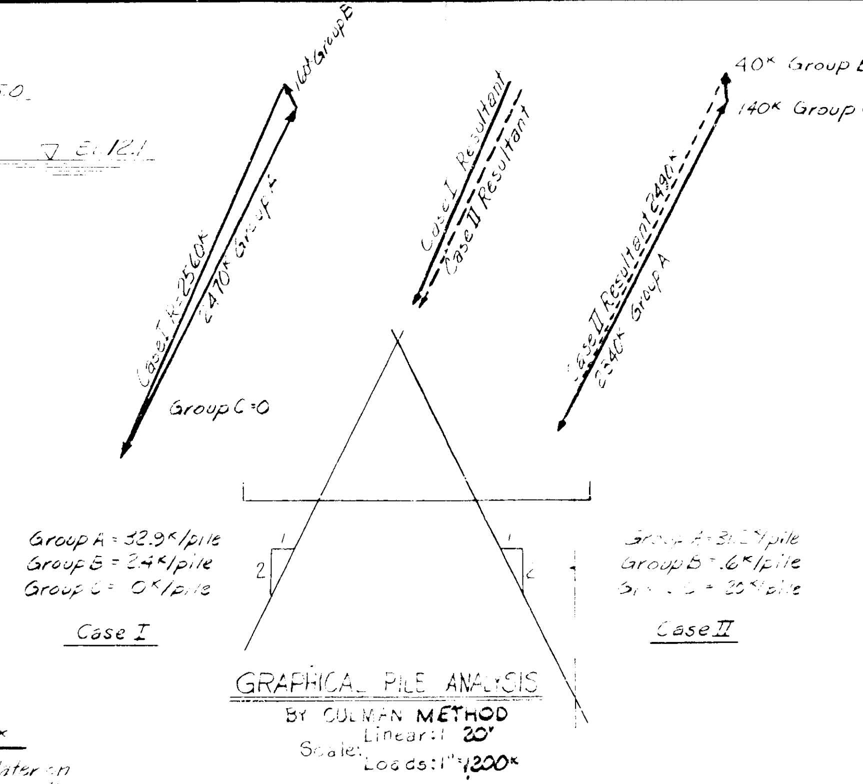
MOMENT DIAGRAM



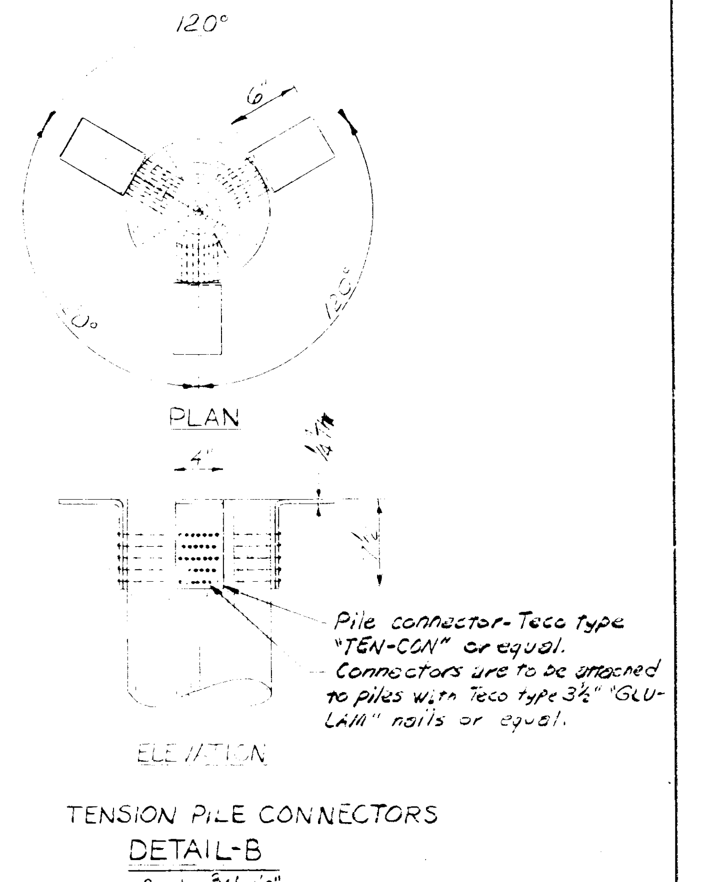
LOADING DIAGRAM
LONGITUDINAL SECTION
Scale: 1" = 10'

UPLIFT
Scale: 1" = 4'

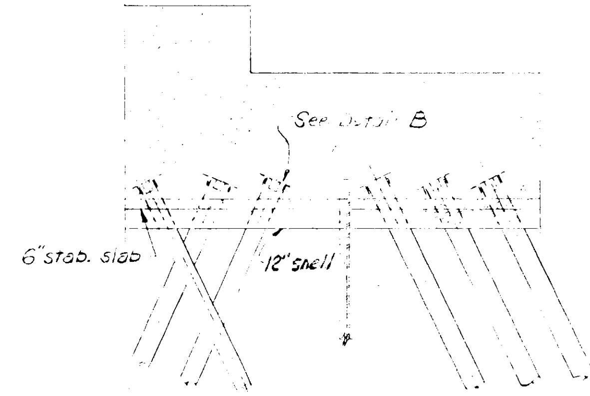
MOMENT DIAGRAM
Scale: 1" = 400'/'



GRAPHICAL PILE ANALYSIS
BY CULMAN METHOD
Scale: Linear: 1" = 20'
Loads: 1" = 1,200ᴷ



TENSION PILE CONNECTORS
DETAIL-B
Scale: ¾" = 1'0"



SECTION OF BASE SLAB
SHOWING TENSION PILE CONNECTORS
Scale 1" = 10'

Case I - Impervious sheet pile
Case II - Pervious sheet pile

COMPUTED PILE LOADS BY HRENNIKOFF METHOD

Pile row from protected side	Dist. from protected side (Ft.)	CASE I: Nº Piles	CASE I: Load/Pile Group A	CASE I: Nº Piles	CASE I: Load/Pile Group B	CASE I: Nº Piles	CASE I: Load/Pile Group C	CASE II: Nº Piles	CASE II: Load/Pile Group A	CASE II: Nº Piles	CASE II: Load/Pile Group B	CASE II: Nº Piles	CASE II: Load/Pile Group C
1	1.5	17.5	33.39ᴷ	0		0		17.5	31.86ᴷ	0		0	
2	4.5	17.5	32.73ᴷ	0		0		17.5	31.39ᴷ	0		0	
3	7.5	10.0	31.77ᴷ	7.5	7.56ᴷ	0		10.0	30.93ᴷ	7.5	4.06ᴷ	0	
4	10.5	10.0	30.82ᴷ	7.5	6.60ᴷ	0		10.0	30.47ᴷ	7.5	3.60ᴷ	0	
5	13.5	10.0	29.86ᴷ	0		0		10.0	30.00ᴷ	0		0	
6	16.5	0.0	28.91ᴷ	0		0		10.0	29.54ᴷ	0		0	
7	22.5	0		10.5	2.78ᴷ	0		0		10.5	1.74ᴷ	0	
8	25.5	0		10.5	1.82ᴷ	0		0		10.5	1.28ᴷ	0	
9	28.5	0		10.5	0.87ᴷ	0		0		10.5	0.82ᴷ	0	
10	31.5	0		10.5	-0.09ᴷ	0		0		10.5	0.35ᴷ	0	
11	34.5	0		10.5	-1.04ᴷ	7	12.57ᴷ	0		10.5	0.11ᴷ	7	14.90ᴷ

Notes:
Elevations are expressed in feet and refer to mean sea level.
Loads shown are for one-half of the structure.
Moments shown are for a one foot wide strip

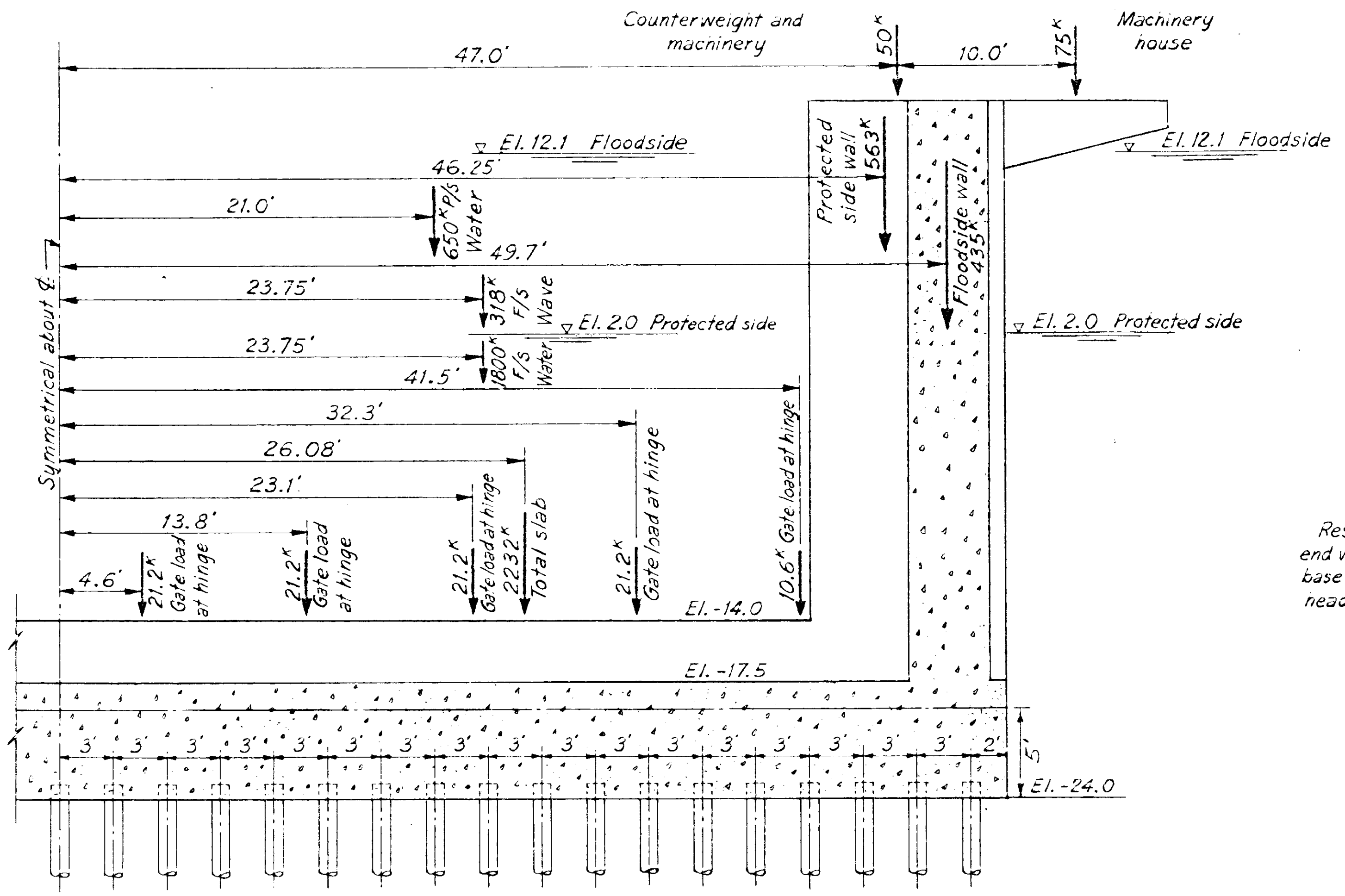
Case I*: Operating condition, gate closed. Water elevation 12.1 on Flood side, and elevation 2.0 on Protected side, sheet pile cutoff considered impervious, pile reactions assumed uniformly distributed in transverse direction.
Case II*: Same as Case I except sheet pile cutoff considered to be pervious.
* Cases I & II are Group I loadings

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO.2 — DETAIL DESIGN
REACH BI — TROPICAL BEND TO FORT JACKSON

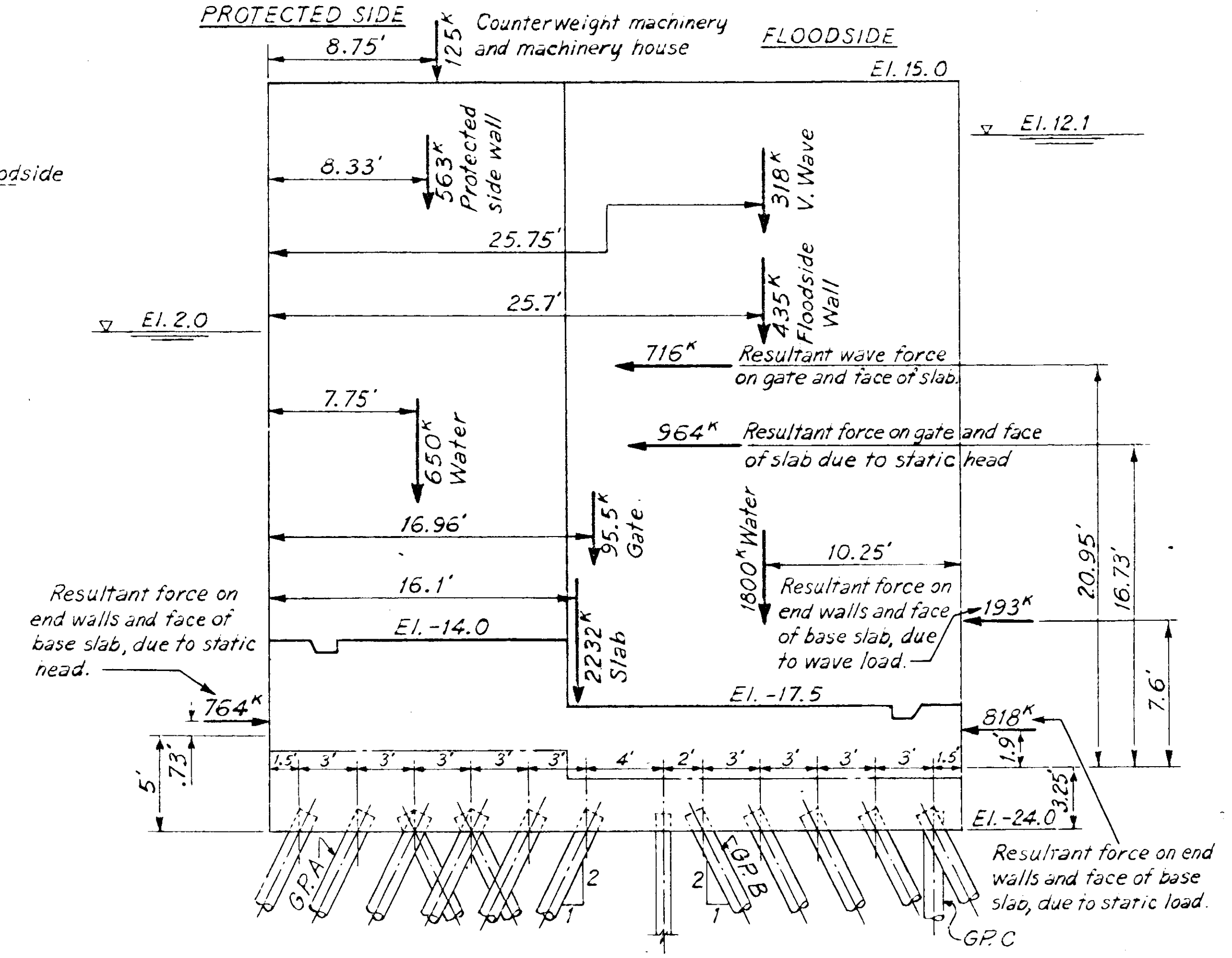
EMPIRE FLOODGATE
MOMENT DIAGRAMS AND PILE REACTIONS FOR BASE SLAB

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

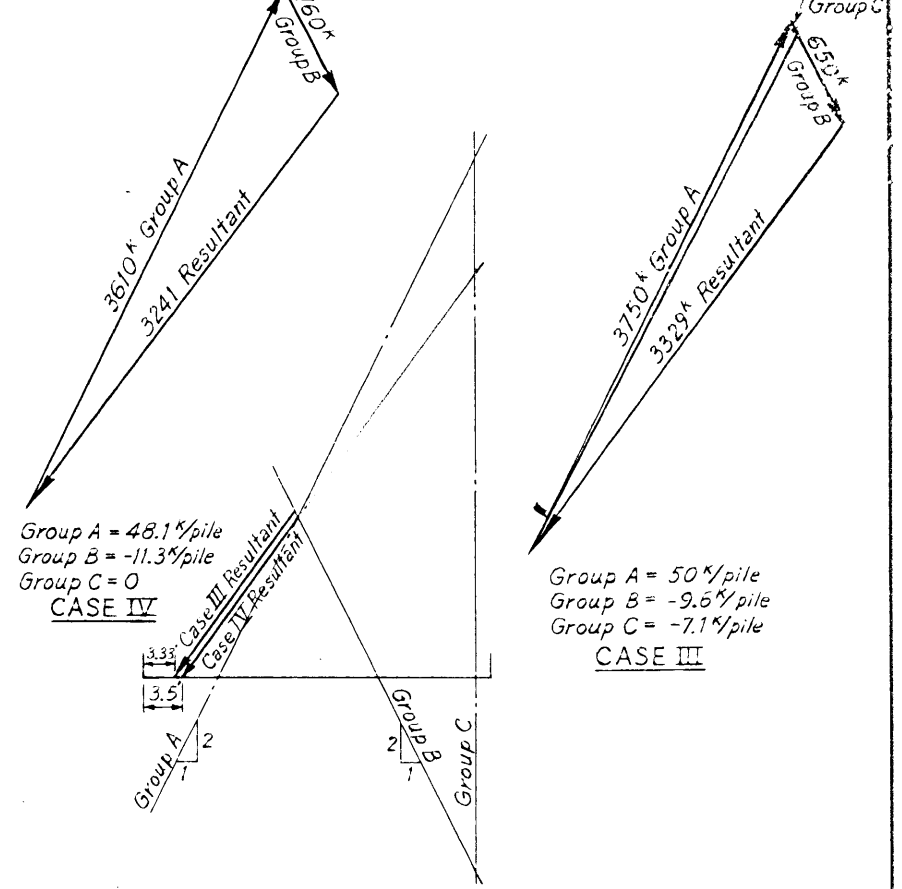
OCTOBER 1970 FILE NO. H-2-25048



LOADING DIAGRAM
TRANVERSE SECTION
Scale: 1"=10'

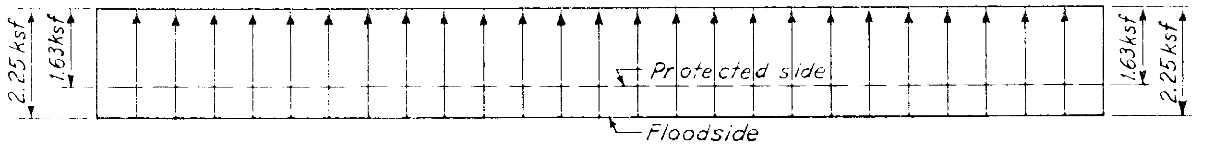


LOADING DIAGRAM
LONGITUDINAL SECTION
Scale: 1"=10'

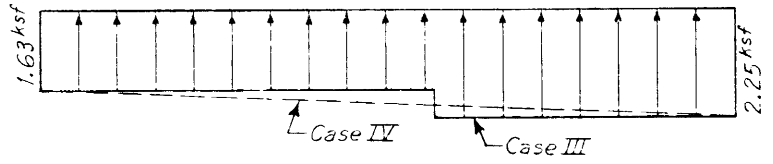


GRAPHICAL PILE ANALYSIS
BY CULMAN METHOD
Scale: 1"=1,200K

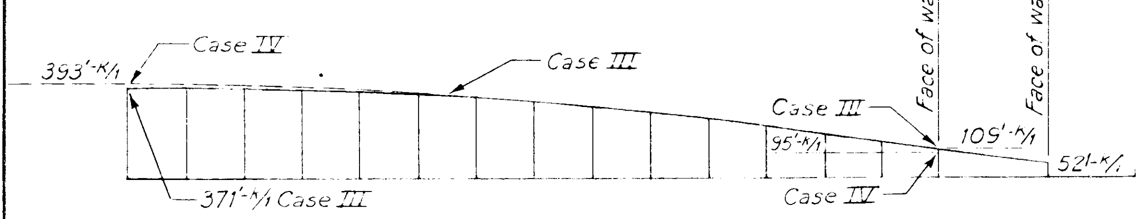
COMPUTED PILE LOADS BY HRENNIKOFF METHOD													
Pile row from protected side	Dist. from protected side (ft.)	CASE III						CASE IV					
		No. Piles	Load/pile Group A	No. Piles	Load/pile Group B	No. Piles	Load/pile Group C	No. Piles	Load/pile Group A	No. Piles	Load/pile Group B	No. Piles	Load/pile Group C
1	1.5	17.5	51.92K	0		0		17.5	49.61K	0		0	
2	4.5	17.5	50.09K	0		0		17.5	48.38K	0		0	
3	7.5	10.0	48.25K	7.5	0.31K	0		10.0	47.14K	7.5	-4.07	0	
4	10.5	10.0	46.41K	7.5	-1.53K	0		10.0	45.91K	7.5	-5.30	0	
5	13.5	10.0	44.58K	0		0		10.0	44.68K	0		0	
6	16.5	10.0	42.74K	0		0		10.0	43.45K	0		0	
7	22.5	0		10.5	-8.87K	0		0		10.5	-10.23	0	
8	25.5	0		10.5	-10.71K	0		0		10.5	-11.46	0	
9	28.5	0		10.5	-12.55K	0		0		10.5	-12.69	0	
10	31.5	0		10.5	-14.38K	0		0		10.5	-13.92	0	
11	34.5	0		10.5	-16.22K	7	8.67K	0		10.5	-15.15	7	11.69
Allowable T = 15 X 1.33 = 20K				Allowable C = 40 X 1.33 = 53K									



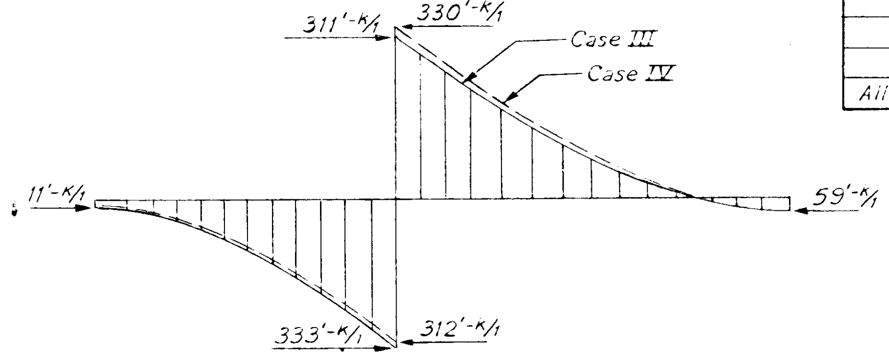
UPLIFT
Scale: 1"=4 K/□'



UPLIFT
Scale: 1"=4 ksf



MOMENT DIAGRAM
Scale: 1" 800'-K/1



MOMENT DIAGRAM
Scale: 1"=400'-K/1

CASE III:
Flood W/wave, gate closed, SWL at 12.1 on flood side and 2.0 on protected side, 100% uplift W/impervious cutoff.

CASE IV:
Flood W/wave, same as III except uplift with pervious cutoff.

CASES III and IV are group 2 loadings.

Note:
Elevations are in expressed in feet and refer to mean sea level.
Loads shown are for one half of the structure.
Moments shown are for a foot wide strip.

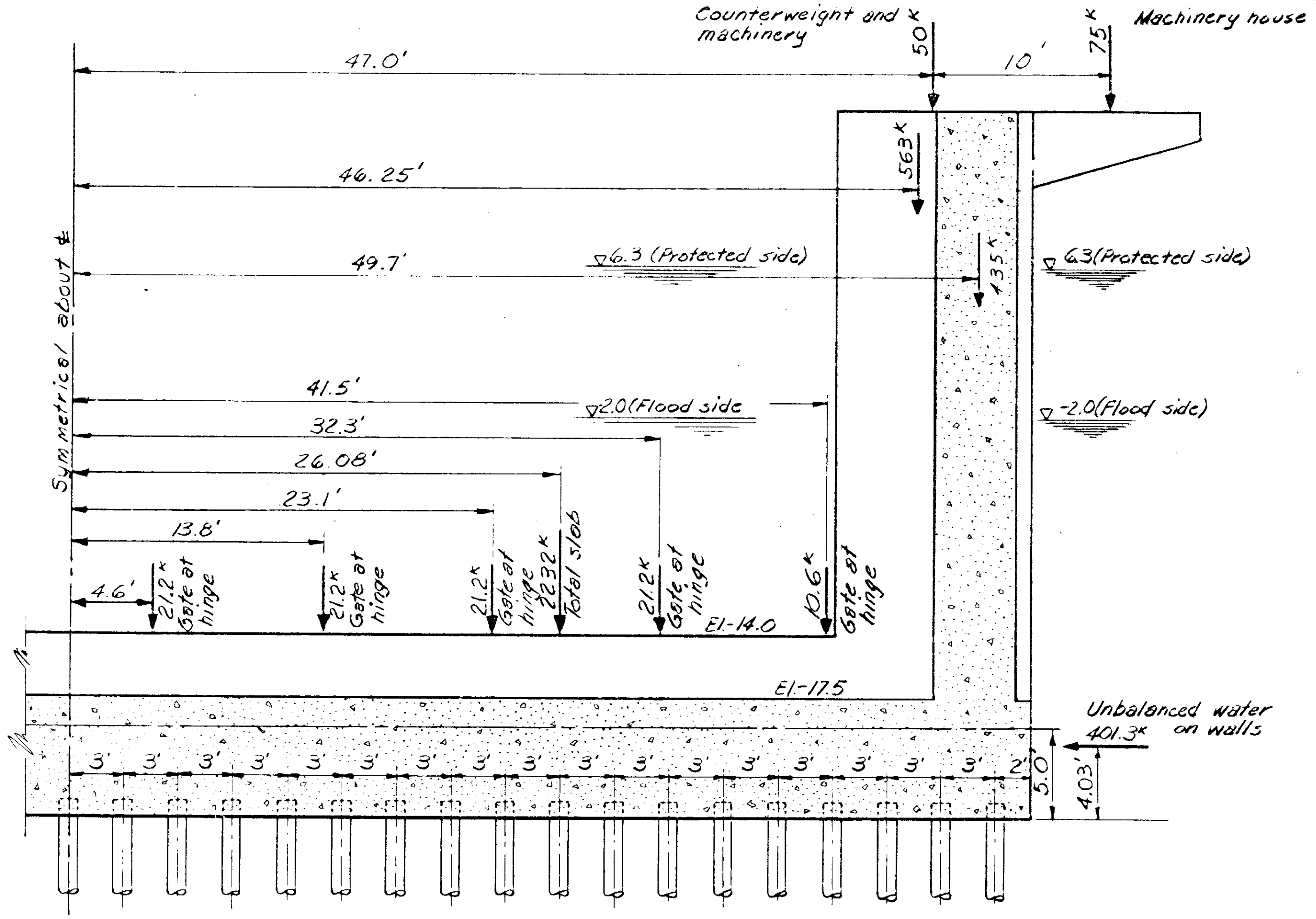
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH BI – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
MOMENT DIAGRAMS AND PILE REACTIONS FOR BASE SLAB

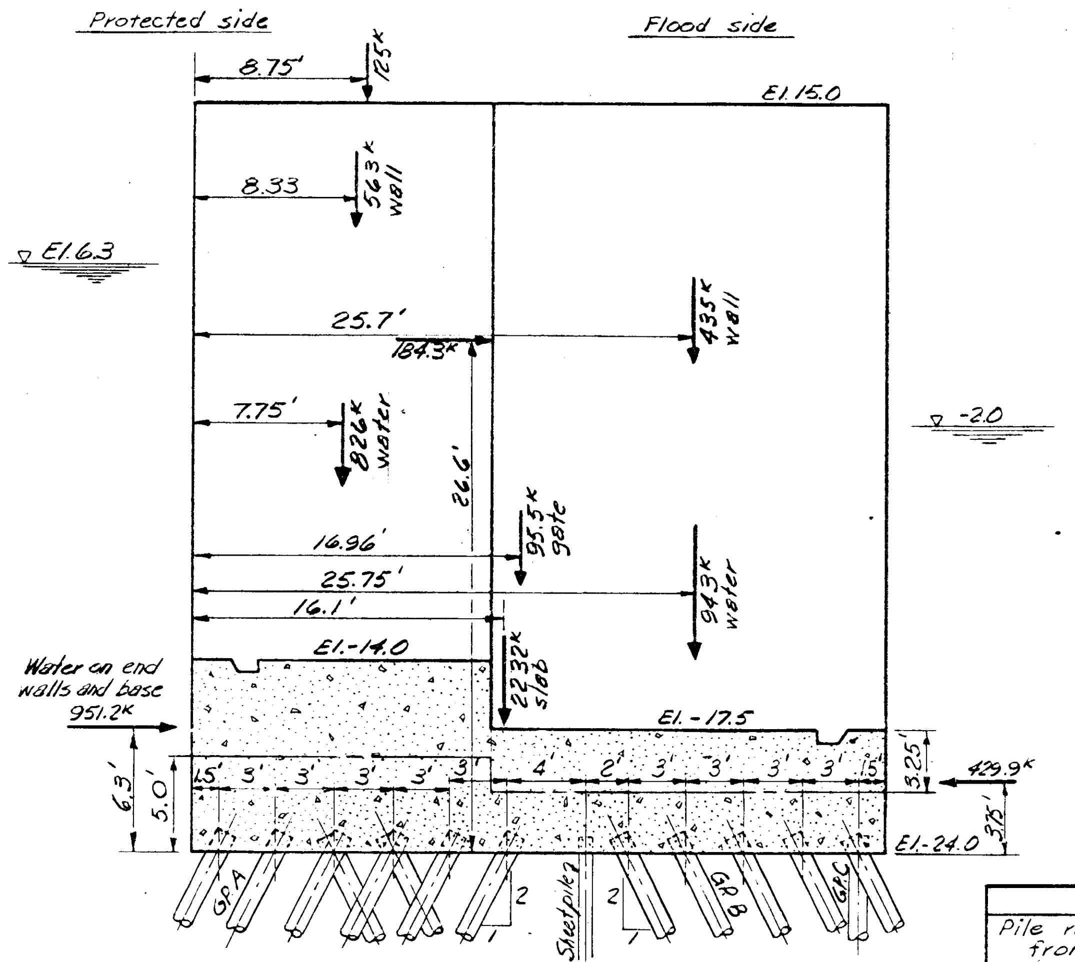
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970
FILE NO. H-2-25048

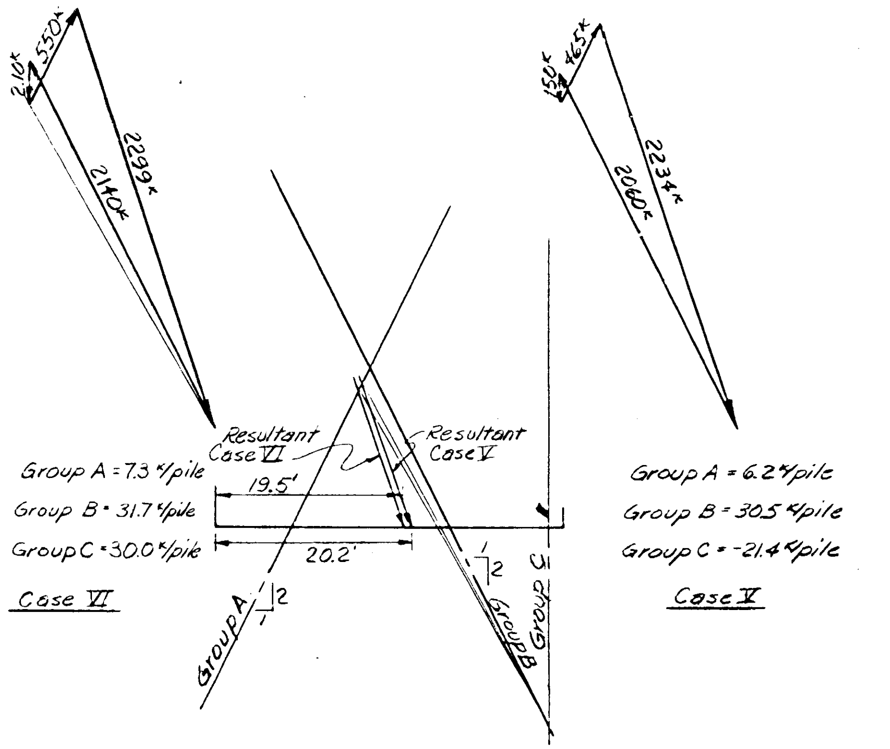
PLATE IV-10



LOADING DIAGRAM
TRANSVERSE SECTION
Scale: 1" = 10'



LOADING DIAGRAM
LONGITUDINAL SECTION
Scale: 1" = 10'



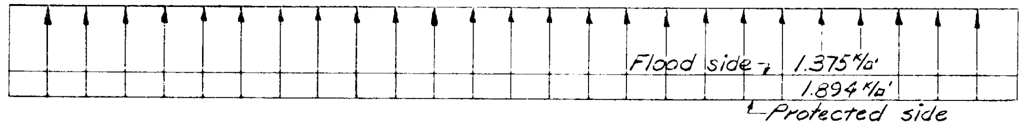
GRAPHICAL PILE ANALYSIS
By Culman method
Scale: 1" = 1000 K

COMPUTED PILE LOADS BY HRENNIKOFF METHOD													
Pile row from protected side	Pile row from protected side	CASE V						CASE VI					
		No. piles	Load/pile Group A	No. piles	Load/pile Group B	No. piles	Load/pile Group C	No. piles	Load/pile Group A	No. piles	Load/pile Group B	No. piles	Load/pile Group C
1	1.5	17.5	8.27K	0		0		17.5	9.77K	0		0	
2	4.5	17.5	6.86K	0		0		17.5	7.95K	0		0	
3	7.5	10.0	5.45K	7.5	35.96K	0		10.0	6.14K	7.5	38.82K	0	
4	10.5	10.0	4.03K	7.5	34.55K	0		10.0	4.32K	7.5	37.01K	0	
5	13.5	10.0	2.62K	0		0		10.0	2.51K	0		0	
6	16.5	10.0	1.21K	0		0		10.0	0.69K	0		0	
7	22.5	0		10.5	28.90K	0		0		10.5	29.75K	0	
8	25.5	0		10.5	27.48K	0		0		10.5	27.93K	0	
9	28.5	0		10.5	26.07K	0		0		10.5	26.12K	0	
10	31.5	0		10.5	24.66K	0		0		10.5	24.30K	0	
11	34.5	0		10.5	23.24K	7	8.93K	0		10.5	22.49K	7	6.87
Allowable C = 40K, allowable T = 15K													

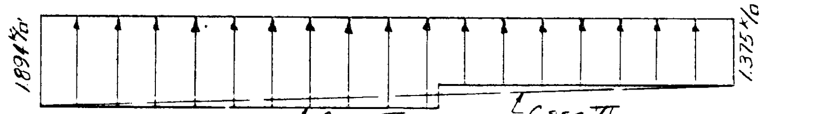
CASE V - Reverse head, gate closed, still water level at -2' M.S.L. on flood side and +6.3' M.S.L. on protected side, 100% uplift with impervious cutoff.

CASE VI - Same as Case V except sheet pile cutoff is pervious.

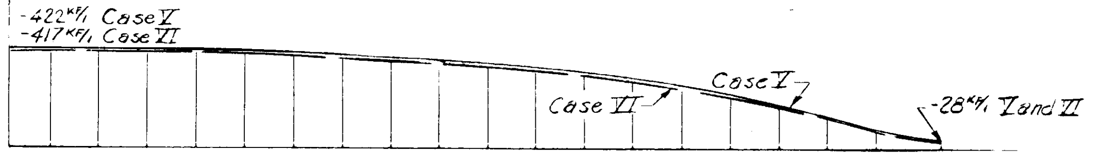
Notes: Elevations are expressed in feet and refer to mean sea level.
All loads are for half the structure.
All moments shown are for 1 ft. wide strip.



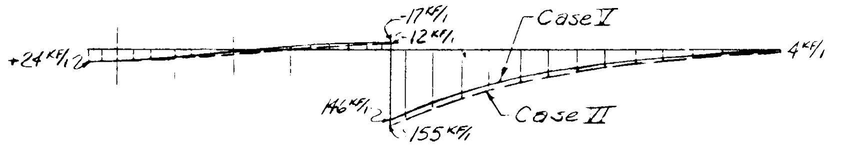
UPLIFT
Scale: 1" = 4 K/□'



UPLIFT
Scale: 1" = 4 K/□'



MOMENT DIAGRAMS
Scale: 1" = 800 'K



MOMENT DIAGRAMS
Scale: 1" = 400 K'/'

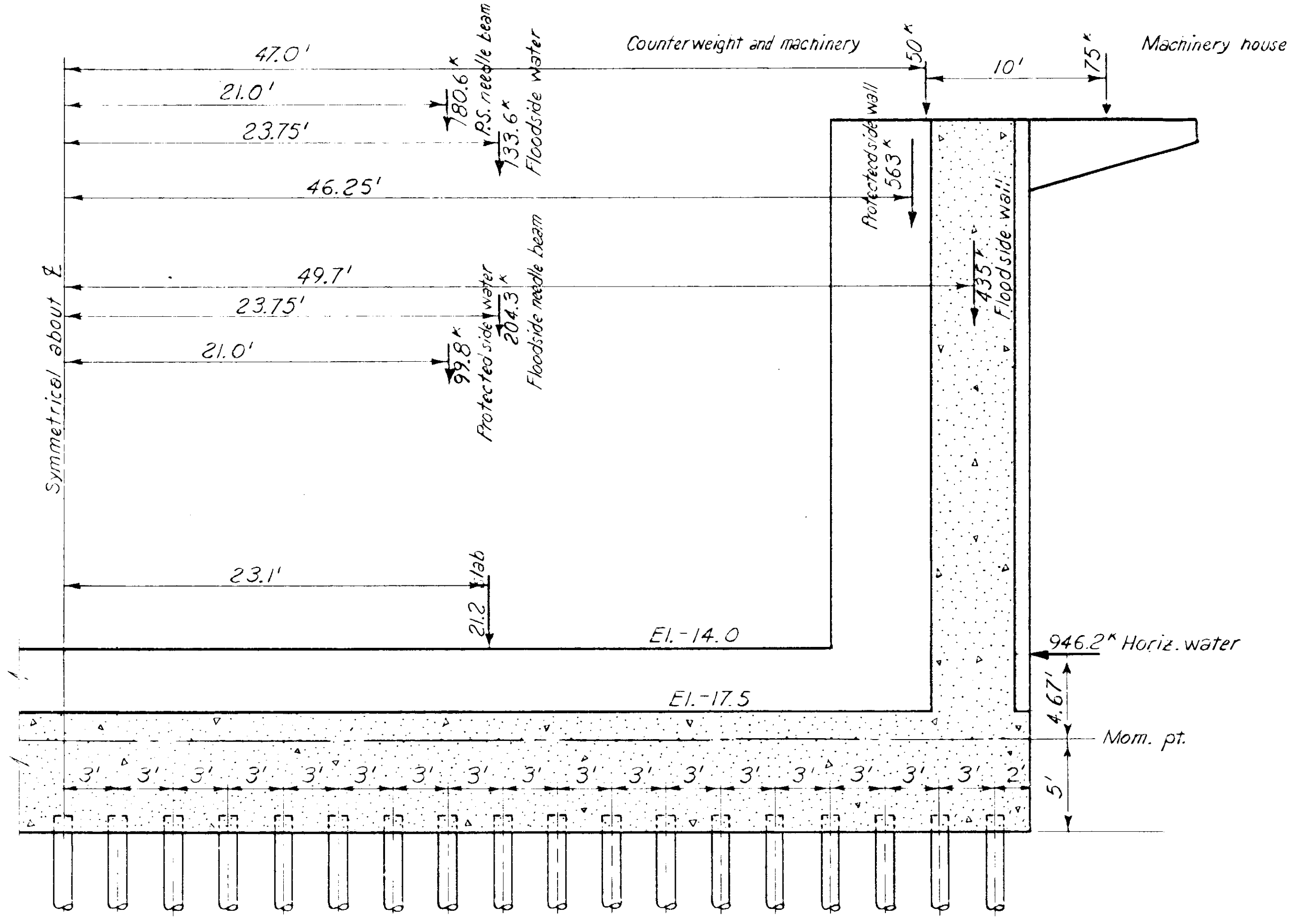
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE MOMENT DIAGRAMS AND PILE REACTIONS FOR BASE SLAB

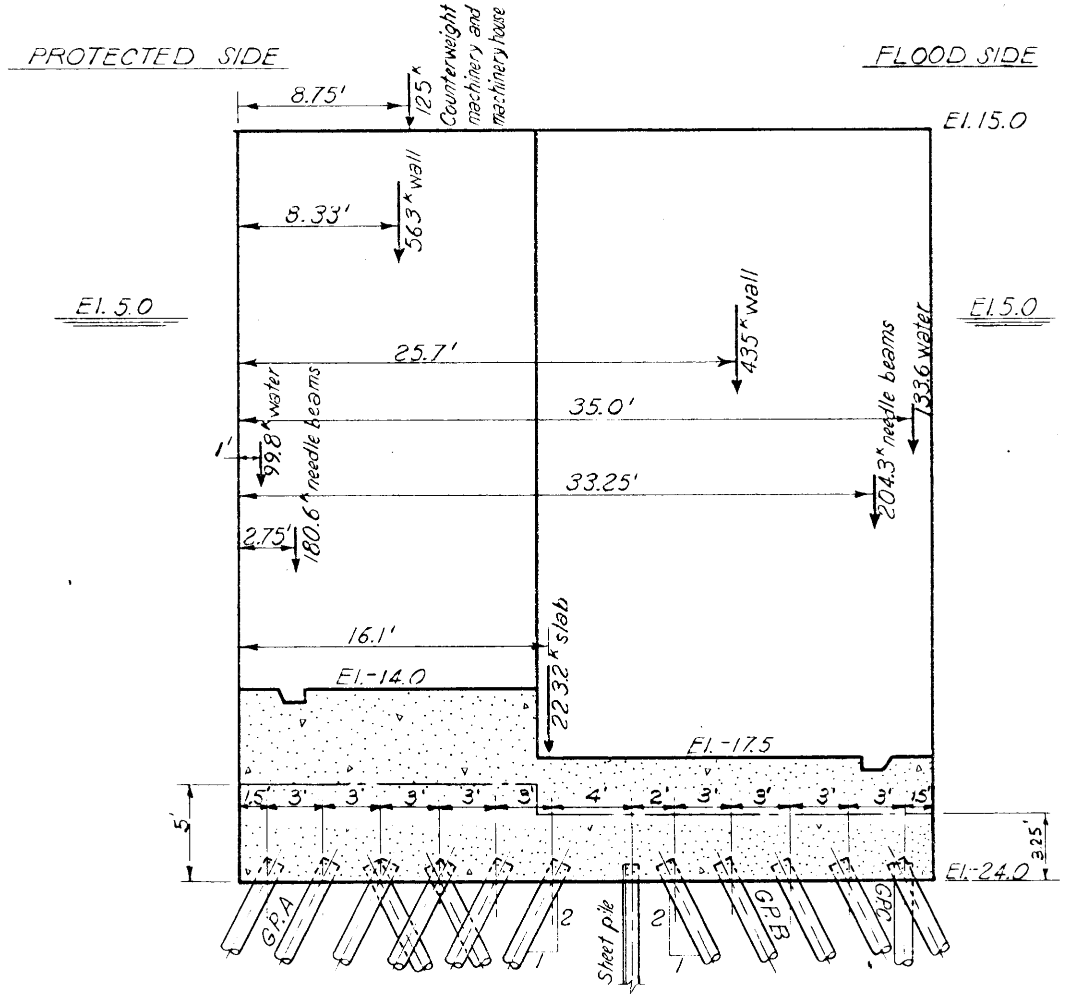
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048

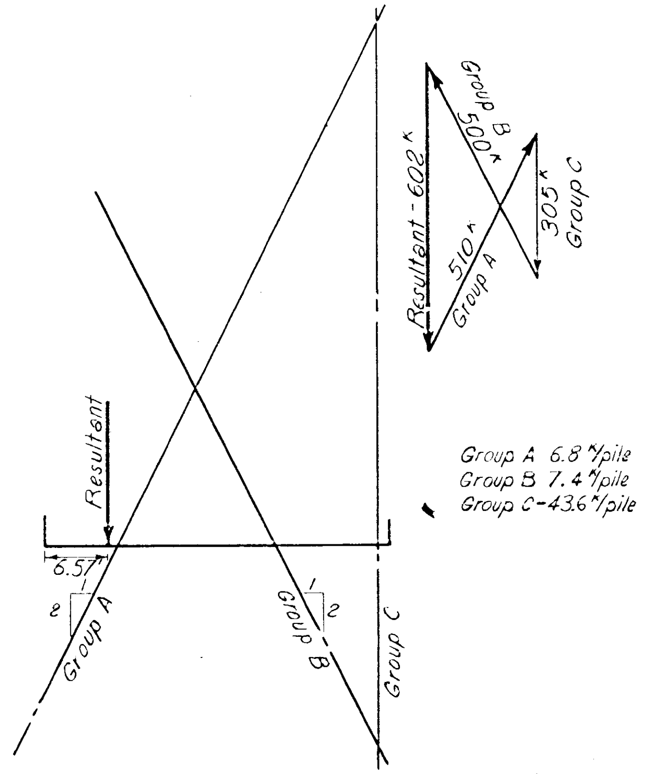
PLATE IV-11



LOADING DIAGRAM
TRANSVERSE SECTION
Scale: 1" = 10'

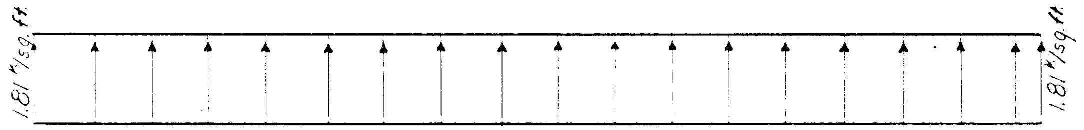


LOADING DIAGRAM
LONGITUDINAL SECTION
Scale: 1" = 10'

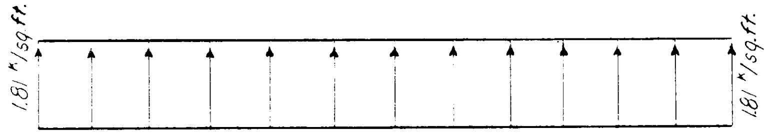


GRAPHICAL PILE ANALYSIS
BY CULMAN METHOD
CASE VII: Dewatered condition, gate removed, needle beams in place water el. + 5.0 on all sides.
Scale: Linear 1" = 20'
Loads 1" = 400^K

COMPUTED PILE LOADS BY HRENNIKOFF METHOD							
Pile row from protected side	Dist. from protected side (ft)	CASE VII					
		No. Piles	Load/pile Group A	No. Piles	Load/pile Group B	No. Piles	Load/pile Group C
1	1.5	17.5	9.21^K	0		0	
2	4.5	17.5	7.25^K	0		0	
3	7.5	10.0	5.29^K	7.5	16.06^K	0	
4	10.5	10.0	3.33^K	7.5	14.10^K	0	
5	13.5	10.0	1.38^K	0		0	
6	16.5	10.0	-0.58^K	0		0	
7	22.5	0		10.5	6.27^K	0	
8	25.5	0		10.5	4.31^K	0	
9	28.5	0		10.5	2.35^K	0	
10	31.5	0		10.5	0.40^K	0	
11	34.5	0		10.5	-1.56^K	7	-7.77^K

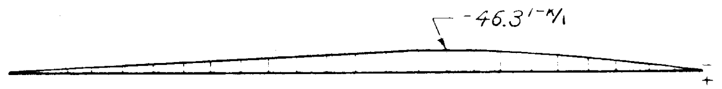


UPLIFT
Scale: 1" = 4^K/sq.ft.



UPLIFT
Scale: 1" = 4^K/sq.ft.

MOMENT DIAGRAM
Scale: 1" = 800'^K



MOMENT DIAGRAM
Scale: 1" = 400'^K

Notes:
Elevations are expressed in feet and refer to mean sea level.
Loads shown are for one-half of the structure.
Moments shown are for a one foot wide strip.

NEW ORLEANS TO VENICE, LA.
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REACH BI-TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
MOMENT DIAGRAMS AND PILE REACTIONS FOR BASE SLAB

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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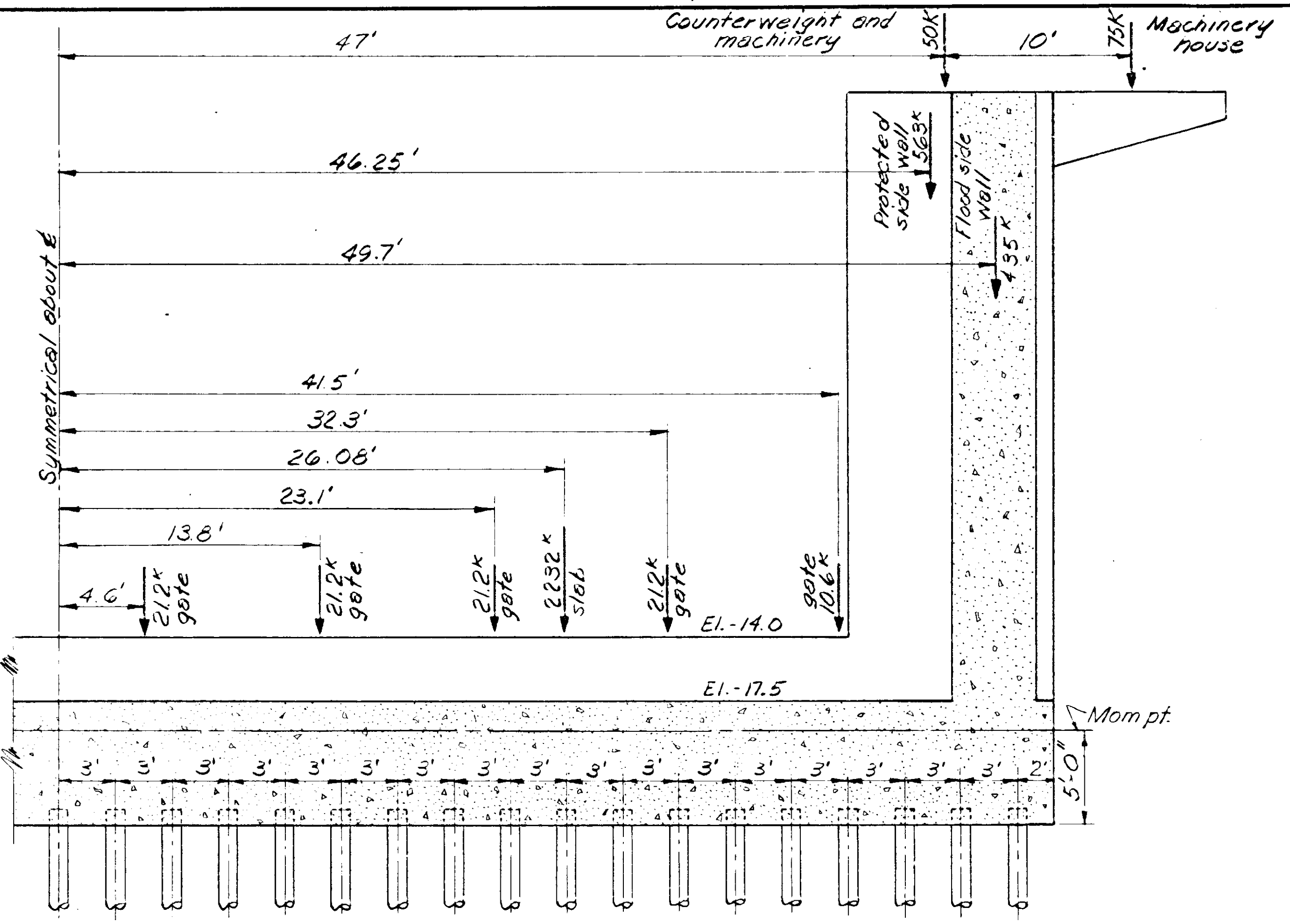
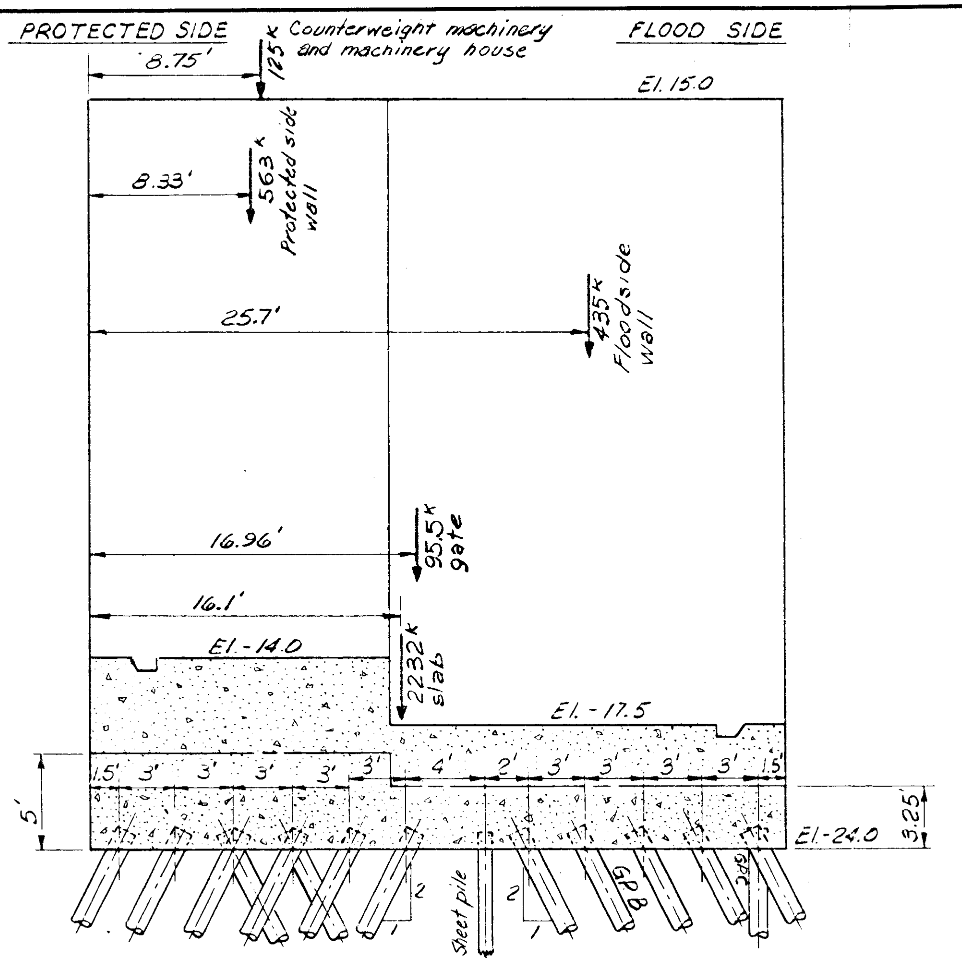
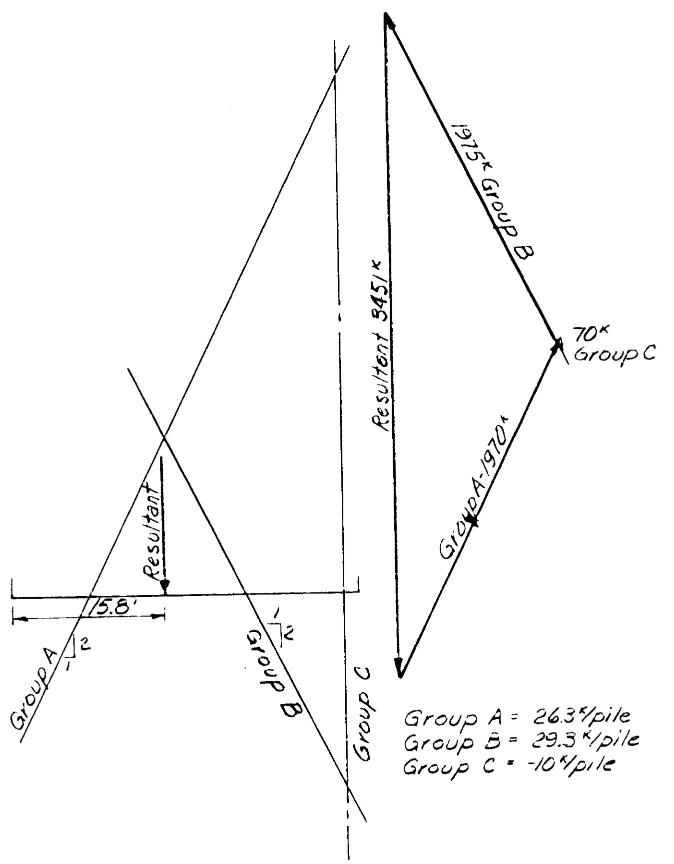
PLATE IV-12



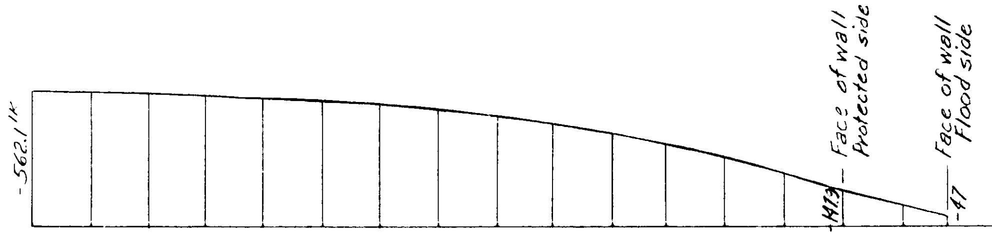
LOADING DIAGRAM
TRANSVERSE SECTION
Scale: 1"=10'



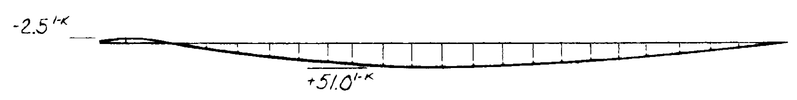
LOADING DIAGRAM
LONGITUDINAL SECTION
Scale: 1"=10'



GRAPHICAL PILE ANALYSIS
BY CULMAN METHOD
CASE VIII: Construction Condition
No Water, No Uplift
Scale: Linear: 1"=20'
Loads 1"=1000K



MOMENT DIAGRAM
Scale: 1"=800'-K



MOMENT DIAGRAM
Scale: 1"=400'-K

COMPUTED PILE LOADS BY HRENNIKOFF METHOD							
Pile row from protected side	Dist. from protected side (ft.)	CASE VIII					
		No. Piles	Load/pile Group A	No. Piles	Load/pile Group B	No. Piles	Load/pile Group C
1	1.5	17.5	27.61	0		0	
2	4.5	17.5	26.28	0		0	
3	7.5	10.0	24.96	7.5	34.52	0	
4	10.5	10.0	23.63	7.5	33.19	0	
5	13.5	10.0	22.31	0		0	
6	16.5	10.0	20.98	0		0	
7	22.5	0		10.5	27.89	0	
8	25.5	0		10.5	26.57	0	
9	28.5	0		10.5	25.24	0	
10	31.5	0		10.5	23.92	0	
11	34.5	0		10.5	22.59	7	19.91

Notes:
Elevations are expressed in feet and refer to mean sea level.
Loads shown are for one-half of the structure.
Moments shown are for a one foot wide strip.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO.2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
MOMENT DIAGRAMS AND PILE
REACTIONS FOR BASE SLAB

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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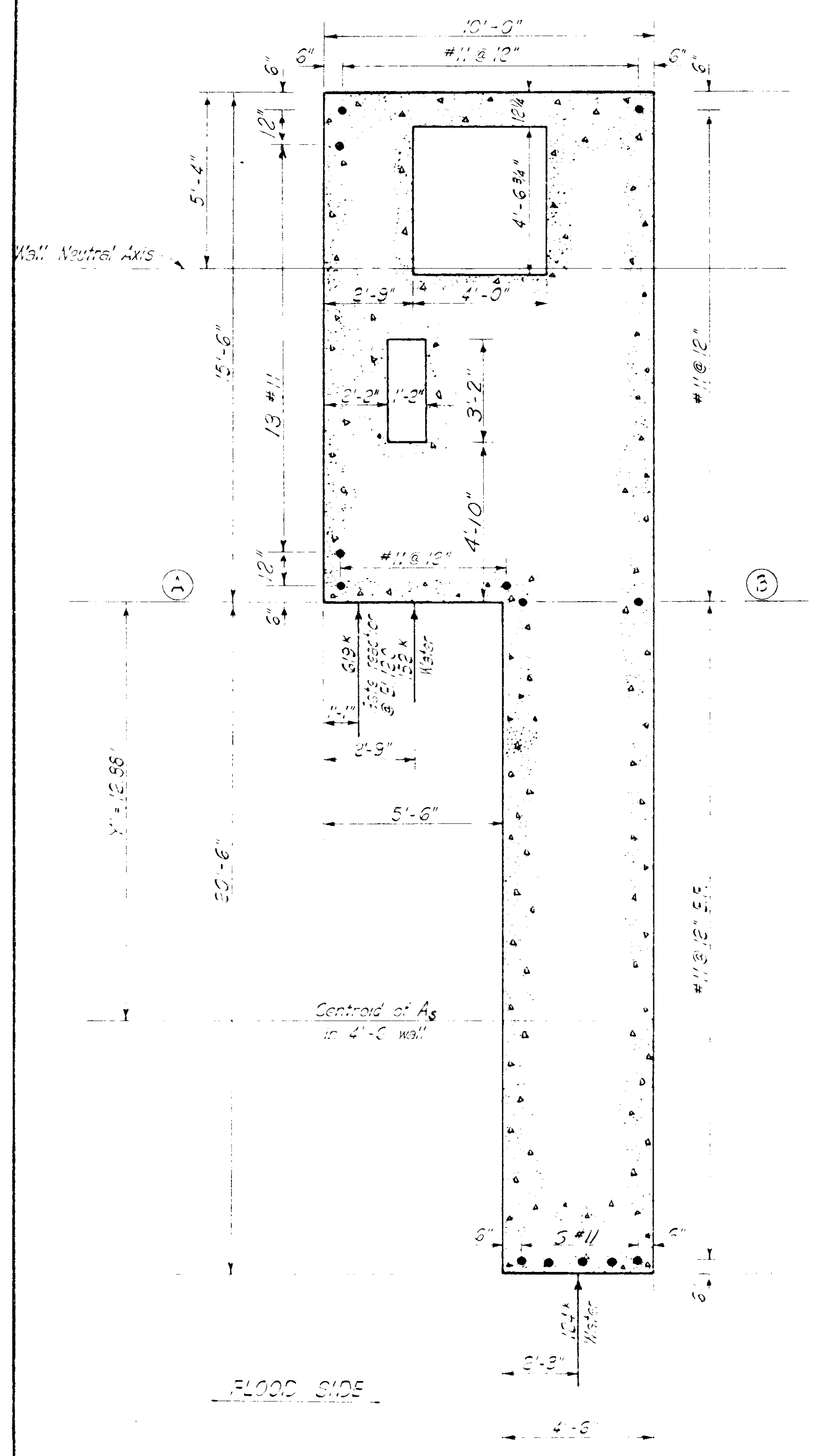
PLATE IV-13

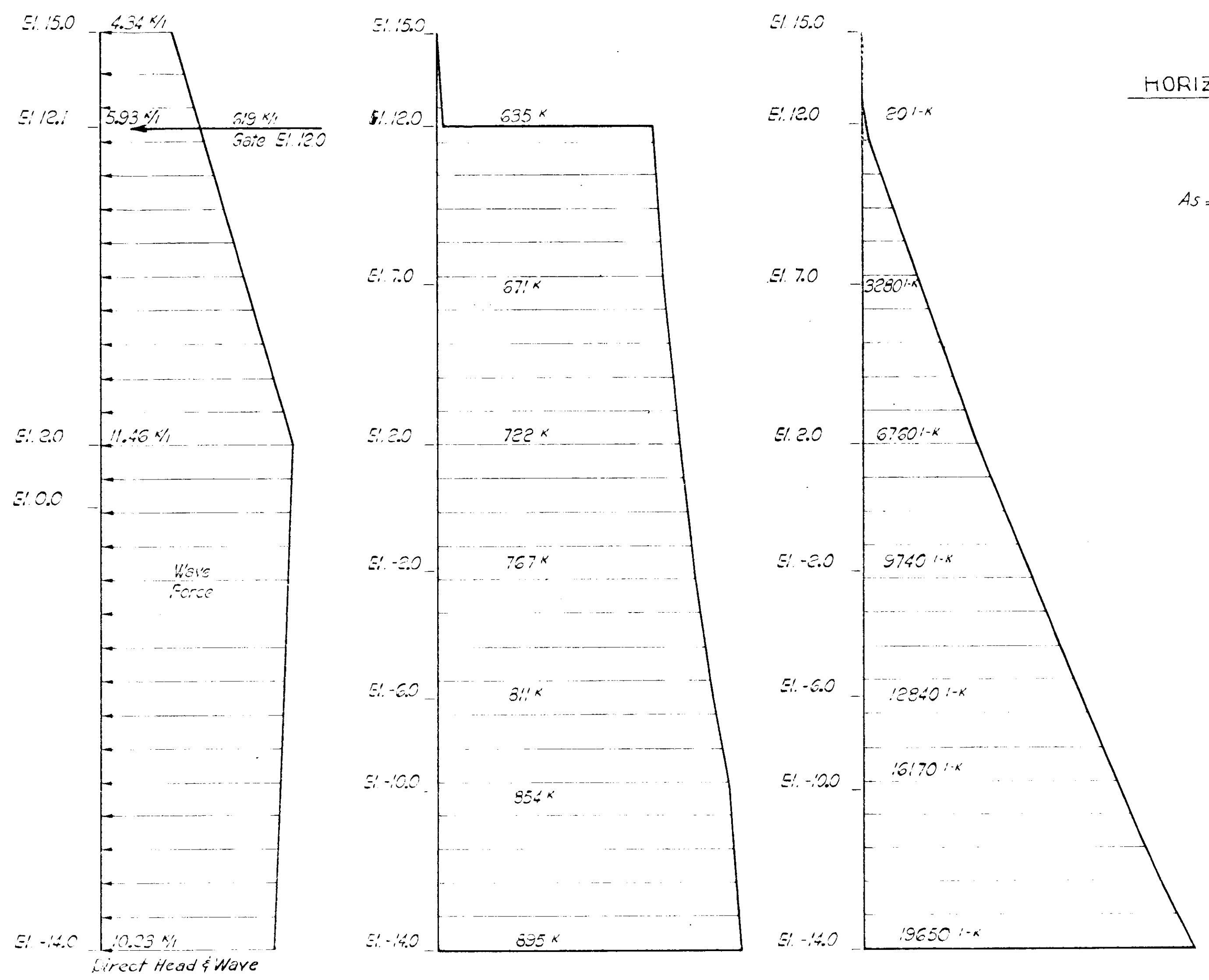
PROTECTED SIDE



FLOOD SIDE

PLAN OF WALL AT EL. -14.0

Scale 3/16" = 1'-0"



LOAD

Scale 1" = 10k

SHEAR DIAGRAM

Scale 1" = 500k

MOMENT DIAGRAM

Scale 1" = 10,000'-k

WALL

Scale 3/16" = 1'-0"

CASE III CRITICAL LOADING

Flood with wave; gate closed. SWL @ +12.1 on flood side and +2.0 on protected side, 100% uplift. Group Z loading 1.33% increase in allowable stresses.

HORIZ. LONG. REINFORCEMENT @ A-B

$\Sigma M_{A-B} = \Sigma f_s A_s$

$= 12.88^K \times 682.03^K$

$= 8788'^{-K}$

$A_s = \dfrac{M}{f_s \times jd} = \dfrac{8788 \times 12}{24 \times 0.9 \times 336} = 14.4 \text{ sq.in.}$

MOMENT @ EL.-14.0

10.23 x 16 x 8	= 1310'-K
.5 x 16 x 1.23 x 10.67	= 100'-K
5.93 x 10.1 x 21.05	= 1260'-K
5.53 x 10.1 x .5 x 19.37	= 540'-K
4.34 x 2.9 x 27.55	= 350'-K
1.59 x 2.9 x .5 x 27.07	= 60'-K
619 x 26	= 16100'-K
Σ M-14	= 19,720'-K

$I_{total} = I_{concrete} + I_{steel}$

$= 8,130,400 \text{ in}^4 + 45,424,000$

$= 53,554,400 \text{ in}^4$

$f_c \text{ Max.} = \dfrac{19,720 \times 12 \times 64}{53,554,400} = 283 \text{ PSI}$

$f_s \text{ Max.} = \dfrac{19,720 \times 12 \times 362 \times 9}{53,554,400} = 14,400 \text{ PSI}$

$f_c = 3000 \times 1.33 = 4000 \text{ PSI}$

$f_c = 1050 \times 1.33 = 1400 \text{ PSI}$

$f_s = 18,000 \times 1.33 = 24,000 \text{ PSI}$

Notes:

Elevations are expressed in feet and refer to mean sea level.

Forces shown are for entire 10' width.

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DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN

REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

WALL DESIGN ANALYSIS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS

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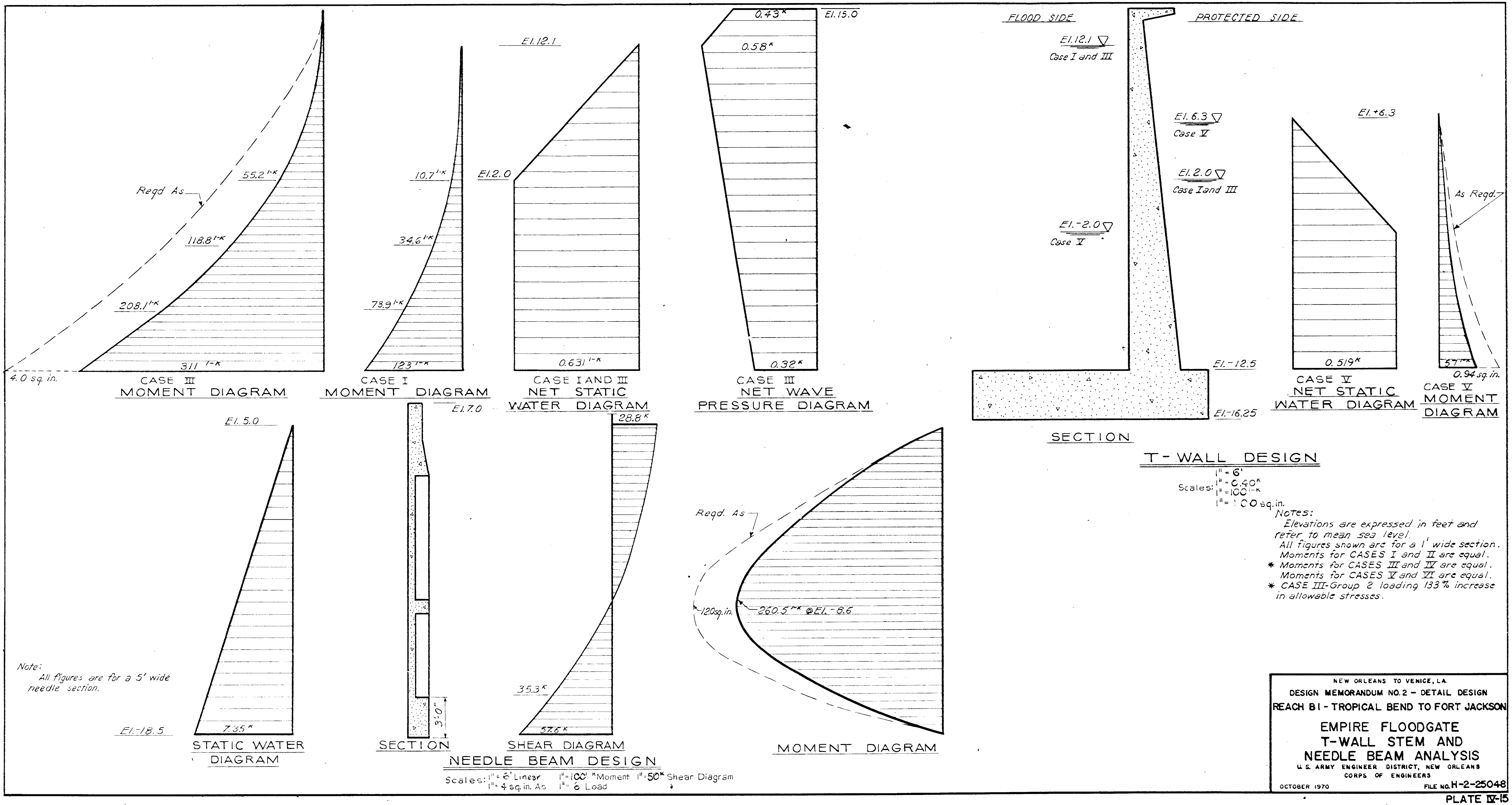
PLATE IV-14



NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
T-WALL STEM AND
NEEDLE BEAM ANALYSIS

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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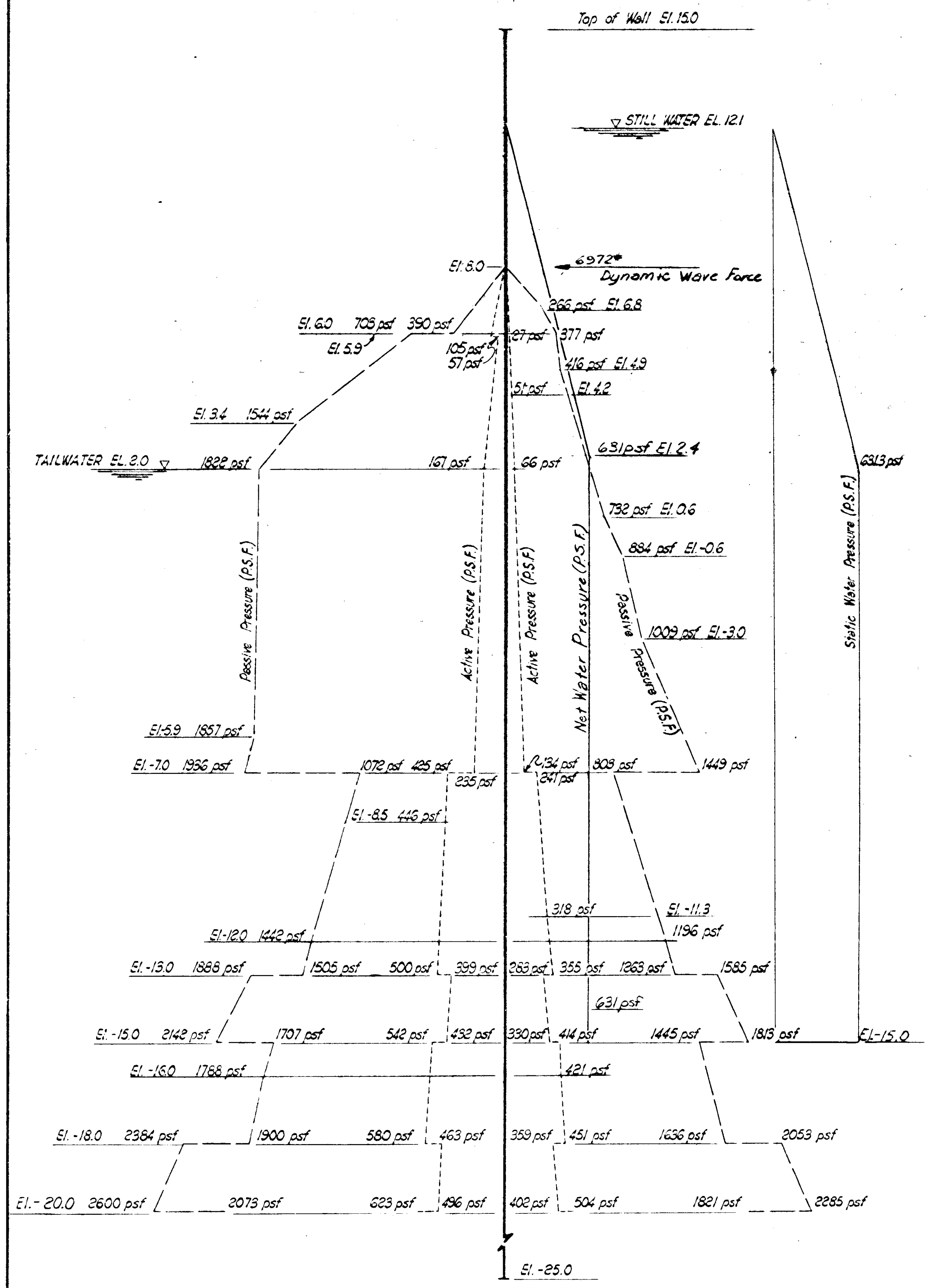
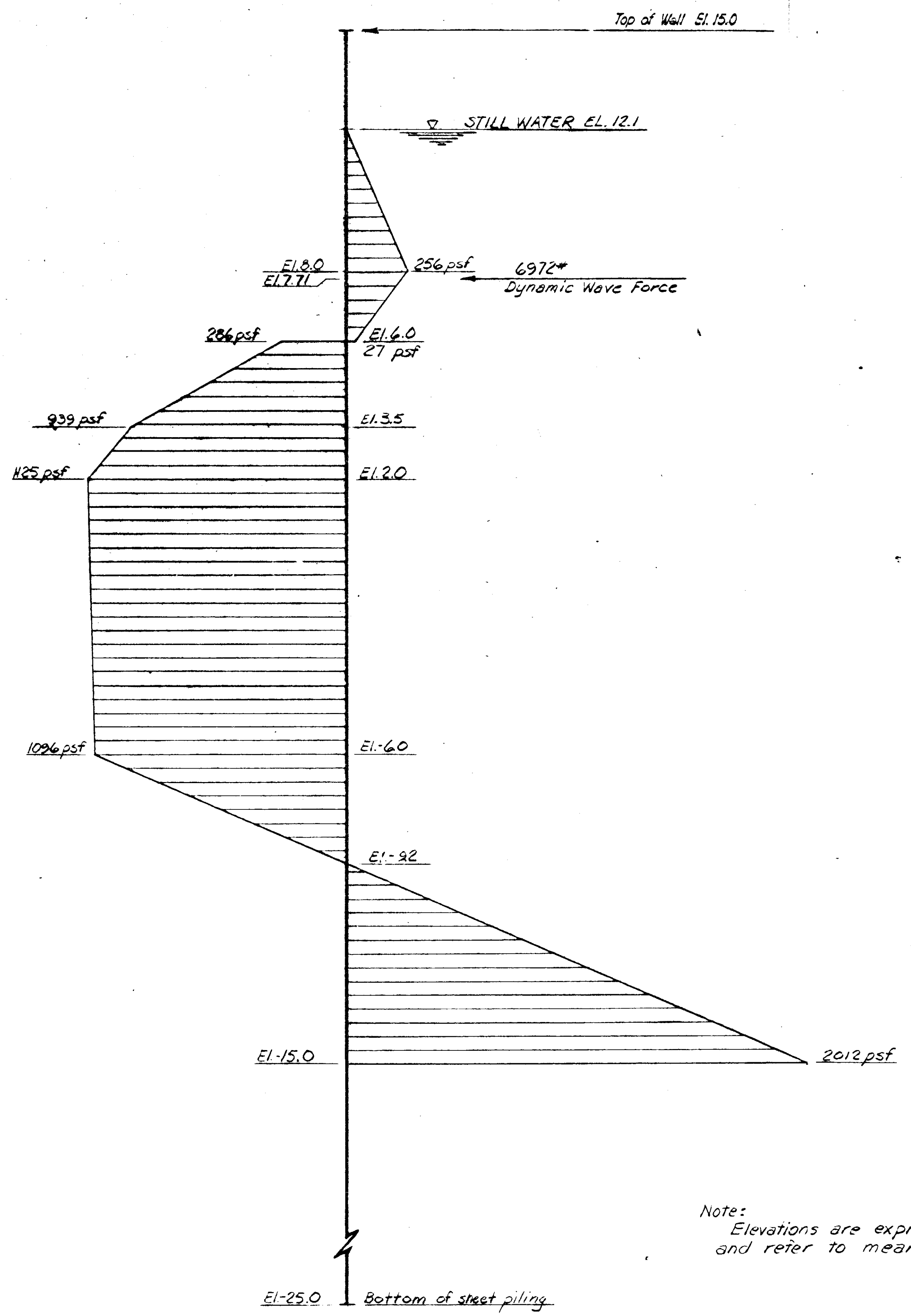


PRESSURE DIAGRAM (F.S. = 1.25 - S CASE)

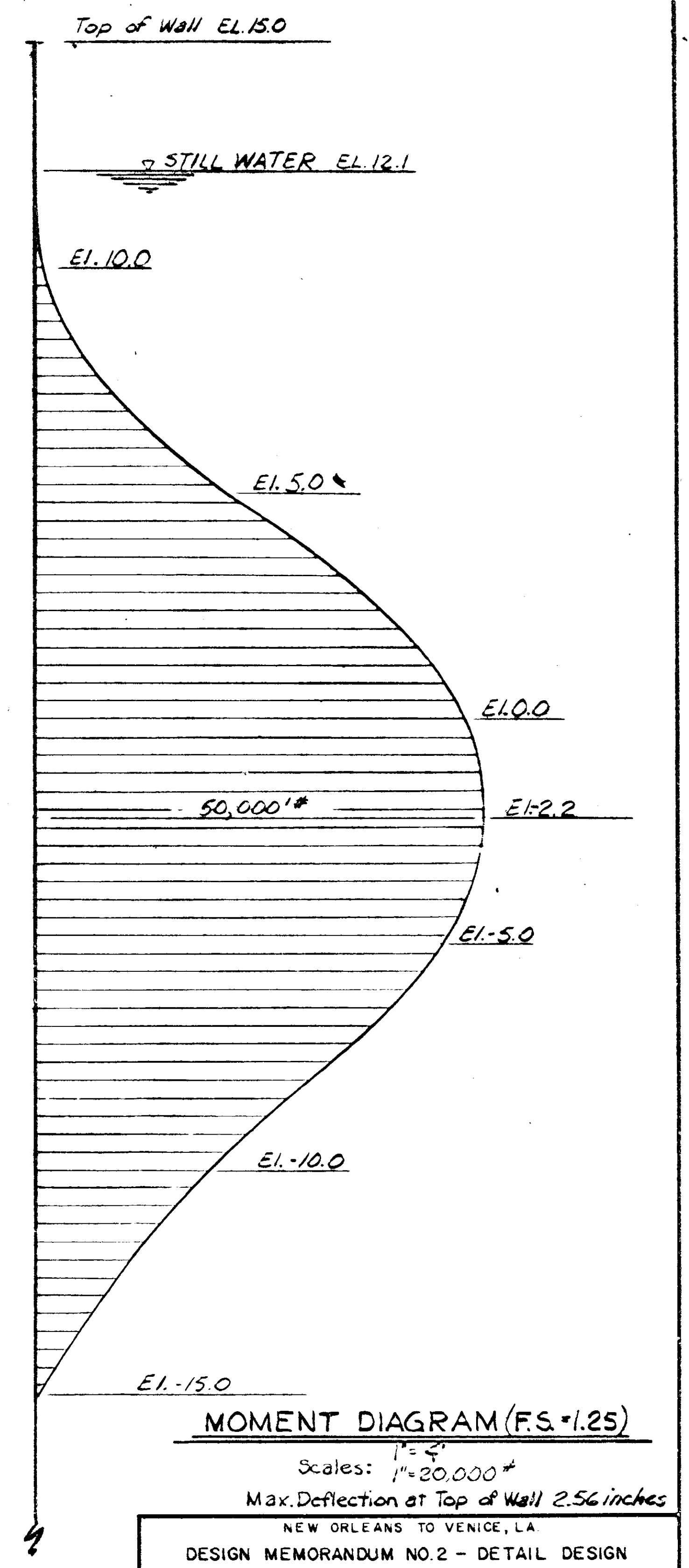
Scales: 1" = 4'
1" = 1000 P.S.F.



NET PRESSURE DIAGRAM (F.S. = 1.25)

Scales: 1" = 4'
1" = 600 PSF

Note:
Elevations are expressed in feet and refer to mean sea level.



MOMENT DIAGRAM (F.S. = 1.25)

Scales: 1" = 4'
1" = 20,000#

Max. Deflection at Top of Wall 2.56 inches

NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

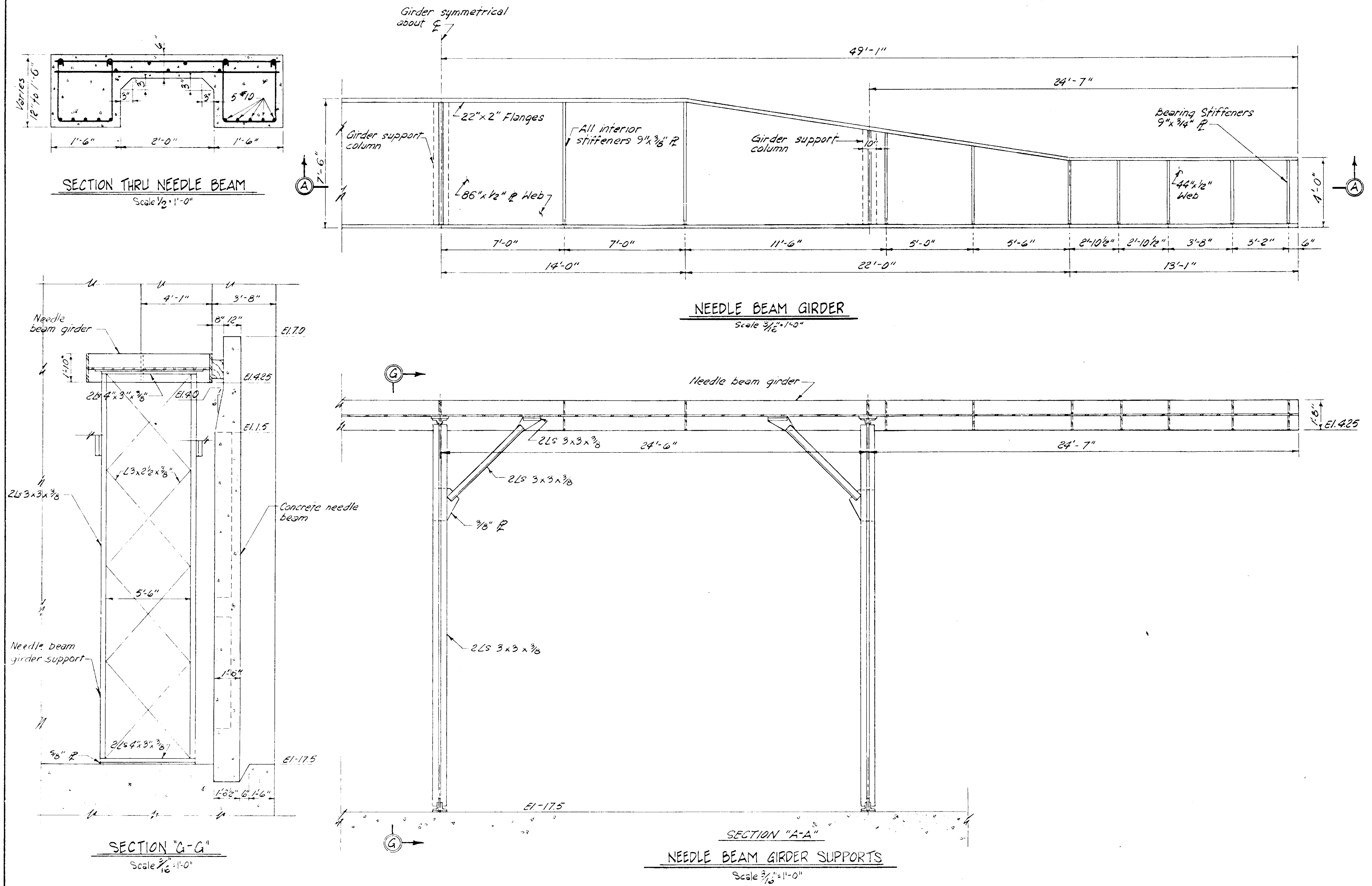
I-WALL DESIGN ANALYSIS

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

OCTOBER 1970

FILE NO. H-2-25048

PLATE IV-16



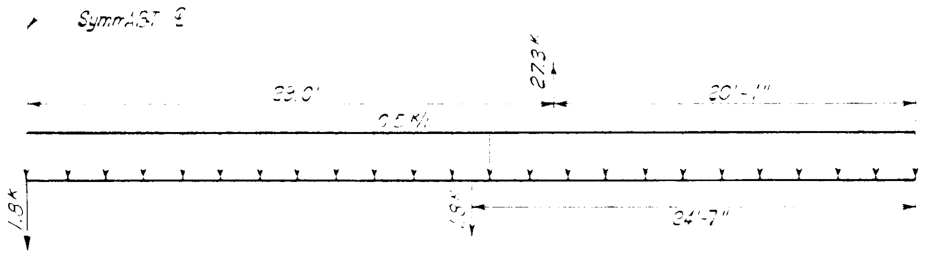
Notes:
Elevations are expressed in feet and refer to mean sea level.
Drill drain holes in web between all stiffners.
Timber bolted to needle girder flange not shown.
Details & design for 87'-2" needle girder on protected side will be similar the girder shown here.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH B I – TROPICAL BEND TO FORT JACKSON

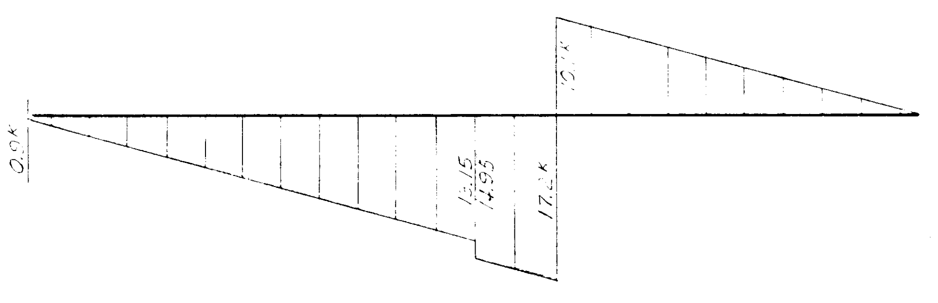
EMPIRE FLOODGATE
NEEDLE BEAM GIRDER

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

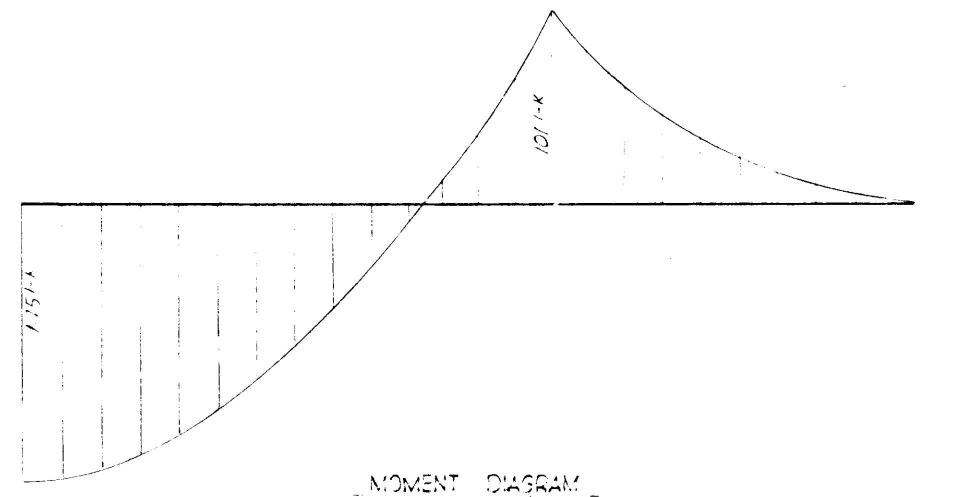
OCTOBER 1970 FILE NO. H-2-25048



LIFTING LOAD DIAGRAM
Scale: 1" = 2 k/l

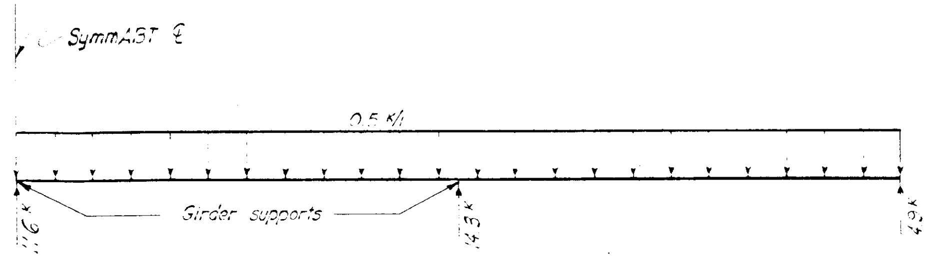


SHEAR DIAGRAM
Scale: 1" 20^k

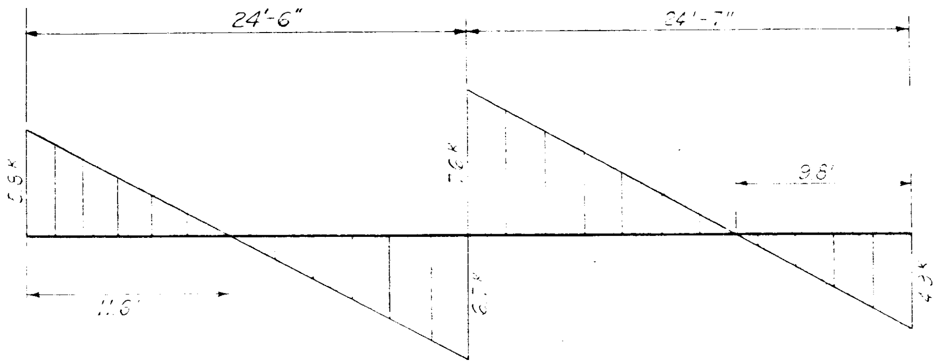


MOMENT DIAGRAM
Scale: 1" 100'-k

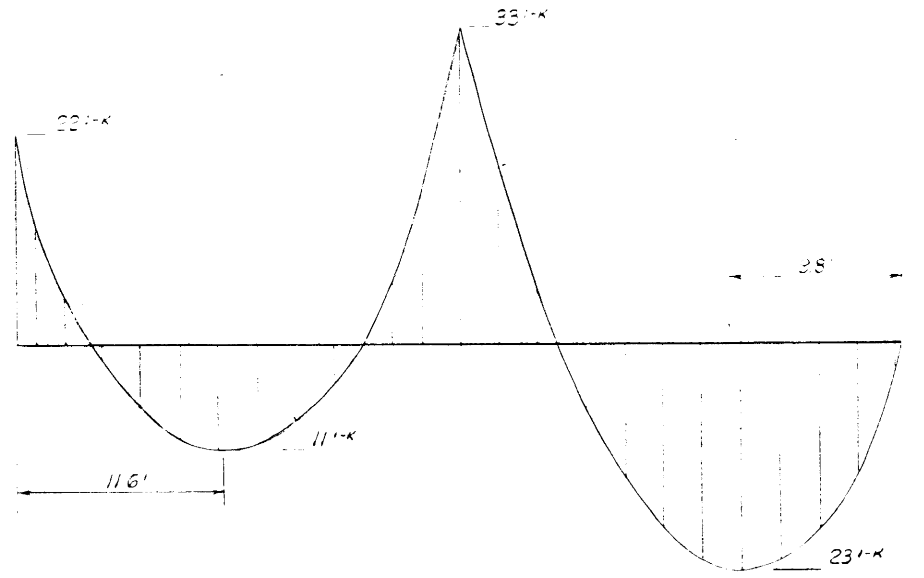
LIFTING LOAD ON GIRDER



DEAD LOAD DIAGRAM
Scale: 1" 2 k/l

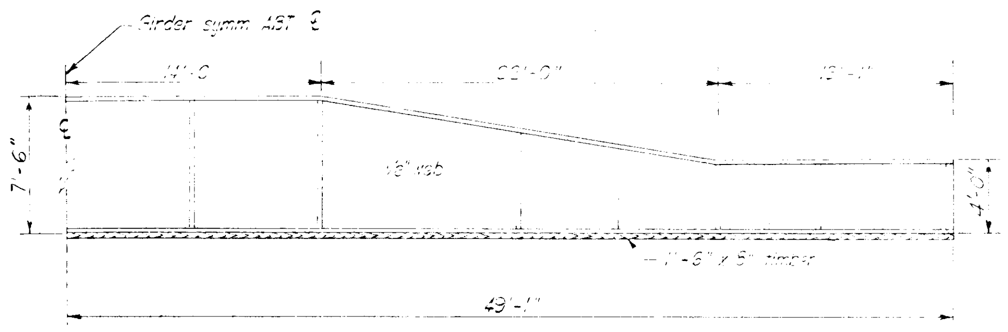


SHEAR DIAGRAM
Scale: 1" = 10^k

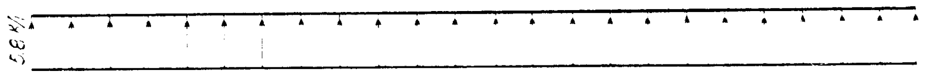


MOMENT DIAGRAM
Scale: 1" 20'-k

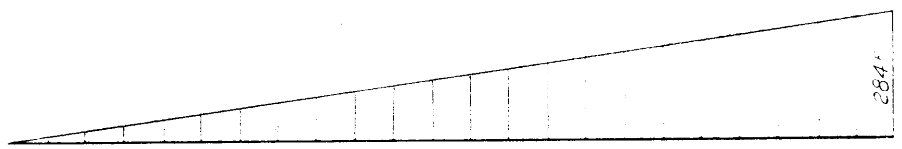
DEAD LOAD OF GIRDER



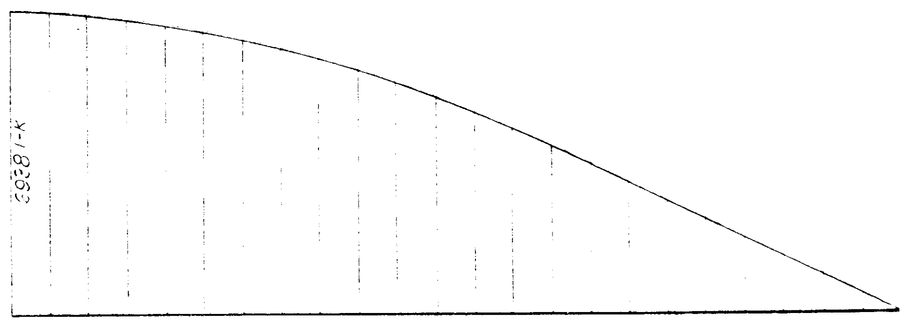
PLAN
Scale: 3/32" = 1'-0"



WATER LOADING DIAGRAM
Scale: 1" = 20 k/l



SHEAR DIAGRAM
Scale: 1" 400^k



MOMENT DIAGRAM
Scale: 1" = 4,000'-k

WATER LOAD

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
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EMPIRE FLOODGATE
NEEDLE BEAM GIRDER ANALYSIS

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

OCTOBER 1970 FILE NO. H-2-25048

PLATE IV 18

SECTION V - MECHANICAL EQUIPMENT

1. General. The gate will be operated by two identical sets of machinery located on the tops of the walls. Each set of machinery will be comprised of a motor-powered chain hoist and a free-wheeling counterweight system. Each chain hoist will consist of an electric motor with rear mounted electric brake, a mechanical load brake, and a speed reducer. A wildcat sheave will be keyed to the extended output shaft of the speed reducer and will engage die lock chain attached to the gate. Each counterweight system will consist of a 40,000 lb. weight attached to the gate by die lock chain passing over free-wheeling wildcat sheaves. Other items of mechanical equipment are gate locking devices, gate shock absorbers, compressed air system which relieves the suction under the gate, and ratchet jacks used to dog-off the counterweight and relieve the tension on the chain when the gate is in the open position. The general arrangement of the machinery is shown on plate V-1.

2. Description.

a. Electric motors. The motors will be a 3-phase, 440 volt, 60 Hz, induction type.

b. Electric brake. The electric brake will be the motor mounted, magnetic disc type.

c. Torque limiting couplings. The couplings between the motors and the mechancial load brakes will be the torque limiting type and will be calibrated to slip at 105 percent of full load torque to provide synchronization of machinery.

d. Mechanical load brakes. The load brakes will hold the gate in any position when the motor is off and will also provide a primary gear reduction.

e. Speed reducers. The speed reducers will be helical type, parallel shaft, quadruple reduction units.

f. Hoist wildcats. The hoist wildcats will be the five whelp type for 2 1/8" die lock chain. (See plate V-3.)

g. Outboard bearings. The outboard bearings for the speed reducers will be bronze bushed pillow blocks for 10"Ø shafts.

h. Hoist chains. The hoist chains will be 2 1/8" Ø die lock, stud link type. (See plate V-3.)

i. Counterweights. Each 40,000 lb. counterweight will consist of lead weights encased in a steel cage.

j. Counterweight wildcats. The counterweight wildcats will be the five whelp type for 1 1/4" Ø die lock chain. (See plate V-4.)

k. Bearings for counterweight wildcats. The bearings will be bronze bushed pillow blocks for a 7" Ø shaft.

l. Counterweight chains. The counterweight chains will be 1 1/4" Ø die lock, stud link type. (See plate V-4.)

m. Locking devices. The gate locking devices will be the package-type electric motor operated cylinders.

n. Shock absorbers. The shock absorbers will be the hydraulic type with spring return.

o. Air compressor. The air compressor will be the two stage type, mounted on a common base without a receiver. It will be located in the control house.

p. Ratchet jacks. Two ratchet jacks will be provided for each counterweight. One end of the jacks will attach to the beams of the wildcat support and the other end will attach to the counterweight.

3. Computations. The basic assumptions, design criteria, allowable stresses, and computations for the machinery are contained in figure V-1 through figure V-25.

MECHANICAL DESIGN DATA

1. General. The hoisting operation and the functions of the counterweight system are illustrated by the kinematic diagram shown on Plate V-2. The counterweights aid the hoist machinery in closing the gate from 0°-48°. The counterweights oppose the hoist machinery in closing the gate from 48°-90°. During this portion of the closing cycle, the counterweights prevent wave action from slamming the gate and causing large stress variations in the hoist chain. The counterweights also provide a positive force on the gate for opening against small differential heads.

2. Hoist Load. The following assumptions were made in determining the maximum hoist load:

   a. A 4 ft. layer of silt will cover the

gate. This will be caused by the 4 ft. difference in elevation between the structure sill and the channel.

b. This layer of silt will remain intact from 0°-30°.

c. The layer of silt will fail and slide off the gate at the 30° position.

d. The compressed air will eliminate all suction.

e. No buoyancy will be present from the air pocket.

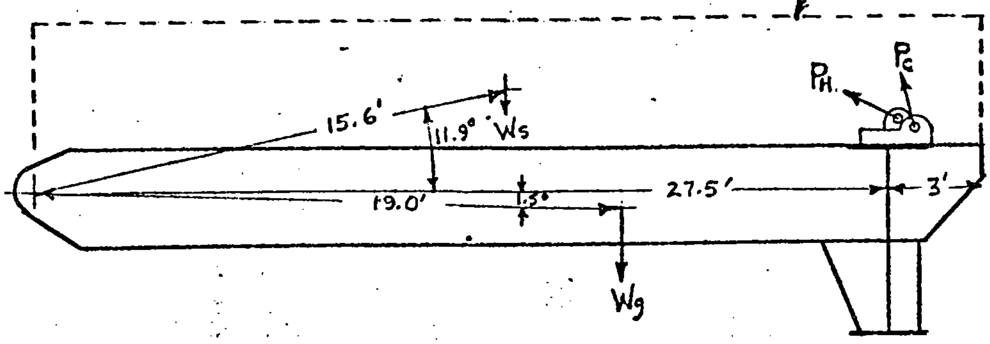
f. No wind pressure will be exerted on the gate.

g. No wave pressure will be exerted on the gate.

h. No differential head will exist.

Hoisting loads were calculated for gate closure angles at 10° intervals, and curves are presented on Fig V-4a for various silting conditions. A sample computation for the maximum load, which is used as a basis for machinery design, is presented as follows:

Fig. V-2



W_g = weight of gate, submerged = 165^K

W_s = weight of silt, submerged = volume x density
$= (30.5')(84.5')(4.00')(37.0\ lb/ft^3) = 381^K$

P_c = pull on both cwt. chains = 80^K

P_H = pull on both hoist chains

Determine P_H by summing moments about the hinges:

$$\Sigma \overset{+}{\curvearrowright} M_H = 0 = (\text{gate wt.})(\text{lever arm}) + (\text{silt wt.})(\text{lever arm}) - (\text{cwt. pull})(\text{lever arm}) - (\text{hoist pull})(\text{lever arm})$$

$$\Sigma M_H = 0 = (165^K)(19.0')(\cos 1.5°) + (381^K)(15.6')(\cos 11.9°) - (80^K)(28.8')(\sin 83.0°) - (P_H)(28.3')(\sin 46.8°)$$

$$P_H = \frac{6670 \text{ ft-kip}}{20.6 \text{ ft}} = 324^K$$

$\frac{1}{2} P_H = 162^K$ (for one chain)

The lever arms for the hoist and cwt. chain were determined from the kinematic diagram on Plate V-2. The friction

Fig. V-3

moment from the hinges was neglected and is shown to be negligible as follows:

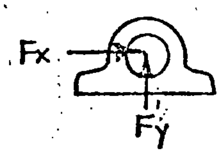
$$\text{Friction Moment} = R\,r\,\mu$$

where: R = resultant force on hinges

r = pin radius = .292'

μ = coefficient of friction = .10

$$R = \sqrt{F_y^2 + F_x^2}$$



$$F_y = 381^K + 165^K - (80^K)(\sin 83.0°) - (324^K)(\sin 46.8°)$$

$$F_y = 231^K$$

$$F_x = (80^K)(\cos 83.0°) + (324^K)(\cos 46.8°)$$

$$F_x = 232^K$$

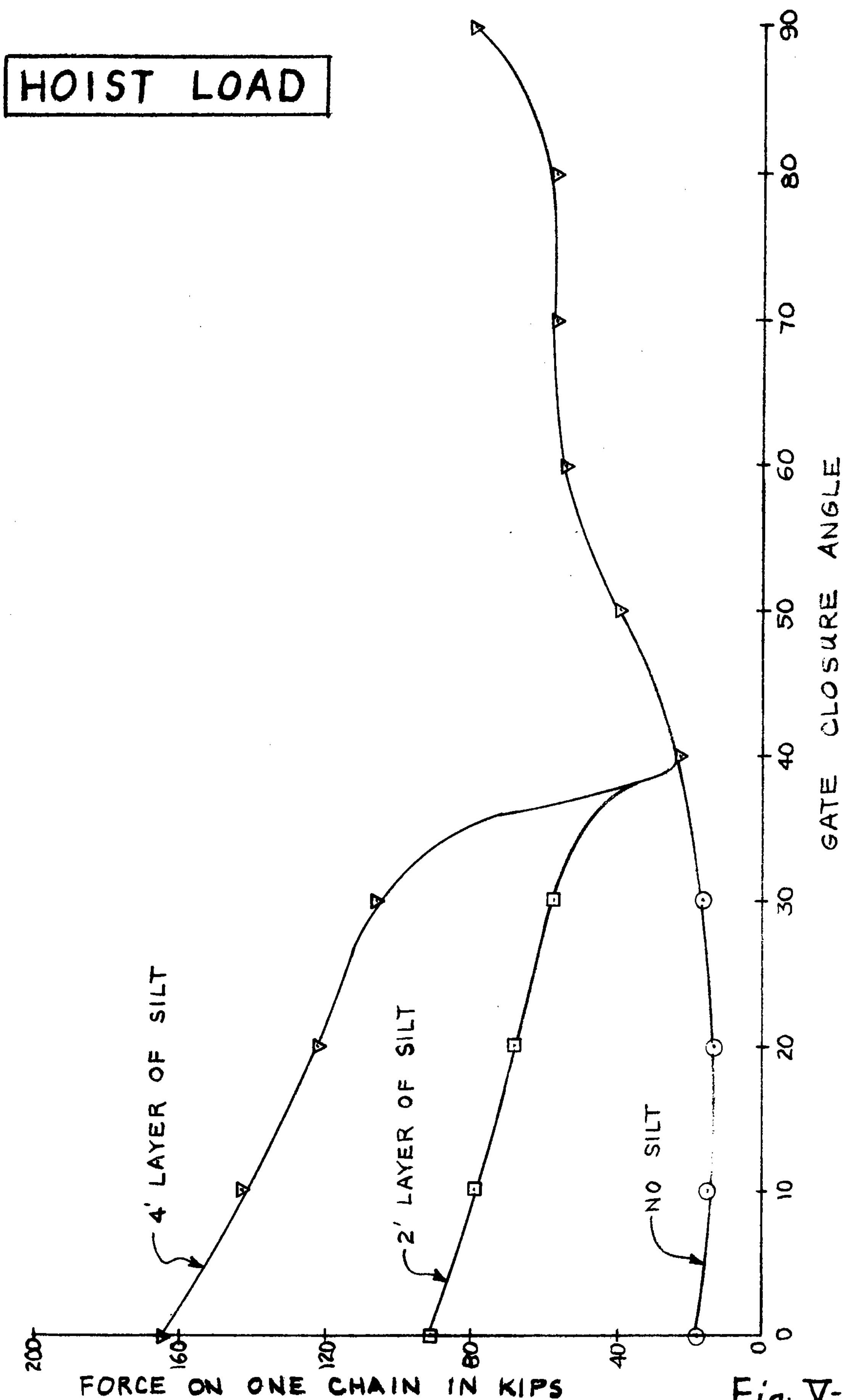
$$R = \sqrt{(231)^2 + (232)^2} = 328^K$$

$$\text{Friction Moment} = (328)(.292)(.1) = 9.6 \text{ ft-kip}$$

Comparison of this moment with the moments obtained in the hoist calculations show that the hinge friction is negligible.

Fig. V-4a shows a plot of hoist load vs. gate closure angle. The hoist load is also shown for 2' of silt and

Fig. V-4



HOIST LOAD
4' LAYER OF SILT
2' LAYER OF SILT
NO SILT
GATE CLOSURE ANGLE
FORCE ON ONE CHAIN IN KIPS
0
10
20
30
40
50
60
70
80
90
200
160
120
80
40
0

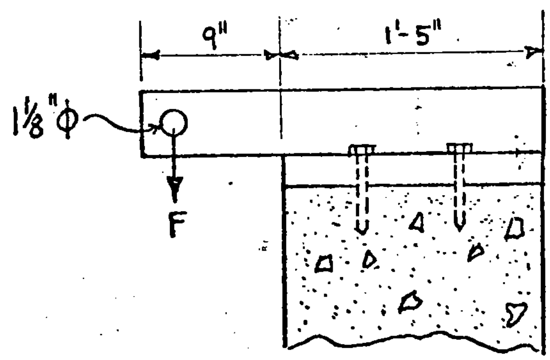
Fig: V-4a

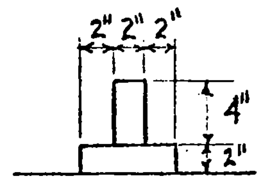
for no silt. The maximum hoist load obviously occurs while hoisting the gate from the fully open position with 4' of silt.

3. Hoist Chain. Each hoist chain will have a maximum load of 162 kips. This maximum load will be of infrequent occurrence, since the normal silt condition should be much less than 4 ft. Thus a factor of safety of 3 is considered adequate for the maximum load. The 2 1/8" die lock chain has a breaking strength of 548,000#. For maximum loading the factor of safety is $F.S. = \frac{548,000}{162,000} = 3.4$. With no silt this chain has a factor of safety of $F.S. = \frac{548,000}{78,000} = 7.0$. The factor of safety for the chain connections to the gate (shown on Plate V-5) exceeds the factor of safety for the chain.

A wildcat sheave on the hoist machinery will engage the die lock chain on the gate. The wildcat will be the five pocket type as shown on Plate V-3.

The free end of the chain in the recess will be anchored as shown below.





F = dead weight of 23' of chain ≃ 1000#

Bearing on Pin Hole: (Allowable = 24,000 psi)

$$S_B = \frac{\text{Load}}{\text{Area}} = \frac{1000\#}{2'' \times 1''} = 500 \text{ psi}$$

Shear on Cross Section: (Allowable = 11,000 psi)

$$S_S = \frac{\text{Load}}{\text{Area}} = \frac{1000\#}{4'' \times 2''} = 125 \text{ psi}$$

Flexure on Cantilever: (Allowable = 18,000 psi)

$$S_f = \frac{M}{Z} = \frac{(1000\#)(9'')}{(2'')(4'')^2/6} = 1{,}690 \text{ psi}$$

Tension on Anchor Bolts: (Allowable = 10,500 psi)

(Assuming the anchorage pivots about the edge of the concrete, the maximum tension is calculated to be 250#. Use 1/2" ϕ anchor bolts.)

$$S_T = \frac{\text{Load}}{\text{Area}} = \frac{250^{\#}}{\pi(.2'')^2} = 2{,}000 \text{ psi}$$

The chain will be connected to the anchorage by a 1" ϕ round pin shackle and an anchor connecting link.

4. <u>Hoist Machinery</u>. The general layout of the hoist machinery is shown on Plate V-1. The individual components were selected as follows:

a. Operating Time. A time of one hour is considered adequate to effect complete closure. From the kinematic diagram it was determined that approximately 40 ft. of chain will be hoisted during a complete closure. A mechanical load brake incorporating a single ratio of reduction of 10:1 and a quadruple reduction, parallel shaft speed reducer with a total reduction ratio of 1480:1 driven by a 1750 rpm electric motor were selected. These units were selected based on commercially available products with standard ratios necessary to provide the appropriate operating time. Based on a pitch radius for the hoist wildcat of 13.8", the operating time will be:

$$T = \frac{(\text{length of chain})(\text{reduction ratio})}{(2\pi)(\text{pitch radius})(\text{input rpm})}$$

$$T = \frac{(40 \times 12 \text{ in.})(1480 \times 10)}{(2\pi)(13.8 \text{ in.})(1750 \text{ rpm})} = 47 \text{ min}$$

b. Speed Reducer. A speed reducer similar and equal to Philadelphia Gear Corp. helical unit #427 with a quadruple reduction of 1480:1 will meet our requirements as follows:

$$\text{Output torque} = (\text{load})(\text{pitch radius})$$
$$= (162{,}000\#)(13.8")$$
$$= 2{,}240{,}000 \text{ in-lb}$$

Fig. V-8

$$\text{Input HP} = \frac{(\text{input torque})(\text{input rpm})}{(63{,}000)(\text{efficiencies})}$$

$$= \frac{\left(\frac{2{,}240{,}000}{1480} \text{ in-lb}\right)\left(\frac{1750}{10} \text{ rpm}\right)}{(63{,}000)(.95 \times .90)}$$

$$= 4.9 \text{ HP}$$

The rating of the #427 unit based on a service factor of 0.7 (uniform load with 2 hrs./day operation) is as follows:

Output torque = 2,880,000 in-lb

Input HP = 5.6

The outboard bearing will reduce the overhung load on the speed reducer to approximately 81,000#. Special bearings can be provided in the speed reducer to handle this overhung load.

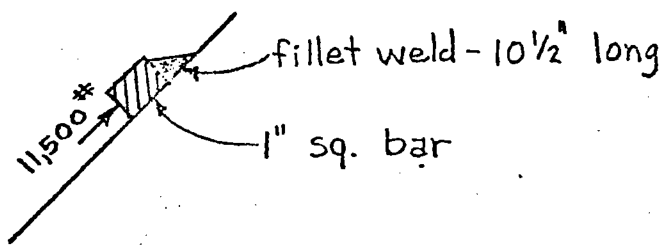
c. Outboard Bearing. The outboard bearing will be mounted on a 45° angle because of the large angular load. The bearing will be a bronze bushed pillow block for a 10" Φ shaft. The pillow block will have

160 sq. in. of projected bearing surface giving a unit load of:

$$S = \frac{\text{Load}}{\text{Area}} = \frac{81{,}000}{160} = 506 \text{ psi}$$

(max. allowable for SAE 660 bronze = 4000 psi)

Since the pillow block will have slotted bolt holes, "stops" will have to be welded to the base plate to prevent movement and absorb the shear load. A maximum shear load of 11,500 lb. occurs at a 30° gate angle.



$$S_s = \frac{\text{Load}}{\text{Area}} = \frac{11{,}500}{(.707)(10.5)(\text{leg})} = 10{,}000 \text{ (allowable)}$$

$$\text{leg} = .155''$$

Use a ¼" fillet weld. The shear loads act in both directions on the pillow block, so stops will be welded on both ends.

Fig. V-10

d. Mechanical Load Brake. The purpose of the mechanical load brake is to hold the gate in any position when the motor is off. This brake is required in addition to the electric brake to assure maximum safety and accuracy of control. A mechanical load brake similar and equal to Shepard Niles Form 38, Class A will meet our requirements. This brake is rated 7.5 HP at 1750 RPM and is available with a 10:1 reduction ratio.

e. Motor. The motor will be induction type, 3ϕ, 440 volt, 60 Hz, with motor mounted magnetic disc brake. The horsepower requirement is as follows:

$$HP = \frac{(\text{input torque})(\text{input rpm})}{(63{,}000)(\text{efficiencies})}$$

$$= \frac{\left(\frac{2{,}240{,}000}{1480 \times 10}\right)(1750)}{(63{,}000)(.95 \times .9 \times .95)}$$

$$= 5.2 \text{ HP}$$

Fig. V-11

A 7.5 HP motor will be used.

f. Couplings. The coupling between the speed reducer and the mechanical load brake will be a flexible coupling. The coupling between the mechanical load brake and the motor will be a torque limiting coupling set to slip at 105% of full load torque.

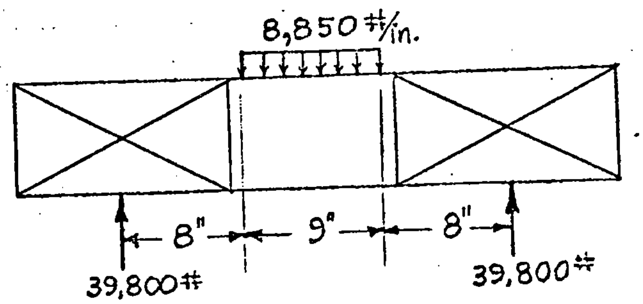
5. Counterweight Chain. The 1¼" ϕ die lock chain has a breaking strength of 198,000#. Each counterweight chain will have a working load of 40,000#. The factor of safety for the counterweight chain is $F.S. = \frac{198,000}{40,000} = 5.0$. The factor of safety for the chain connections exceeds the factor of safety for the chain. The gate connection is shown on Plate V-5, and the counterweight connection is

similar.

The counterweight chains will pass over five-pocket wildcat sheaves (see Plate V-4). Each wildcat will be keyed to a shaft which rotates in pillow blocks. All shafts and pillow blocks in the counterweight system will be the same size for uniformity.

The wildcat at the edge of the wall is subjected to the greatest load. The resultant force diagram is shown on Fig. V-13a. All shafts and pillow blocks were designed according to this loading. The shaft size was determined as follows:

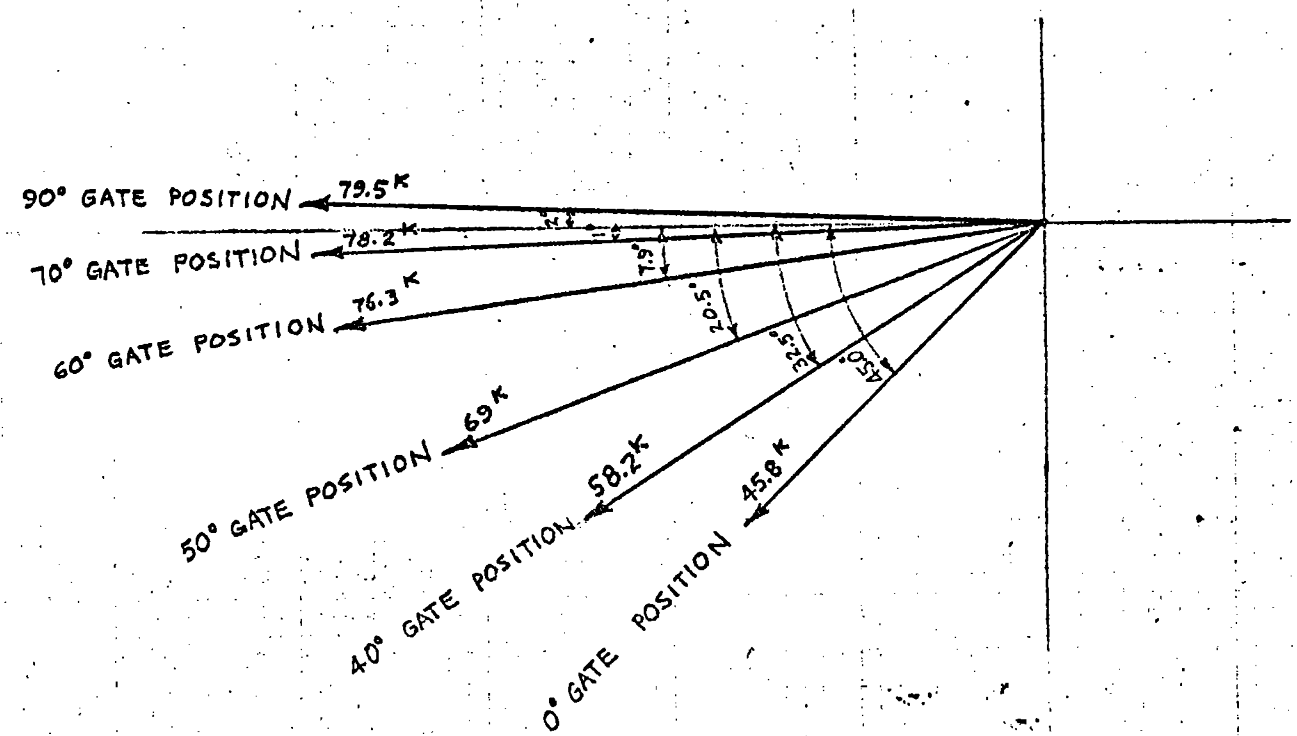
(max. load = 79,500#)



$$M_{max} = 39{,}800\left[8 + \frac{39{,}800}{(2)(8{,}850)}\right] = 408{,}000 \text{ in-lb}$$

$$S_f = \frac{M_{max}}{Z} = \frac{408{,}000}{Z}$$

Fig. V-13



RESULTANT FORCE ON
FREEWHEELING WILDCAT
AT WALLS EDGE

Fig. V-13a

The allowable flexural stress is 12,000 psi for ASTM A306 steel (grade 65) with allowance for keyways.

$$12{,}000 \geq \frac{408{,}000}{Z}$$

$$Z \geq 34.0$$

$$\frac{\pi d^3}{32} \geq 34.0$$

$$d \geq 7.0''$$

A 7" φ shaft with spacers between the wildcat and pillow block will be used.

The pillow blocks will be the angular joint, bronze bushed type for a 7" φ shaft. They will have 98 sq. in. of projected bearing surface, giving a unit load of:

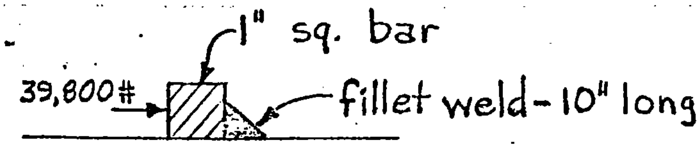
$$S = \frac{\text{Load}}{\text{Area}} = \frac{39{,}800}{98} = 406 \text{ psi}$$

(max. allowable for SAE 660 bronze = 4000 psi)

Since the pillow block will have slotted bolt holes, a "stop" will be welded to the base plate to prevent movement and absorb the shear load. The

Fig. V-14

maximum shear load is 39,800 #.



$$S_s = \frac{\text{Load}}{\text{Area}} = \frac{39{,}800}{(.707)(10)\ \text{leg}} = 10{,}000 \ (\text{allowable})$$

$$\text{leg} = .56''$$

Use a 5/8" fillet weld.

6. <u>Shock Absorber</u>. The hydraulic shock absorber will be clevis mounted in the location shown on Plate V-1. It will engage the gate as shown on Plate V-5.

Wave impact calculations show that the gate will move a small amount under the following conditions:

a. The gate in a nearly closed position
b. Water el. +5 (70 mph wind)
c. Hurricane path directly up the waterway
d. 10% design wave hits gate

The probability of these conditions occuring is very slight. However, a shock absorber will be provided to resist possible slamming as follows:

$$T.E. = .1865\, W k^2 \omega^2 + T\theta$$

where T.E. = total energy, in-lb

W = weight of gate, lb

k = radius of gyration, ft

ω = angular velocity of gate, $\frac{\text{rad}}{\text{sec}}$

T = total propelling torque, in-lb

θ = stopping angle, rad

The quantity Wk^2 for the gate was found to be 77,900,000 lb-ft². Wave impact calculations for the aforementioned conditions show that the gate would move approximately 1 ft. at the top at a velocity of .083 $\frac{\text{rad}}{\text{sec}}$. The propelling torque is the sum of

Fig. V-16

the positive torque due to the wave load, the positive torque due to wind, the negative torque due to the inertia of water in front of the gate, the negative torque due to the inertia of the gate, and the negative torque due to the counterweight. The total propelling torque was calculated to be 1,550,000 in-lb.

$$T.E. = (.1865)(77{,}900{,}000)(.083)^2 + (1{,}550{,}000)\left(\frac{1}{30.5}\right)$$

$$T.E. = 151{,}000 \text{ in-lb}$$

$$\tfrac{1}{2}\, T.E. = 75{,}500 \text{ in-lb (for one shock absorber)}$$

A Hanna S Series shock absorber, or equal, with a 2" bore and 8" stroke is within the desired capacity and will fit the available space.

Fig. V-17

7. Locking Device. The location of the locking device is shown on Plate V-1. It will be designed to hold the gate with the counterweight attached, the hoist detached, and water elevations of +6.3 on the protected side and -2.0 on the flood side.

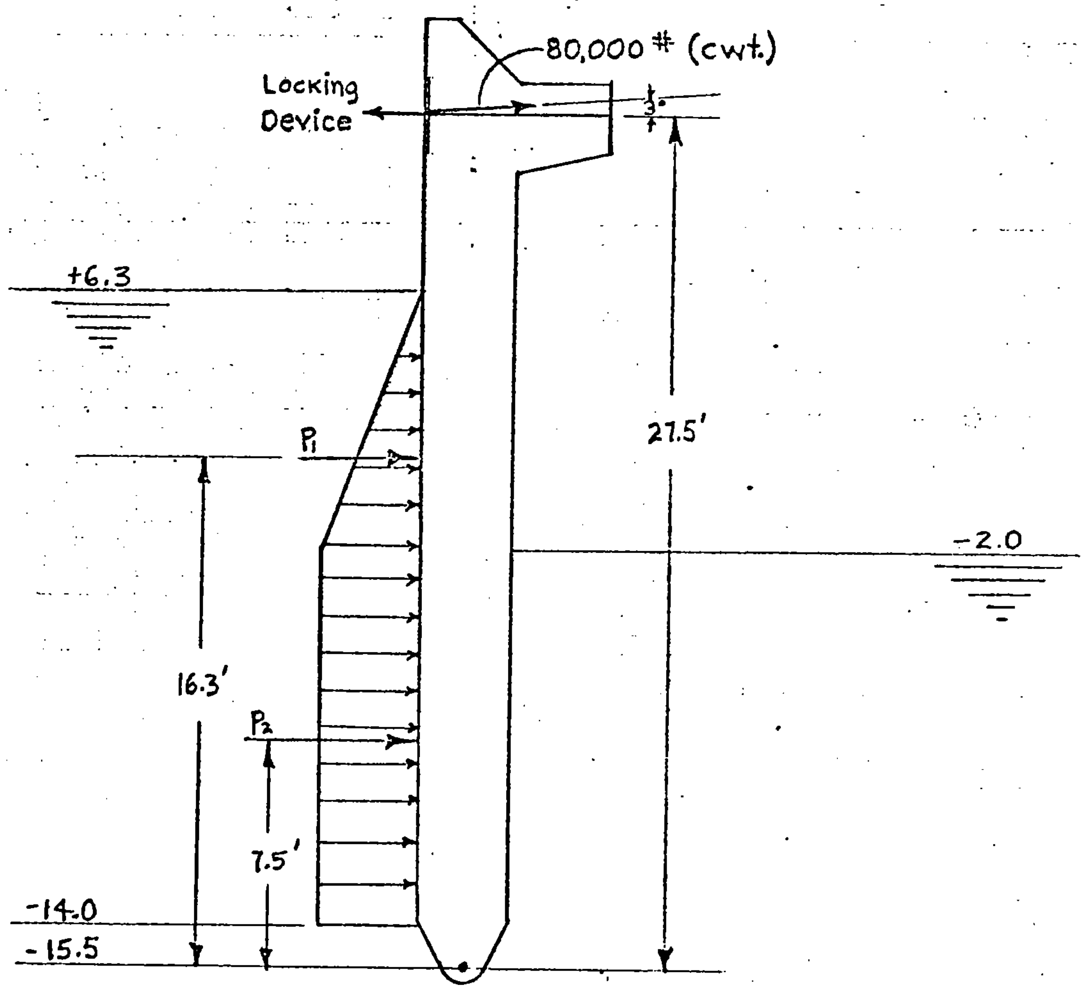


Fig. V-18

$$P_1 = (62.4\ \#/ft^3)(8.3\ ft)^2(84\ ft)(.5) = 181{,}000\ \#$$

$$P_2 = (62.4\ \#/ft^3)(8.3\ ft)(12\ ft)(84\ ft) = 522{,}000\ \#$$

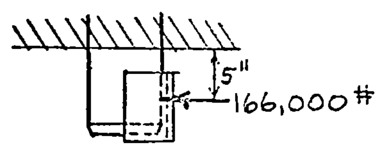
Sum moments about the hinge:

$$\Sigma \overset{\curvearrowright}{M}_H = 0 = (80{,}000\ \#)(27.5\ ft)(\sin 87^\circ) + (181{,}000\#)(16.3\ ft.)$$
$$+ (522{,}000\ \#)(7.5\ ft.) - (F)(27.5\ ft.)$$
$$F = 331{,}000\ \#$$

(for one locking device) $\frac{1}{2}F = 166{,}000\ \#$

A round pin attached to the push-pull rod of the electric cylinder will pass through a sleeve embedded in the concrete and will engage a sleeve attached to the gate girder.

Flexure on Pin: (Allowable = 18,000 psi)



$$M = (166{,}000\ \#)(5") = 830{,}000\ \text{in-lb}$$

$$S_f = \frac{M}{Z} = \frac{830{,}000\ \text{in-lb}}{\pi d^3/32} \leq 18{,}000\ \text{psi}$$

$$d \geq 7.77"$$

Use an 8" ϕ round pin

Shear on Pin: (Allowable = 11,000 psi)

$$S_s = \frac{\text{Load}}{\text{Area}} = \frac{166{,}000\ \#}{\pi\ (4")^2} = 3{,}300\ \text{psi}$$

Fig. V-19

Bearing on Gate Sleeve: (Allowable = 24,000 psi)

Use 4" long piece of 9½" OD steel tube

$$S_B = \frac{Load}{Area} = \frac{166{,}000\ \#}{(8")(4")} = 5{,}200 \text{ psi}$$

Bearing on Sleeve in Concrete:

(Allowable = 24,000 psi)

Use 7" long piece of 9½" OD steel tube

$$S_B = \frac{Load}{Area} = \frac{166{,}000\ \#}{(8")(7")} = 3{,}000 \text{ psi}$$

The electric cylinder will be similar and equal to "Raco" Electrical Ram, type ERO with a 7½" stroke.

8. Air Compressor.

a. Purpose. The purpose of the compressed air is to relieve the suction created when the gate is lifted in silt. A secondary function is the buoyancy caused by the air pocket formed under the gate. However, this buoyancy was not considered in sizing the hoist machinery.

b. Relieving Suction. The compressed air will exit from the piping network

Fig. V-20

... become trapped under the gate. Holes will be drilled in the webs of the girders to limit the air pocket to 8". The trapped air will displace water out the sides and end of the gate. This circulating action will relieve suction during the hoisting operation.

c. Buoyancy. Although the buoyancy was not considered in sizing the hoist machinery, a large amount of buoyant force will be available. The maximum buoyancy available (8" air pocket) is determined as follows:

$$\text{Buoyancy} = \text{volume of air pocket} \times \text{density}$$
$$= (29.0')(83')(8/12')(62.4\ \#/ft^3)$$
$$= 101{,}000\ \#$$

The buoyancy required to lift the gate with no silt is determined as follows:

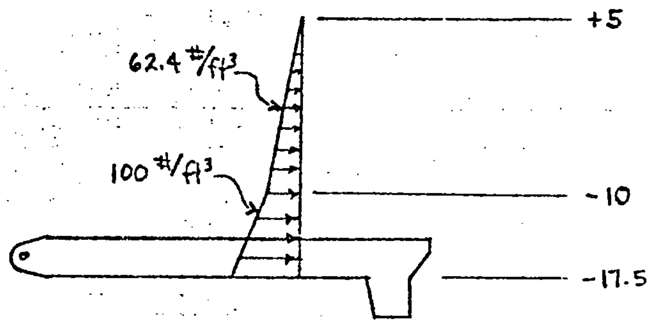
$$\Sigma \overset{\curvearrowright}{M}_H = 0 = (\text{gate wt.})(\text{lever arm}) - (\text{cwt.})(\text{lever arm}) - (\text{buoyancy})(\text{lever arm})$$

$$0 = (165{,}000\#)(19.0)(\cos 1.5°) - (80{,}000\#)(28.8')(\sin 83°) - B(16)$$

$$B = 53{,}200\#$$

Thus enough buoyant force is available to lift the gate without using the hoist. If this happens, the gate will rise until enough air escapes to establish an equilibrium position. The hoist chain will be slack until the hoist machinery "catches up" to hoist it the remaining distance.

d. Capacity of Compressor. The air pressure must be at least equal to the water and silt pressure:



The water and silt pressure at el. -17.5 is:

$$P = (62.4\ \#/ft^3)(15\ ft) + (100\ \#/ft^3)(7.5\ ft.)$$

$$P = 1685\ psf = 11.7\ psi$$

A 7½ HP, 43 cfm, 100 psi compressor will be sufficient for this application. A receiver

Fig. V-22

will not be necessary. The check valves in the piping network will have a cracking pressure of 1 psi; thus the exit air pressure will equal the water and silt pressure plus the cracking pressure. A maximum pressure of 100 psi could be obtained if the pipe line becomes obstructed. The compressor operating time required to achieve maximum buoyancy will be:

$$\text{time} = \frac{\text{volume of air pocket}}{\text{compressor displacement}}$$

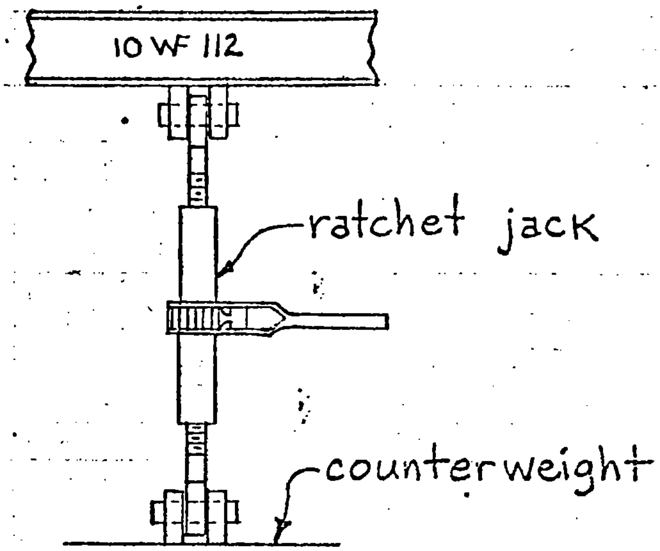
$$\text{time} = \frac{(29')(83')(8/12')}{43 \text{ cfm}} = 37 \text{ min.}$$

Most of the suction would be relieved long before this maximum buoyancy is achieved.

e. Piping Layout. The piping layout will be as shown on Plate V-6. The air compressor will be located in the control house (see Plate IV-3). The pipe from the compressor to the wall will be 1½" Φ, schedule 40, galvanized steel pipe. At the wall, 1½" Φ plastic pipe will embedded in the concrete. This pipe will be schedule 40, PVC II (high impact) pressure pipe. The header pipe in the gate recess will branch from tees into the exit pipes. Each exit pipe will consist of a 1½" male x ½" female reducing bushing connected to the tee and a ½" check valve connected to the reducing bushing. The check valve will be a corrosion proof design with a plastic housing.

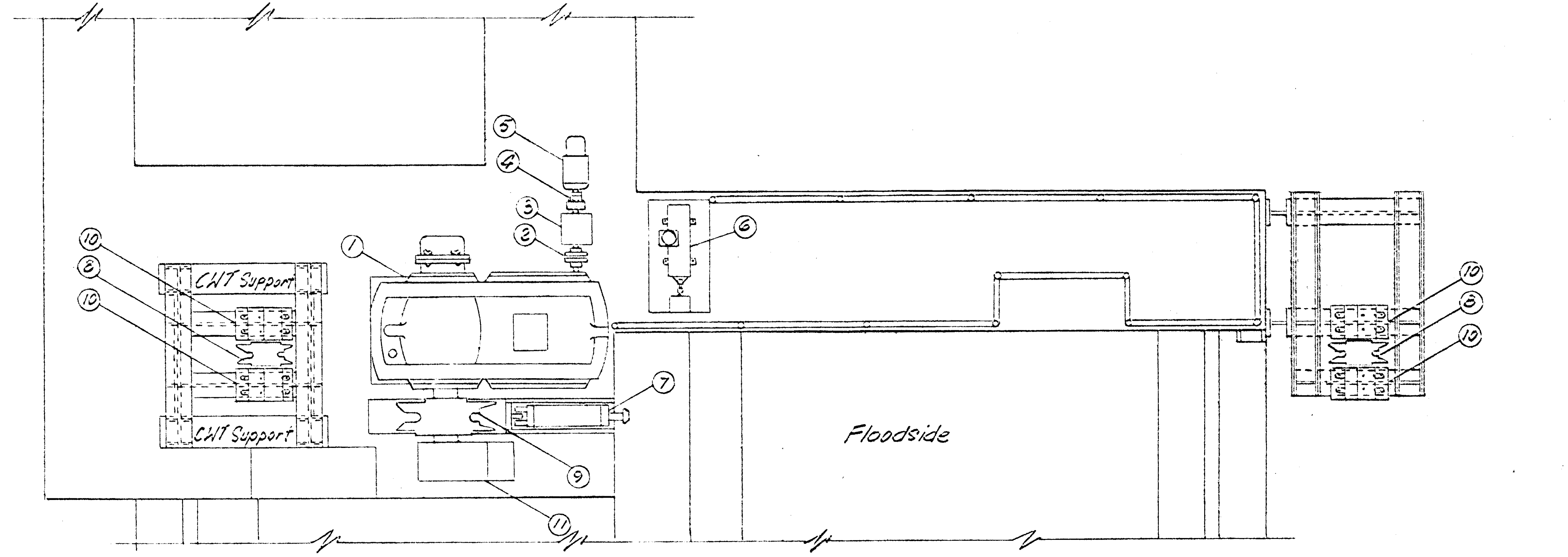
9. Ratchet Jack. The ratchet jacks will be designed to lift the counterweights to relieve the tension on the chains. An "American Forge" Ratchet Pipe Turnbuckle with a 1¾" screw diameter or equal will meet the requirements. This ratchet will require an effort of 60 # with a 24" handle. Two ratchets will be provided for each counterweight. Each ratchet will have a breaking strength

of 126,000 #. The ratchets will be attached as shown below:

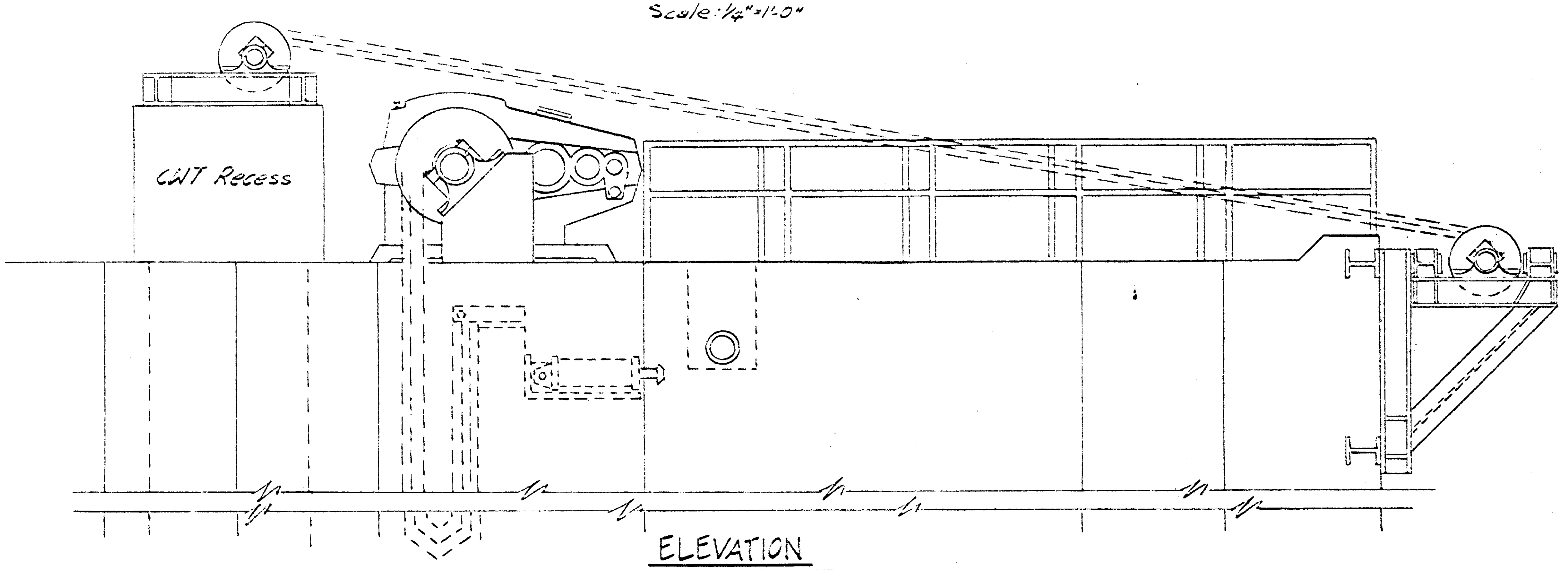


When not in use, the ratchet jacks will be stored in the control house.

Fig. V-25



PLAN
Scale: 1/4" = 1'-0"

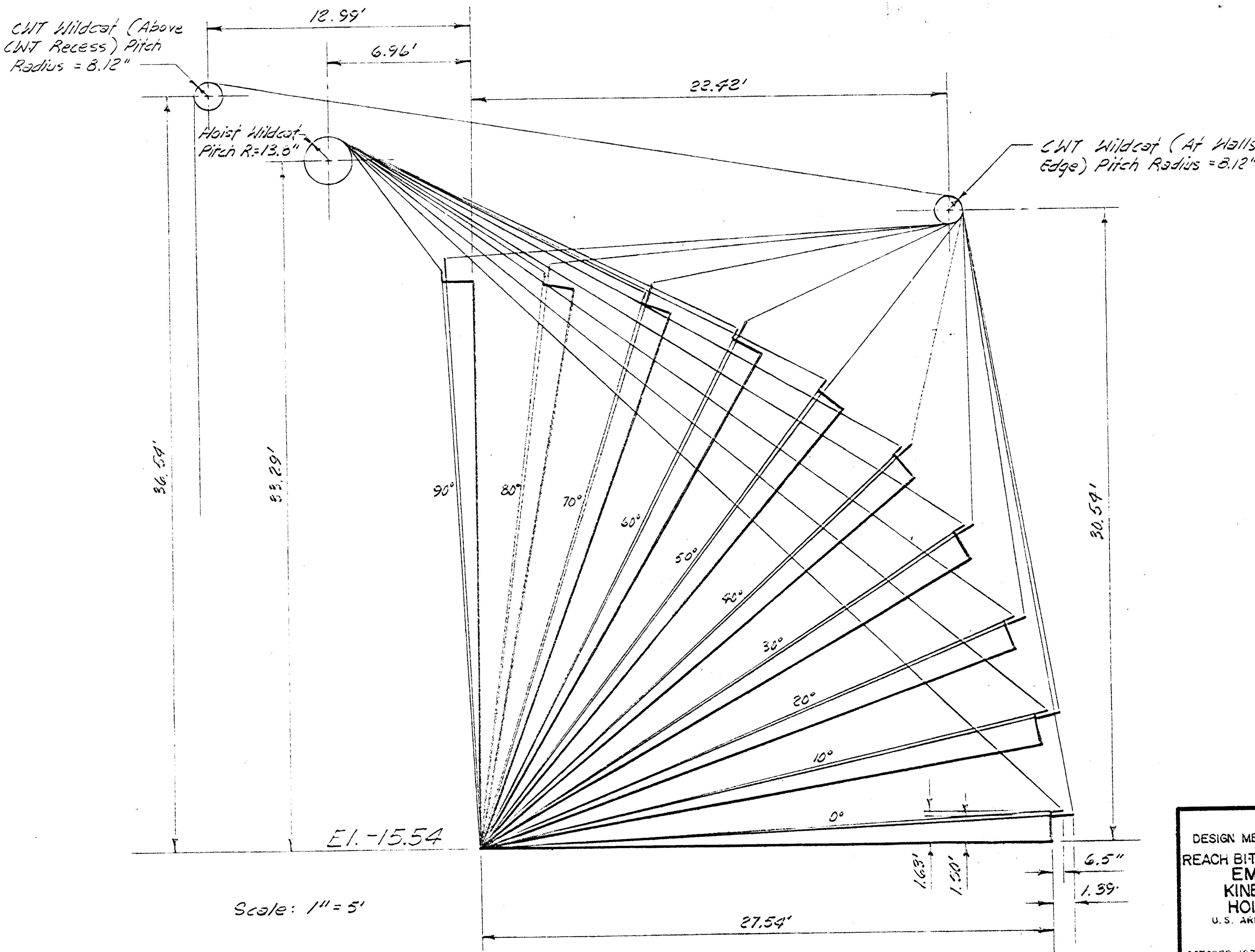


ELEVATION
Scale: 1/4" = 1'-0"

ITEM LIST		
	NAME	DESCRIPTION
1	HELICAL SPEED REDUCER	Quadruple Reduction, 1480 TO 1 Ratio, with face mounted limit swith
2	FLEXIBLE COUPLING	
3	MECHANICAL LOAD BRAKE	10 to 1 Reduction Incorporated in unit
4	TORQUE LIMITING COUPLING	Similar and equal to falk controlled torque coupling
5	ELECTRIC MOTOR	7½ HP, Induction type, 3φ, 440 volt, 60 Hz, with motor-mounted magnetic disc. brake
6	LOCKING DEVICE	Similar and equal to Raco Machine Co. Electric Ram, type ERO
7	SHOCK ABSORBER	Similar and equal to Hanna Hydraulic Shock Absorber - series H
8	COUNTERWEIGHT WILDCAT	For 1¼" Die-Lock chain
9	HOIST WILDCAT	For 2⅛" Die-Lock chain
10	PILLOW BLOCK	Angular joint, Bronze bushing, for 7" φ shaft
11	PILLOW BLOCK	Bronze bushing for 10" φ shaft

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
GENERAL LAYOUT
FOR HOIST AND CWT
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048

PLATE V-1



NEW ORLEANS TO VENICE, LA.

DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN

REACH BI-TROPICAL BEND TO FORT JACKSON

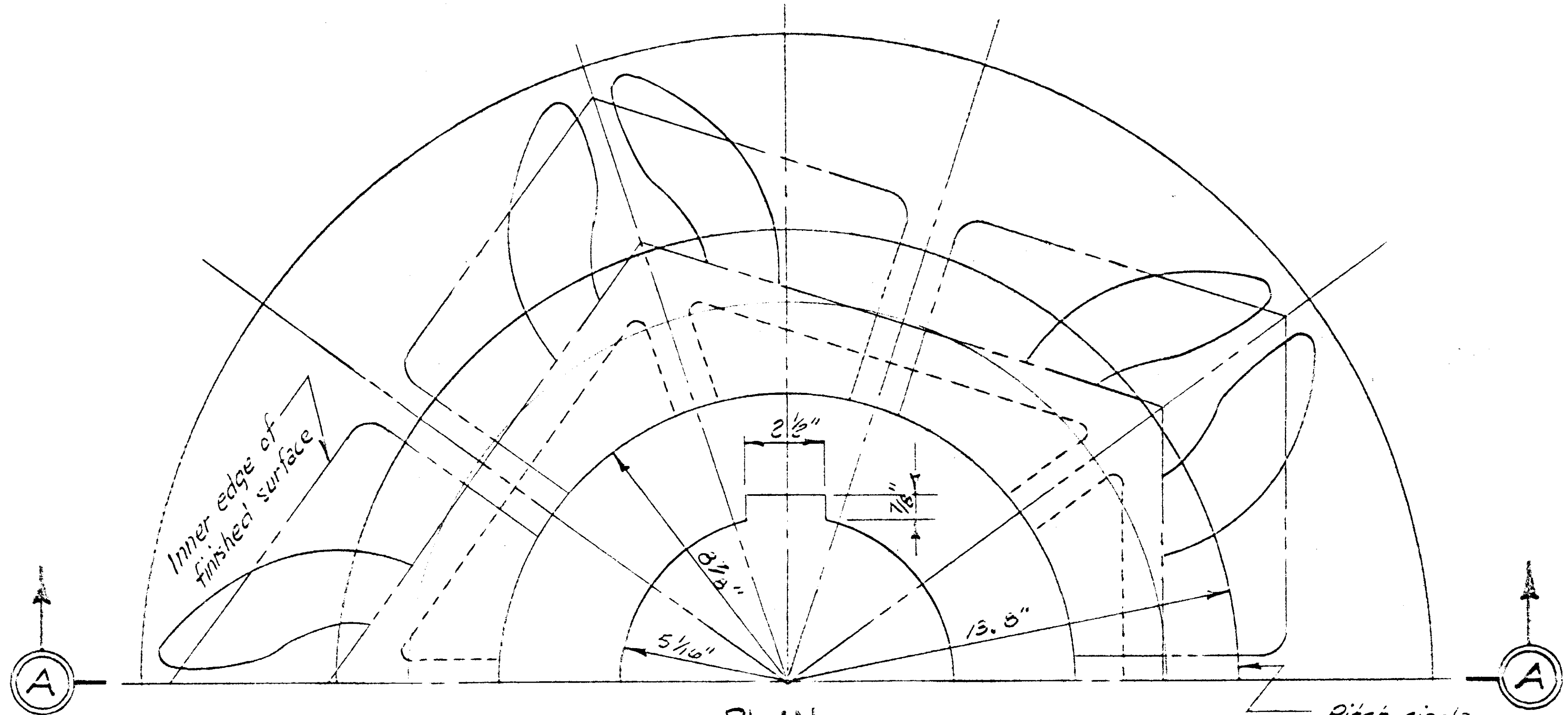
EMPIRE FLOODGATE

KINEMATIC DIAGRAM FOR HOIST AND CWT CHAINS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS

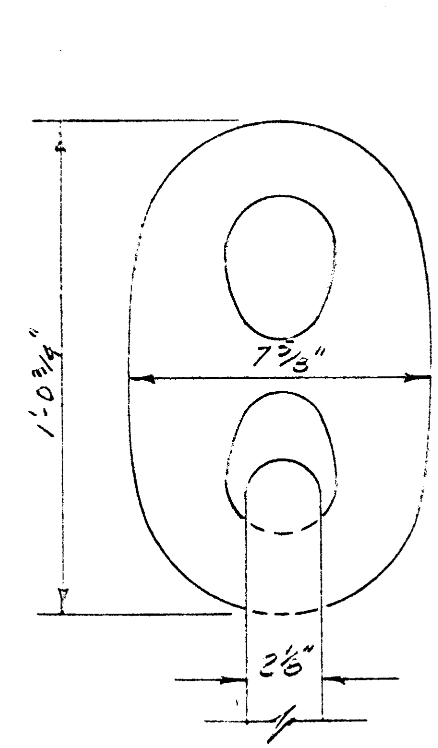
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048



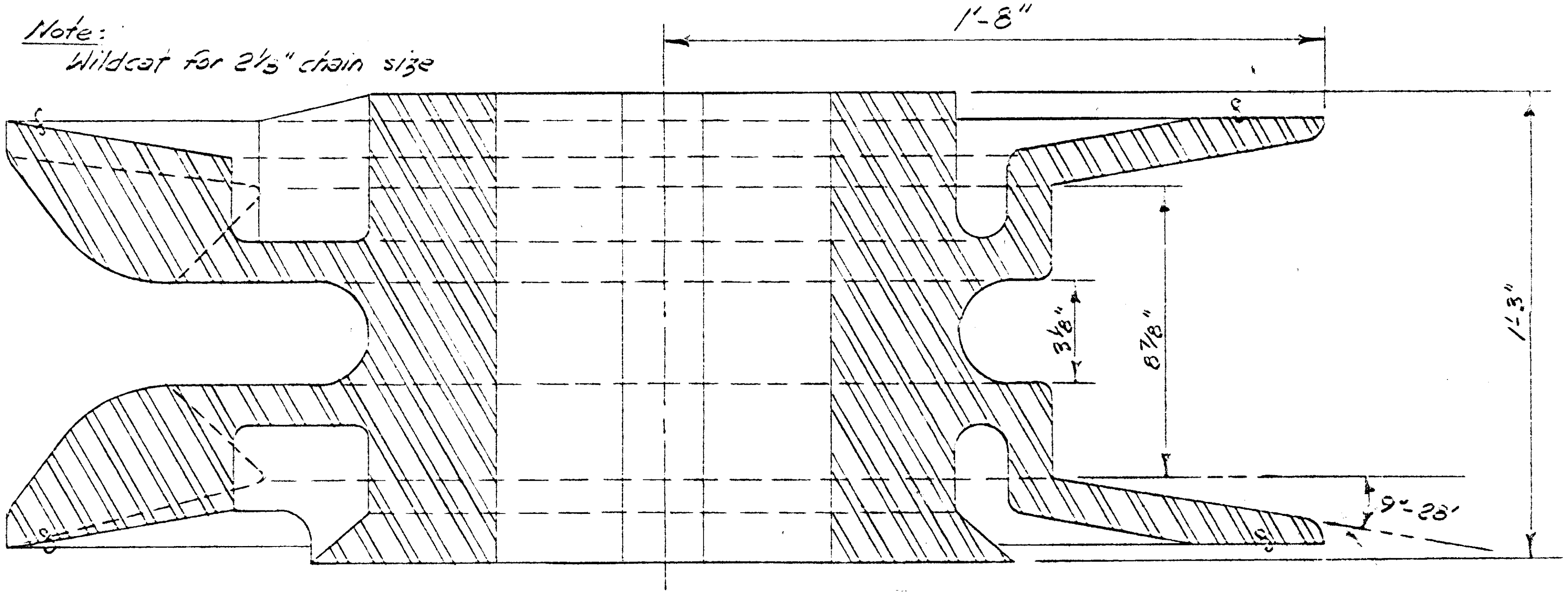
PLAN

N.T.S.



DETAIL OF CHAIN

Note:
Wildcat for 2 1/8" chain size



SECTION "A-A"

N.T.S.

NEW ORLEANS TO VENICE, LA.

DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN

REACH B-TROPICAL BEND TO FORT JACKSON

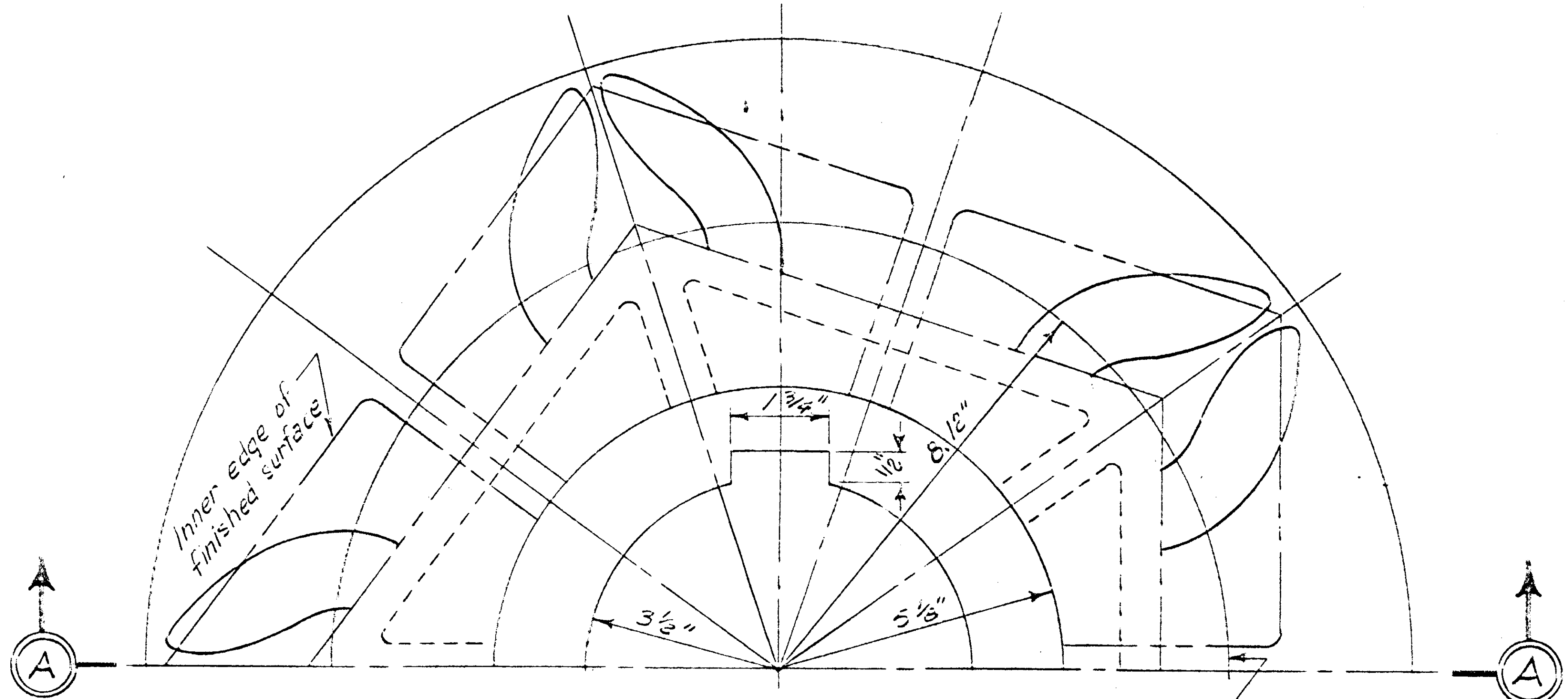
EMPIRE FLOODGATE

GEAR-DRIVEN WILDCAT

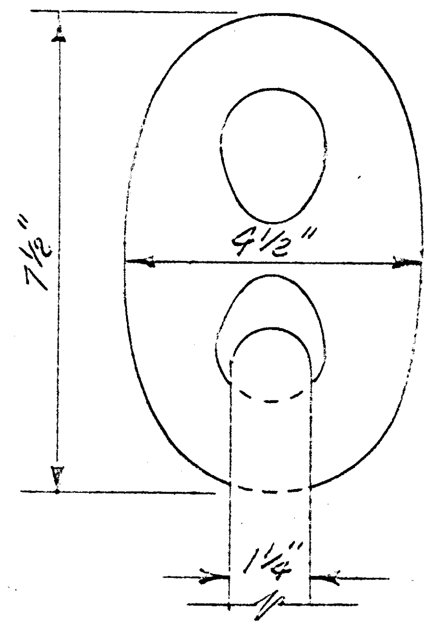
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

OCTOBER 1970 FILE NO. H-2-25048

PLATE V-3

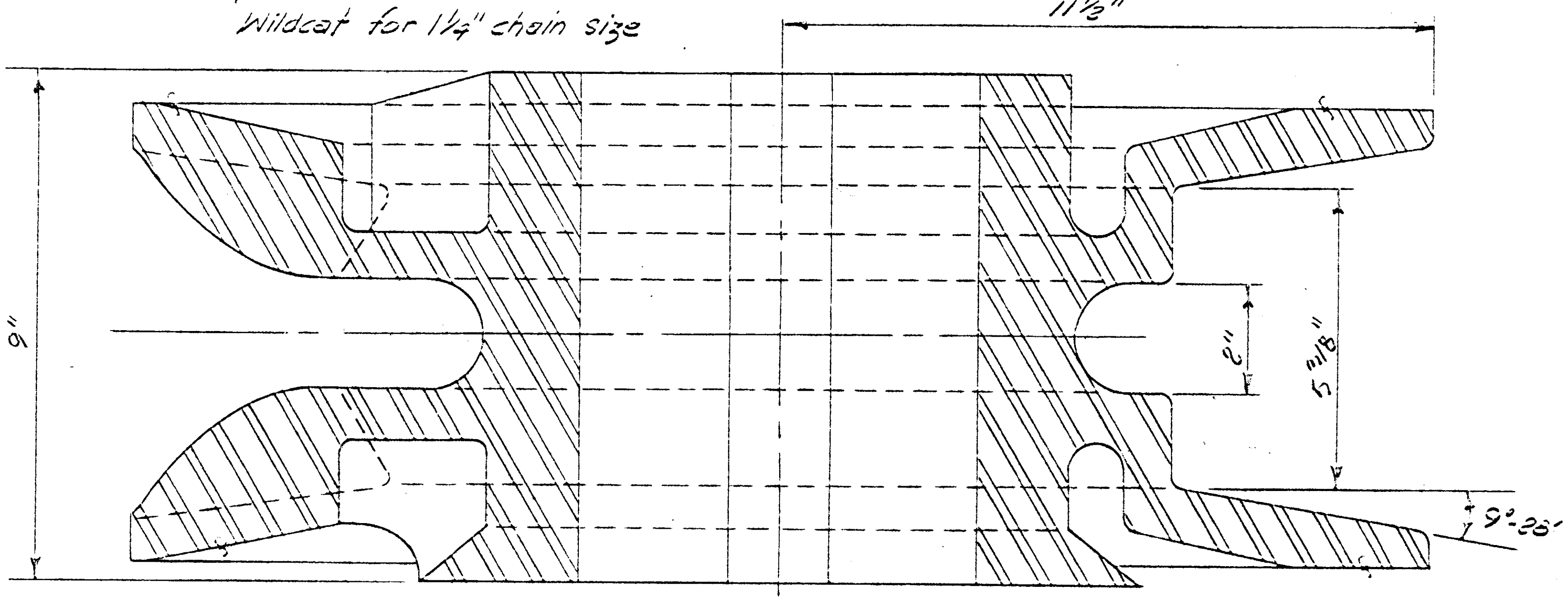


PLAN
N.T.S.



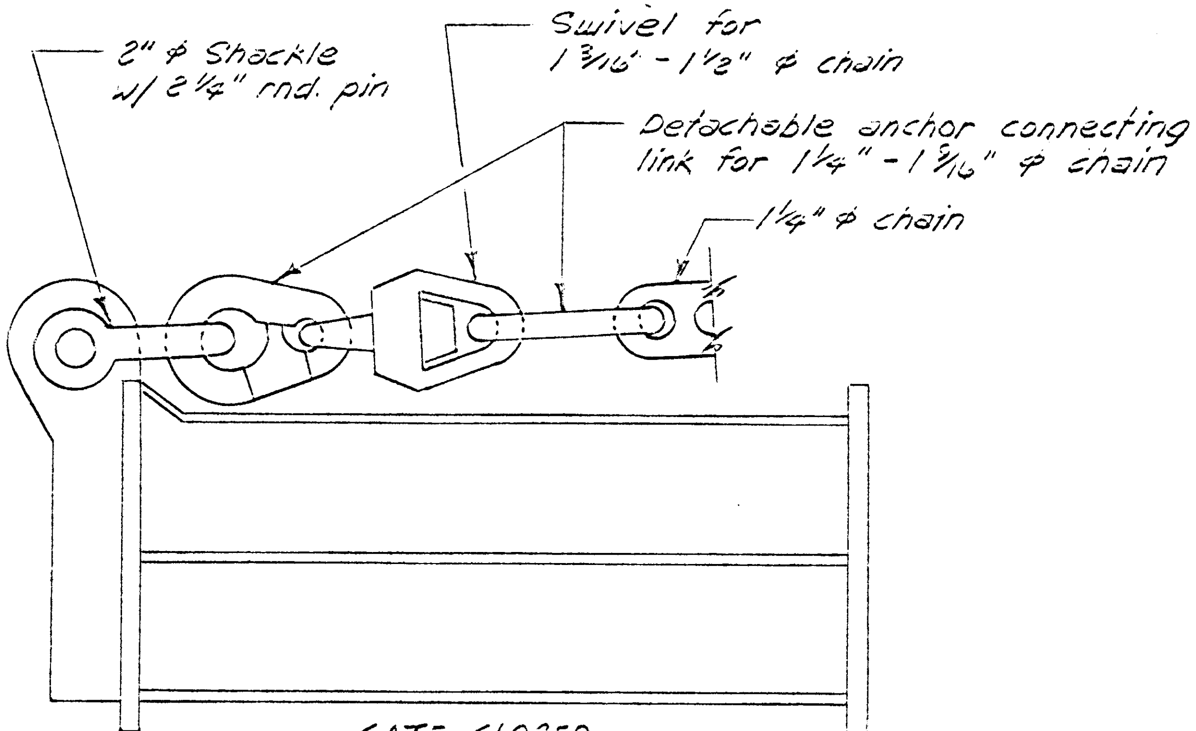
DETAIL OF CHAIN

Note:
Wildcat for 1¼" chain size



SECTION "A-A"
N.T.S.

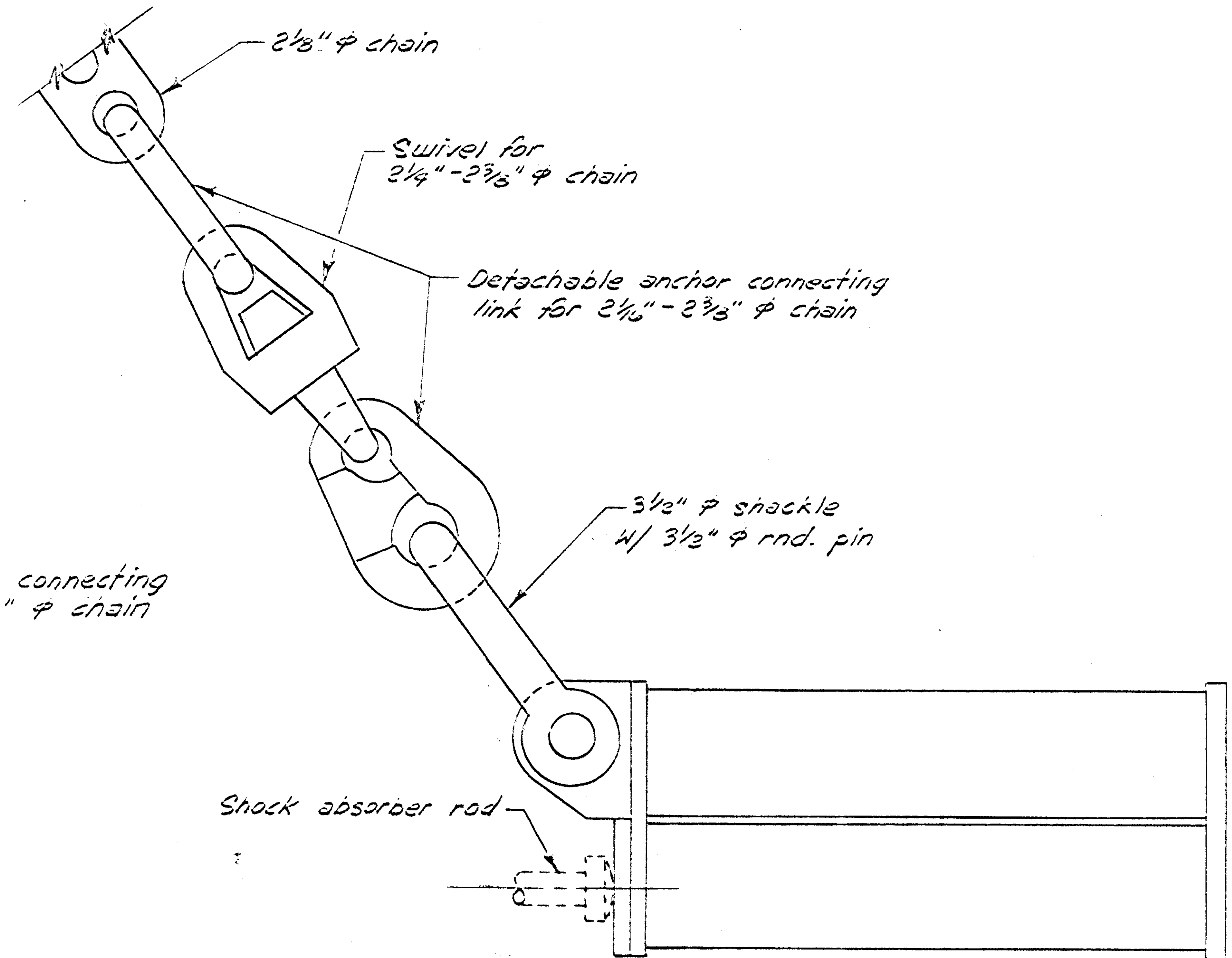
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
FREE-WHEELING WILDCAT
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



GATE CLOSED

CWT. CHAIN CONNECTION TO GATE GIRDER

N.T.S.

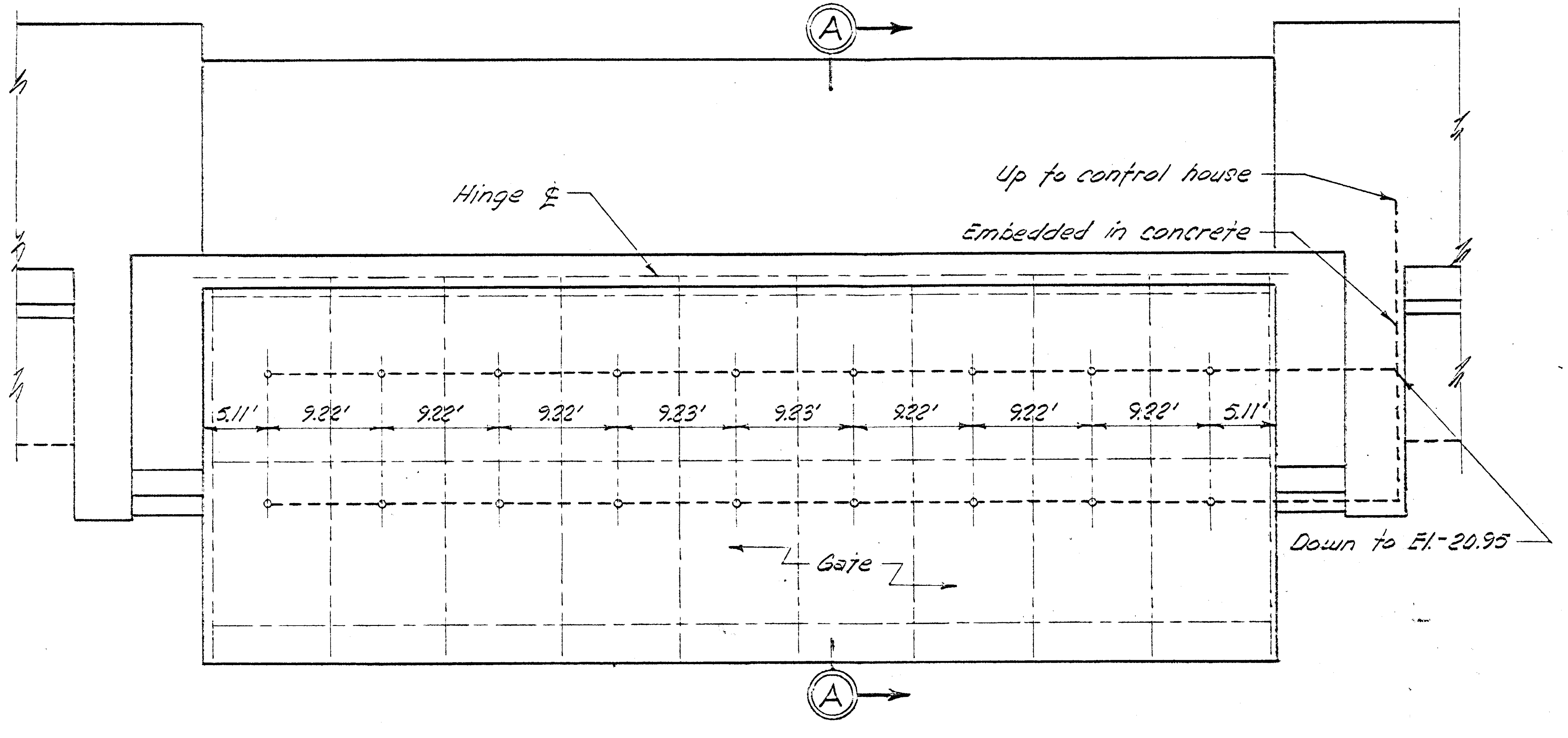


GATE CLOSED

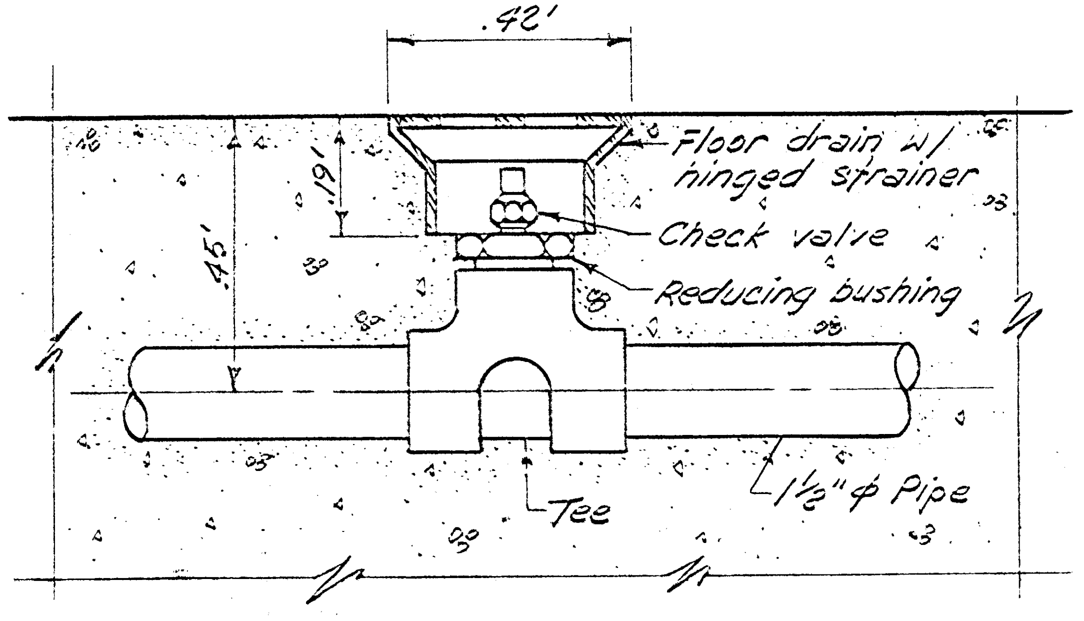
HOIST CHAIN CONNECTION TO GATE GIRDER

N.T.S.

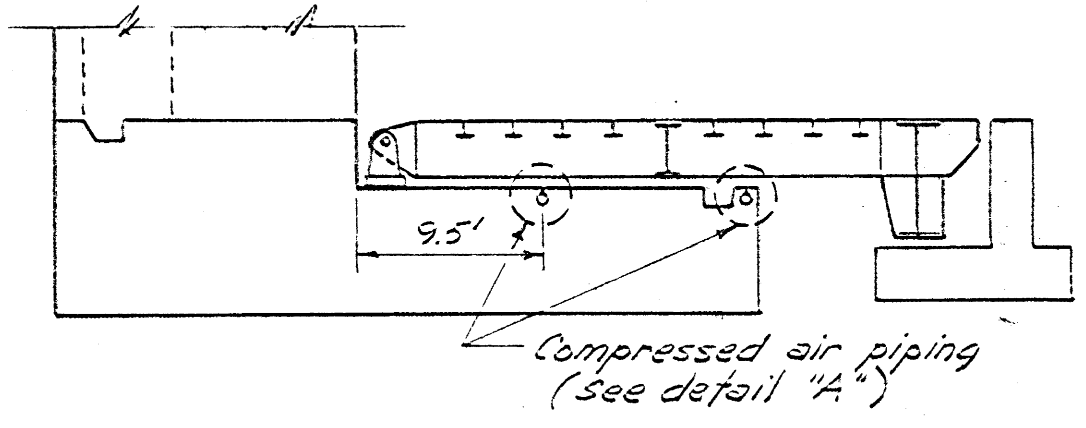
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
CHAIN CONNECTIONS TO GATE
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048



PLAN
Scale: 1" = 10'



DETAIL "A" - AIR OUTLET
Scale: 3" = 1'-0"



SECTION "A-A"
Scale: 1" = 10'

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
AIR COMPRESSOR PIPING
LAYOUT VICINITY FLOODGATE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970 FILE NO. H-2-25048

SECTION VI - ELECTRICAL DESIGN

POWER SYSTEM

1. Power supply. Because of the remote location and the intermittent operation of the gate, power will be provided by an L-P gas, engine-generator set. The generator will be rated 30 kw, 0.8 P.F., 480 volts, 3 phase, 60 Hz.

2. Power distribution.

a. General. Power will be distributed from the main switchboard in the control house at 480 volts, 3 phase to the control center. The power circuits are shown on the single line diagram, plate VI-1.

b. Main switchboard. A main switchboard of the free standing, ~~deadfront distribution~~ motor control center type, including generator and flapgate control sections, will be provided in the control house.

~~c. Control center. A motor-control center-type switchboard will be provided in the control house to control the flap gate.~~

OPERATIONAL CONTROL

3. Gate control.

a. General. The flap gate will be operated from the control desk in the control house on the east side of the structure. The schematic control diagram for the flap gate is shown on plate VI-2.

b. Gate closure. To close the gate the air compressor must first be started to pump air under the gate to break the suction. Starting of the air compressor will energize a time delay relay which will prevent operation of the gate for 10 minutes. The gate may then be closed by turning the control switch to the "closed position" and energizing the hoist motors. Limit switches on each side of the gate will stop the hoist motors when the gate is fully closed.

c. Gate opening. To open the gate, the latching devices are withdrawn from the gate which will close the limit switches in the "open circuit." The gate control switch is then turned to the open position to energize the hoist motors. Limit switches on the machinery will stop the hoist motors when the gate is fully opened.

d. Emergency stop. The gate control circuit is provided with an emergency stop button to stop the gate at any time during its operation. To restart the gate when it is closing, the "bypass" switches must be pressed to energize the hoist motors when the gate control switch is turned to the closed position. To restart the gate when it is opening, the gate control switch is turned to the open position. The limit switches will stop the hoists when the gate reaches the fully open or fully closed position.

SIGNAL AND COMMUNICATION SYSTEMS

4. Navigation signals.

a. General. Lighted navigation signals will be provided on the control house on the east side of the structure and will provide directional beams up and down the channel.

b. Control. The navigation signals will be controlled by means of a relay in the gate control circuit. The signals will be powered from primary dry cell batteries and will display a red signal at all times except when the flap gate is in the fully opened position. Energizing the gate control circuit will automatically energize the navigation signals during normal operation. A bypass switch will be provided for operation of the signals when the gate is in the closed position.

5. Horn signals. Back-to-back air horns complete with integral air compressor will be installed on the control house. Energizing the gate control circuits will sound the air horns.

6. Telephone system. A sound powered, common-talking, selective-ringing telephone system with three stations will be provided. One station will be located in the control house and one station at each of the machinery locations.

LIGHTING SYSTEM

7. Lighting system.

a. General. Power will be provided by the engine generator for gate bay lighting during emergency operations. Power for the structure guard lights will be provided by dry-type primary batteries.

b. Luminaires. Gate bay luminaires will be mounted on the control house and will be wall-mounted-type floodlights with 400 watt mercury vapor lamps, integral ballasts, reflectors, and glassware.

c. Guard lights. Luminaires will be 150-millimeter marine lanterns with 360 degree red acrylic Fresnel lens, integral sunswitch, and four lamp automatic lampchanger. Lanterns will be equipped with 6.2 volt, 0.46 amp lamps.

GROUNDING SYSTEM

8. Grounding system. A grounding system will be provided to which all metal conduits and electrical equipment will be connected. Ground electrodes will be copper-clad steel ground rods three-fourths inch diameter by 15 feet long. Ten ground rods will be driven under the base slab of the gate bay. All conduits and electrical equipment will be interconnected with the grounding system.

INSULATED WIRE AND CABLE

9. Power, control, and lighting circuits. Insulated wire and cable for power, control, and lighting circuits will conform to Guide Specification No. CE 1404.04, Insulated Wire and Cable (for Hydraulic Structures).

10. Switchboard wire. Insulated wire for the main switchboard, control centers, and control desks will conform to the requirements of NEMA Standards Publication WC-1-1963 for Thermoplastic Asbestos Insulated Wire.

11. Communication cables. Cables for the telephone systems will have polyethylene insulated conductors with metalized paper shielding tape and polyvinyl chloride sheath.

ELECTRICAL LOAD CALCULATIONS

12. General. Electrical loads for lighting and switchboard operations will be as follows:

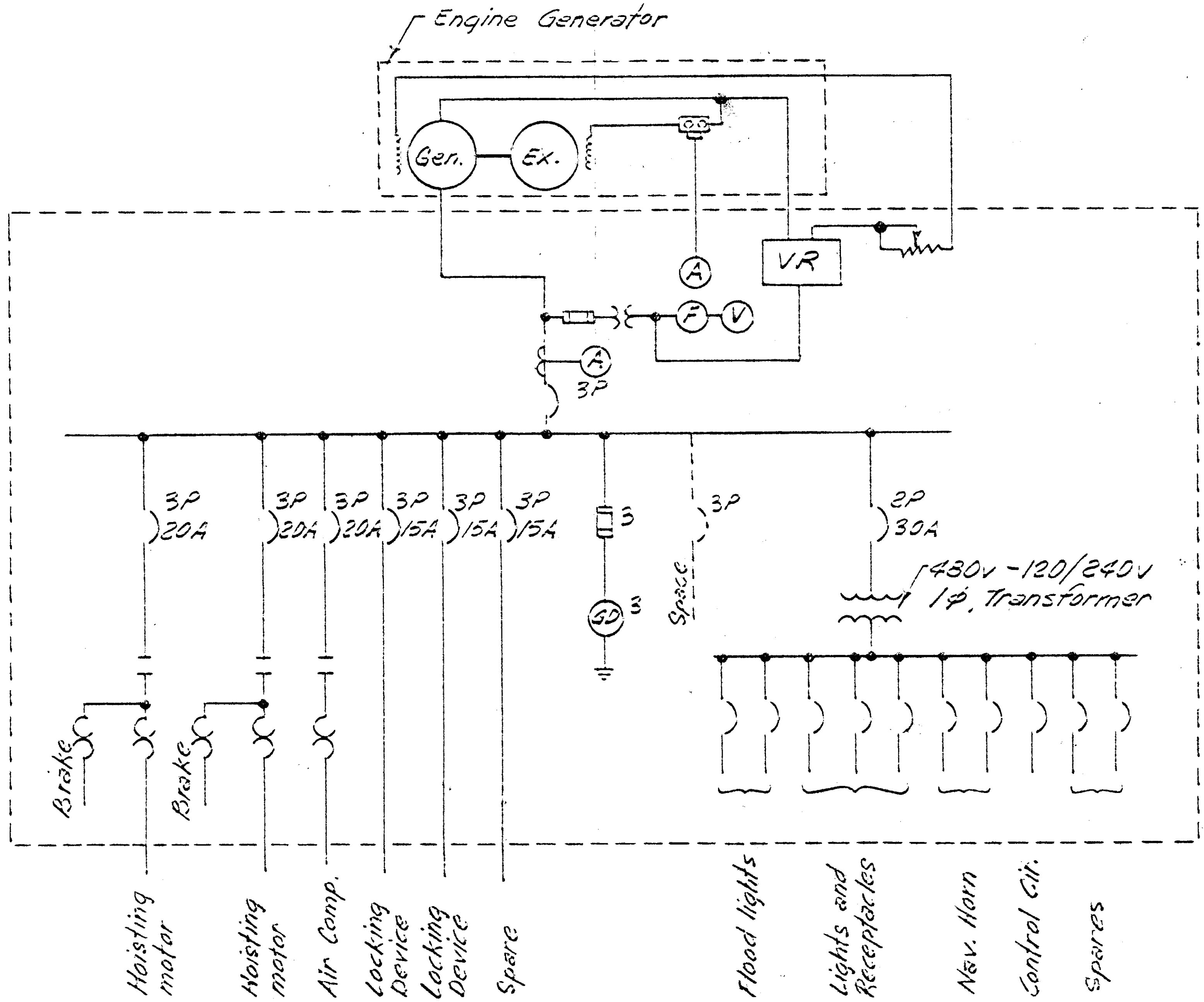
a. Lighting loads. The load on the lighting transformer in the control center will be:

	Connected load Kva	Demand load Kva
Gate bay lights	2.0	2.0
C.H. lights & receptacles	2.0	.4
Navigation signals	0.6	0.6
Total	4.6	3.0

b. Switchboard loads. In calculating the switchboard loads, line currents are assumed to be in phase with each other, and are added arithmetically. Actual currents will be somewhat less than the calculated values.

	Phase A-B Kva	Phase B-C Kva	Phase C-A Kva
Hoist motors (2)	5.8	5.8	5.8
Air compressor	2.9	2.9	2.9
Locking device (2)	1.0	1.0	1.0
Light transformer	3.0	-	-
Total	12.7	9.7	9.7

Total 3ϕ Kva = 12.7 + 9.7 + 9.7 = 32.1 Kva
A generator rated 30 Kw - 37.5 Kva will be selected.

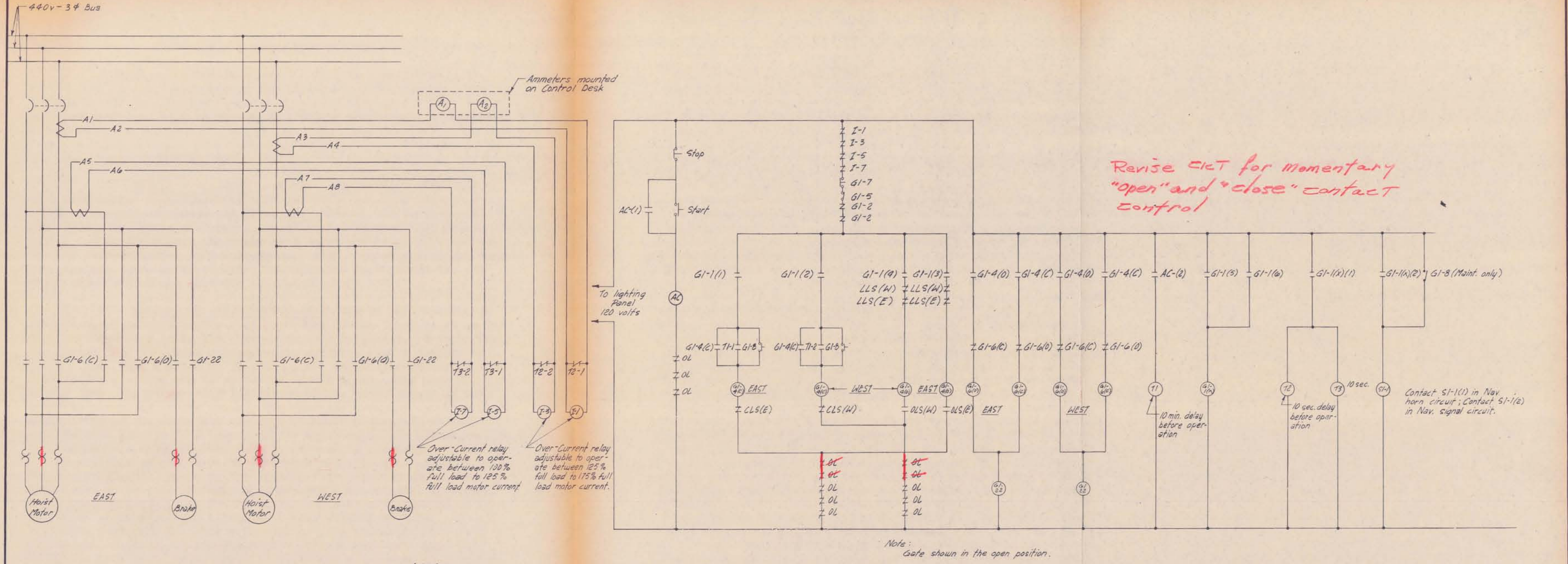


NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
SINGLE LINE DIAGRAM

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

OCTOBER 1970 FILE NO. H-2-25048



Note:
Gate shown in the open position.

GATE CONTROL CIRCUIT

LEGEND

G1-2	Brake release interlock
G1-4	Main control relay
G1-5	"Safe-on" Switch
G1-6	Hoist motor Reversing Contactor
G1-7	Emergency stop
G1-8	Emergency by-pass switch
G1-22	Brake contactor
G1-1(x)	Auxiliary relay
S1-1	Navigation signal relay
OLS	Open limit switch
CLS	Close limit switch
LLS	Latch limit switch
AC	Air compressor relay
T	Time delay relay
I	Overcurrent relay, hand reset
E	East
W	West

Contact No	Open	~~Off~~ NEUT	Closed
1			X
2			X
3	X		
4	X		
5			X
6	X		

Development of Gate Control Switch No G1-1

Use spring-return-to-neutral control switch

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE GATE CONTROL CIRCUIT

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

OCTOBER 1970 FILE NO. H-2-25048

SECTION VII - CATHODIC PROTECTION

1. General. The Empire Floodgate will be constructed near Empire, La., at the river end of the Empire to Gulf Waterway. The floodgate will be a bottom hinged single-leaf flap gate which, in the open position, will be stored in a recess in the base slab of the structure.

2. Water characteristics.

a. Salinity records. Salinity observations were made in this area by the Louisiana Wild Life and Fisheries Commission and the U. S. Bureau of Commercial Fisheries, Federal Aid Section, at Bay Pomme d'or, Bayou Maringouin, Grand Bayou, Halfmoon Bay, Sandy Point, and Scofield Bay. Readings were taken at the above locations for the period October 1966 through July 1969. Locations of sampling stations are shown on plate VII-1. Applicable data sheets for the above observations are included in appendix A.

b. Summary of salinity records. The inland penetration of salt water in this area varies according to the fresh water runoff, tidal changes, and sustained changes in wind direction. Records for the period 1966-69 reveal variations in salinity from a minimum of 1,800 p.p.m. to a maximum of 30,100 p.p.m. Salinity readings of the four inland stations are in excess of 10,000 p.p.m. more than 90 percent of the time. Salinity is an indication of the total amount of salt which is dissolved in the water. The chloride ions not only lower the resistivity of the water but are objectionable chemically as they combine with iron to form iron chloride which is easily hydrolyzed to produce an acid solution thus further increasing corrosion. Lowering of the resistivity of the water increases the intensity of the currents flowing from anodic areas on the structure to cathodic areas thereby increasing corrosion. Reference is made to the textbook, Corrosion and Corrosion Control, by Uhlig,[1] which contains a discussion on the corrosion of iron and the effect of dissolved NaCl in water. It states in part that the corrosion rate of iron increases with NaCl concentration, reaching a maximum ~~of~~ at 3 percent NaCl and then decreases to a value below that for distilled water when saturation is reached (26% NaCl). Uhlig's curve showing the relationship between NaCl concentration and corrosion rate is shown on plate VII-2. Comparison of the salinity readings from the Empire area with the curve indicates that a high corrosion rate must be anticipated. The maximum salinity readings obtained during the period of observation indicate an environment in which ferrous metal components will be subject to severe corrosion. Accordingly, the design has been made adequate for the maximum corrosion rate.

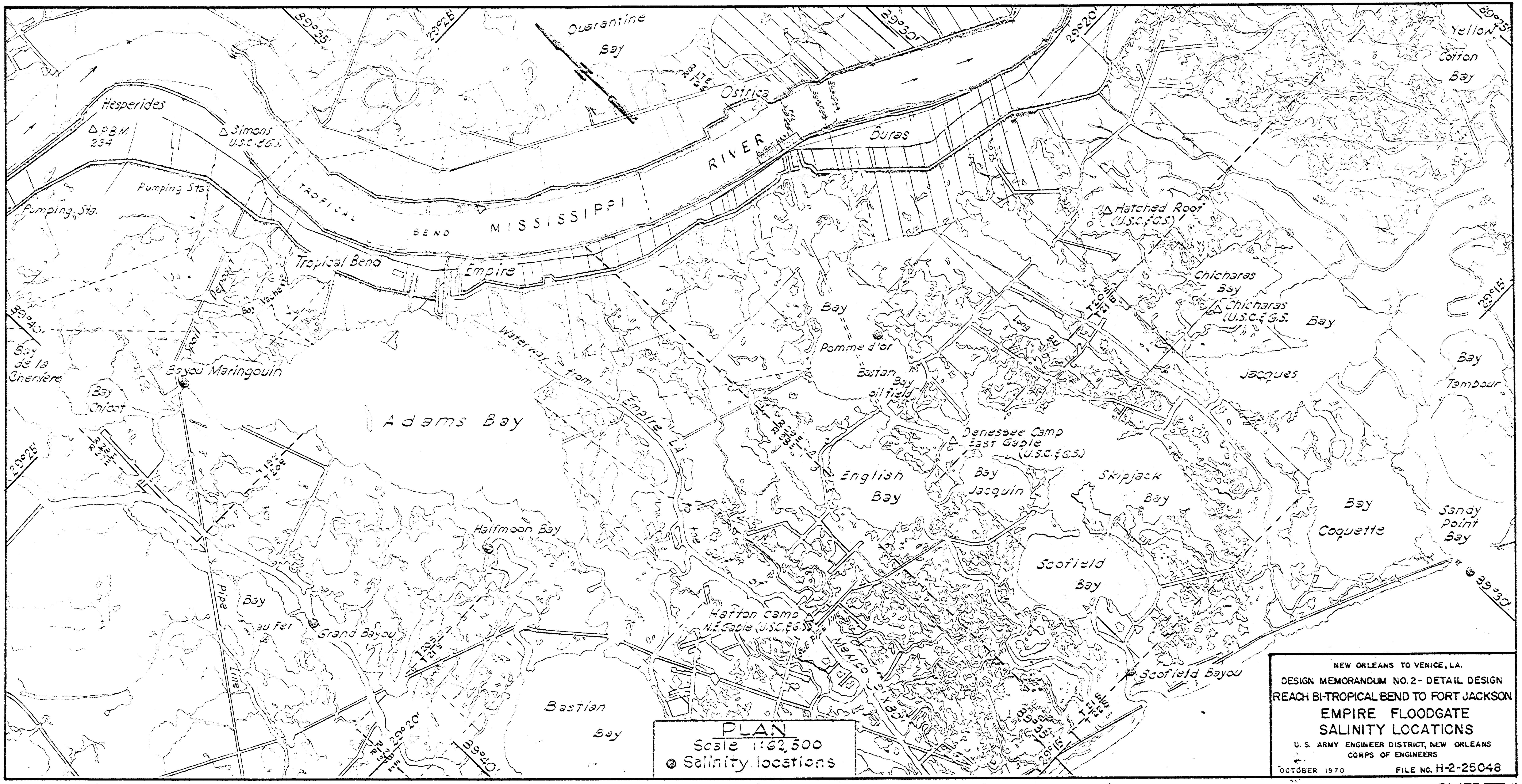
---

[1] H. H. Uhlig, Corrosion and Corrosion Control, John Wiley & Sons, Inc.

3. <u>Corrosion protection measures</u>.

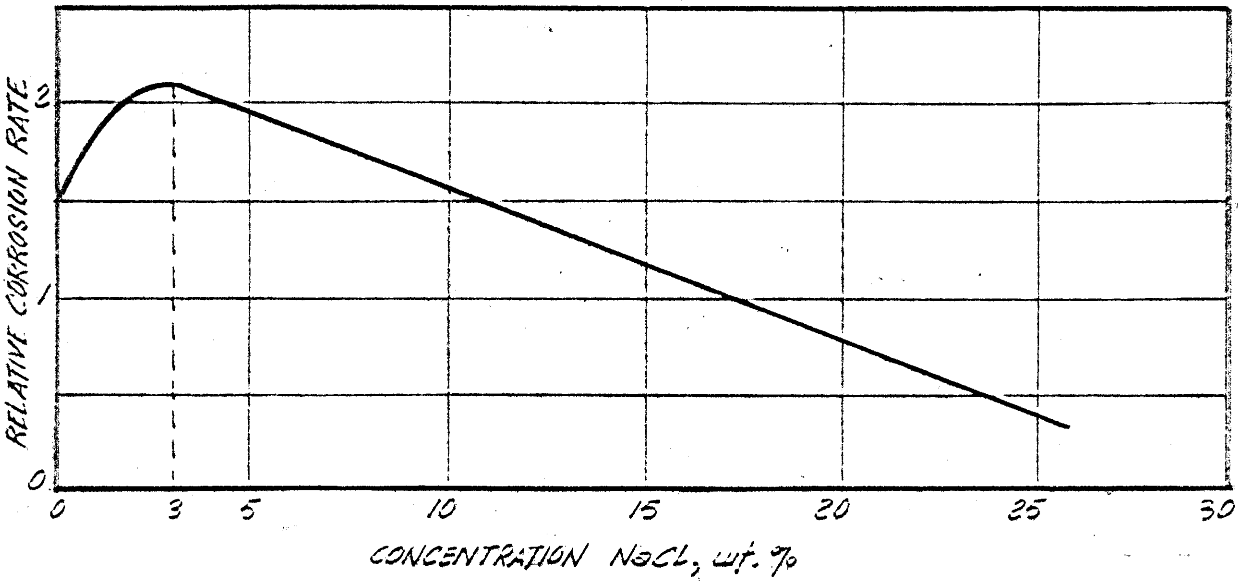
a. <u>General</u>. Cathodic protection provided for the flap gate will be of the sacrificial metal type as a supplement to 7.5 mils of a zinc rich vinyl paint and is designed to protect both sides of the gate. A current density of not less than 0.0003 amperes per square foot of protected surface is provided for the painted areas. The sacrificial metal type system was selected because the structure is unmanned and commercial power is not available at the site.

b. <u>Anodes</u>. The anodes will be high purity zinc anodes rated 335 ampere-hours per pound at a 90 percent efficiency with a solution potential of -1.10 volts relative to a Copper-copper sulphate reference half-cell. The system is designed to provide a polarization potential of -0.85 volts measures to a reference half-cell. Slab (hull) type anodes weighing 12 pounds will be utilized on the skin plate and 1.4" x 1.4" square anodes weighing 6 pounds per foot will be used on the structural members on the underside of the gate. The number and size of the anodes were selected to obtain 20-year life and to insure current distribution to shielded areas of the gate. The anodes will be welded to the gate members and the skin plate. Details of anodes, mountings, and locations are shown on plate VII-3. Design calculations are shown in appendix A.



Quarantine Bay
Mississippi River
Hesperides
P.B.M. 234
Simons U.S.C.&G.S.
Pumping Sta.
Pumping Sta.
Tropical Bend
Empire
Ostrica
Buras
Yellow Cotton Bay
Hatched Roof (U.S.C.&G.S.)
Chicharas Bay
Chicharas (U.S.C.&G.S.)
Bay Jacques
Bay Tambour
Long Bay
Bay Pomme d'or
Bastian Bay oil field
Bay de la Cheniere
Bay Chicot
Bayou Maringouin
Adams Bay
Waterway from Empire La. to the Gulf of Mexico
Denessee Camp East Gable (U.S.C.&G.S.)
English Bay
Bay Jacquin
Skipjack Bay
Bay Coquette
Sandy Point Bay
Scofield Bay
Halfmoon Bay
Hafton Camp N.E. Gable (U.S.C.&G.S.)
Bay au Fer
Grand Bayou
Pipe Line
Bastian Bay
Scofield Bayou
PLAN
Scale 1:62,500
Salinity locations
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
SALINITY LOCATIONS
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970
FILE NO. H-2-25048

PLATE VII-1



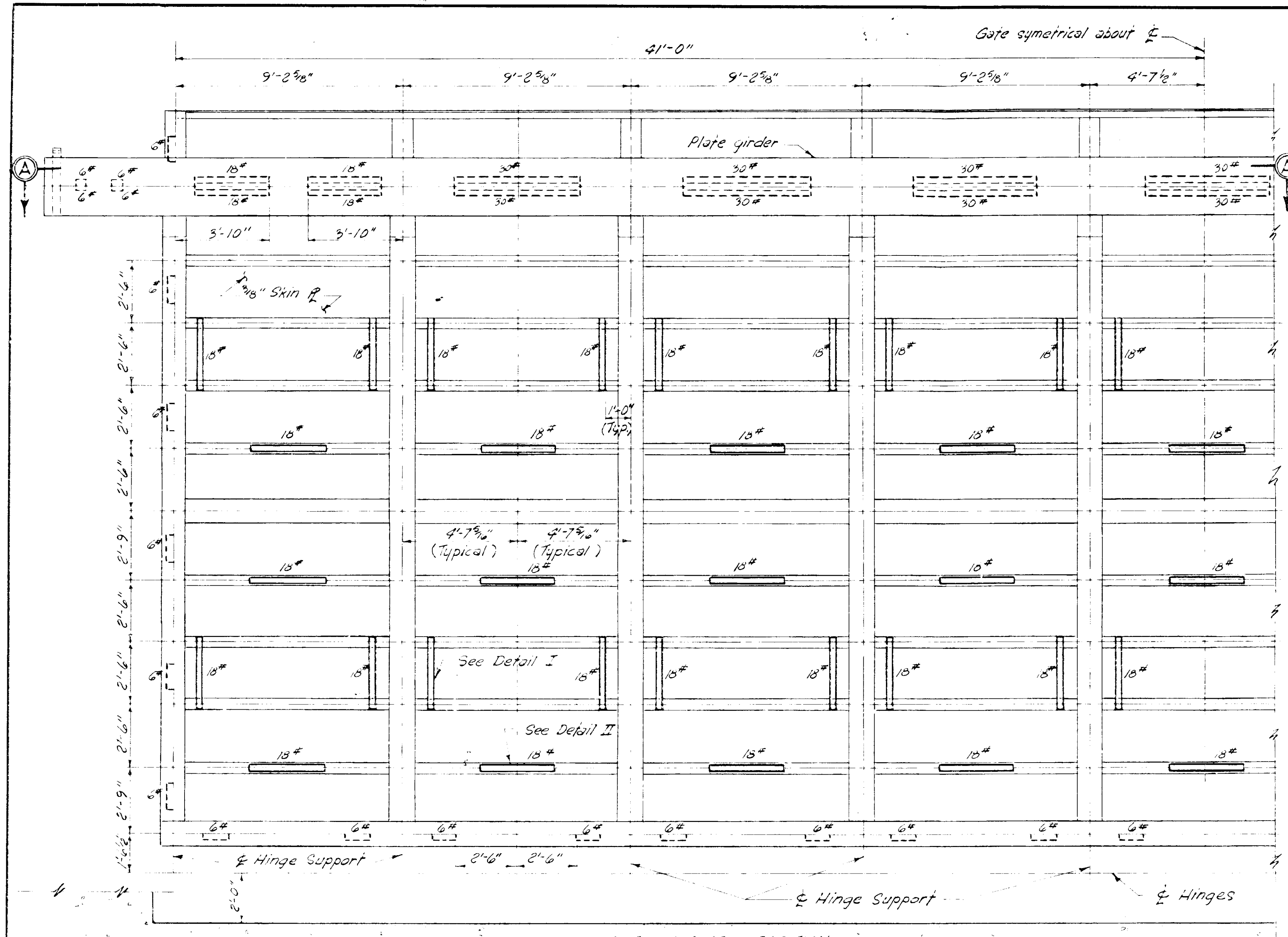
Note: 3% NaCL concentration (sea water concentration) is approximately 16,000 p.p.m. chlorides.

NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH BI-TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE
RELATIVE CORROSION RATE
VS.
CONCENTRATION NaCl, wt.%

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

OCTOBER 1970 FILE NO. H-2-25048



GATE IN CLOSED POSITION

FLOOD SIDE ELEVATION

Scale 3/16"=1'-0"

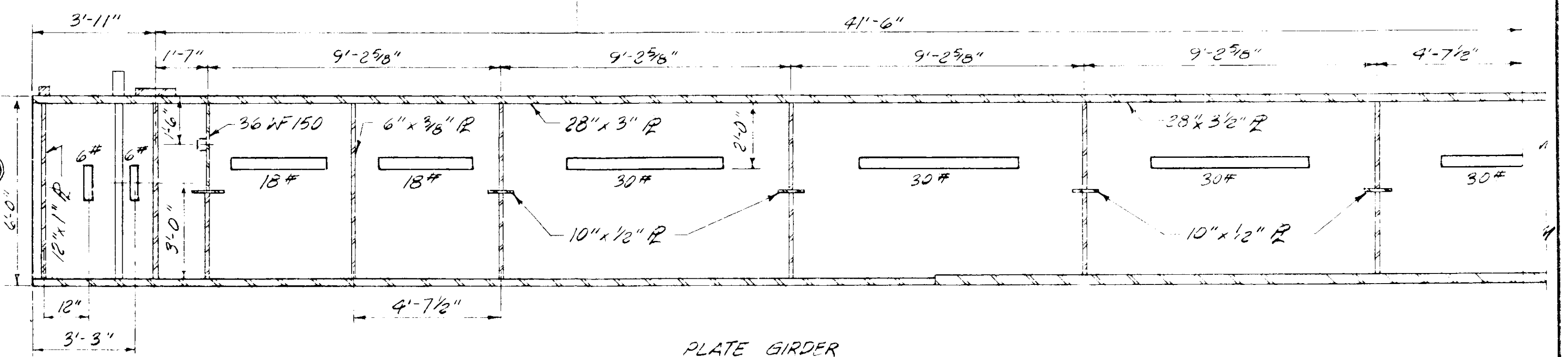
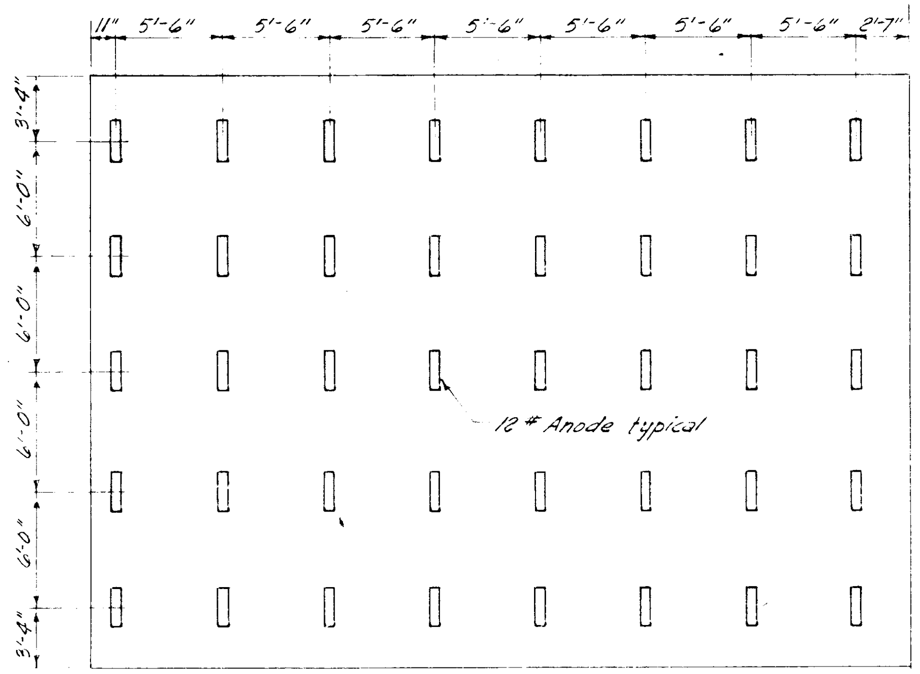


PLATE GIRDER

SECTION "A-A"

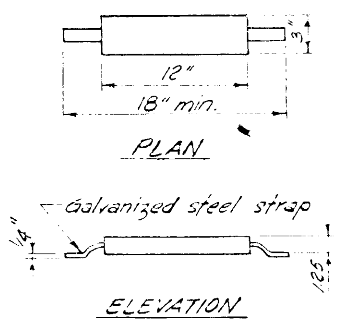
Scale 3/16"=1'-0"



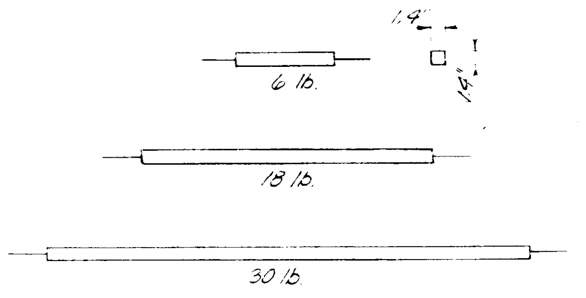
PROTECTED SIDE

SKINPLATE ANODE LAYOUT

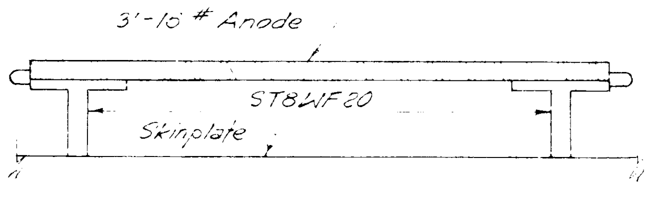
N.T.S.



12 lb. ZINC ANODE

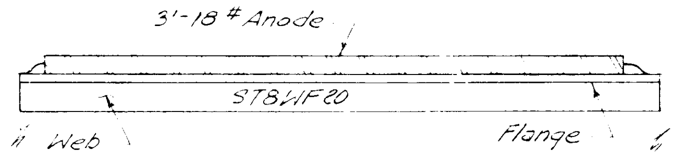


1.4"x1.4" ZINC ANODE



DETAIL I (Typical)

N.T.S.



DETAIL II (Typical)

N.T.S.

NEW ORLEANS TO VENICE, LA

DESIGN MEMORANDUM NO. 2 – DETAIL DESIGN

REACH B 1 – TROPICAL BEND TO FORT JACKSON

EMPIRE FLOODGATE

CORROSION PROTECTION

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

OCTOBER 1970 FILE NO. H-2-25048

SECTION VIII - ESTIMATE OF COST

1. Detailed estimate of first cost. The items of work included herein for the Empire Floodgate structure are included in the feature "Levees and Floodwalls (Account 11)" in the New Orleans to Venice, La. (Miss. River Delta at and below New Orleans) Reach B1 Project Cost Estimate (PB-3). Detailed estimates of first costs are shown in table 1.

Table 1
Detailed Estimate of First Cost
Empire Floodgate
July 1970 price levels

Item	Description	Quantity	Unit	Unit price $	Est.Cost $
STRUCTURE					
11	Levees and floodwalls				
	Cofferdam shell fill	15,500	c.y.	5.50	85,250
	Cofferdam clay fill	20,500	c.y.	3.50	71,750
	Excavation (struc)	28,000	c.y.	3.00	84,000
	Dewatering and piezometers	Lump sum	L.S.		125,000
	Levee fill	16,500	c.y.	3.50	57,750
	Shell backfill and blanket	22,500	c.y.	6.50	146,250
	Riprap	10,600	tons	12.00	127,200
	Piling - steel sheet				
	Z-32	7,200	s.f.	6.00	43,200
	MA-22	7,700	s.f.	4.00	30,800
	Piling untreated timber	21,200	l.f.	2.50	53,000
	Piling - prestressed concrete 12"x12"	16,500	l.f.	8.00	132,000
	Pile anchors (timber)	660	ea.	10.00	6,600
	Pile loading tests	2	ea.	3,000.00	6,000
	Treated timber guide walls	800	l.f.	200.00	160,000
	Floating guidewall(camel)	2	ea.	8,000.00	16,000
	Conc. in stabilization slab	150	c.y.	40.00	6,000
	Conc. in sills and T-wall base	1,670	c.y.	40.00	66,800
	Conc. in struc. walls and stems	1,200	c.y.	70.00	84,000
	Conc. in needle beams	200	c.y.	60.00	12,000
	Portland cement	4,800	bbls.	5.25	25,200
	Steel reinforcement	485,000	lbs.	0.16	77,600
	Misc. metal	10,000	lbs.	0.80	8,000
	Gate, hinged	191,000	lbs.	0.70	133,700

Item	Description	Quantity	Unit	Unit price $	Est.cost $
	Needle girders	89,500	lbs.	0.70	62,650
	Structural steel	34,500	lbs.	0.70	24,150
	Pipe handrail	1,210	l.f.	8.50	10,290
	Waterstop	280	l.f.	5.00	1,400
	Control house	Lump sum	L.S.		10,000
	Operating machinery	Lump sum	L.S.		195,000
	Chain	Lump sum	L.S.		10,400
	Counterweights, lead	Lump sum	L.S.		15,000
	Engine generator	Lump sum	L.S.		13,000
	Electrical	Lump sum	L.S.		100,000
	Cathodic protection	Lump sum	L.S.		25,000
	Dressing and seeding	2.5	acres	350.00	880
	Compressor & piping	Lump sum	L.S.		7,000
	Access road	Lump sum	L.S.		50,000
					2,082,870
	Contingencies (20%±)				417,130
11	Levees and floodwalls - total construction cost				2,500,000
30	Engineering and design (11.4%±)				285,000
31	Supervision and administration (10.2%±)				255,000
	Total				3,040,000

2. Comparison of costs. The cost of $3,040,000 for the Empire Floodgate structure represents an increase of $700,000 from the latest PB-3 effective 1 July 1970. Table 2 shows a comparison of the PB-3 and detail design memorandum estimate.

Table 2
Comparison of Estimates
Empire Floodgate

Feature		PB-3 eff. 1 Jul 70	DM No. 2	Diff.DM No. 2 - PB-3
11	Levees and floodwalls	$2,000,000	$2,500,000	$500,000
30	Engineering and design	200,000	285,000	85,000
31	Supervision and administration	140,000	255,000	115,000
	Total	$2,340,000	$3,040,000	$700,000

3. Explanation of difference between the latest approved cost estimate and the DDM estimate. Major differences between the latest approved cost estimate (PB-3 dated July 1970) and the design memorandum cost estimate (July 1970 price level) are as follows:

a. Levees and floodwalls. The DM estimate of $2,500,000 is $500,000 more than the PB-3 allocation of $2,000,000. This increase is due principally to the items listed in table 3.

Table 3
Levees and Floodwalls
Significant Differences

	PB-3 eff. 1 Jul 70	Design Memo No. 2	Diff. DM No. 2 PB-3
(1) Cofferdam fill	$ 99,200	$156,500	+$ 57,300
(2) Backfill	23,050	146,250	+ 123,200
(3) Riprap	50,000	127,200	+ 77,200
(4) Operating machinery	61,000	195,000	+ 134,000
(5) Service and office bldg.	36,500	0	- 36,500
(6) Contingencies	214,000	417,130	+ 203,130
Total	$483,750	$1,042,080	+$558,330[1]

[1] Other miscellaneous items reduce this figure to $500,000 (rounded).

Explanation:

(1) Cofferdam fill. Because of the required excavation plan the quantities of fill required for the cofferdam were increased by approximately 14,400 cubic yards to 36,000 cubic yards (15,500 c.y. of shell fill and 20,500 c.y. of clay fill). It was also necessary to use a shell core in the stream closures.

(2) Backfill. The structure will be backfilled with shell instead of using random fill as originally planned, because of foundation strength requirements and a lack of suitable borrow material at the structure site.

(3) Riprap. The quantities for riprap have been increased by 6,500 tons from the latest PB-3 estimate.

(4) Operating machinery. The size of the operating machinery was increased in order to operate under required design conditions which were not previously investigated.

(5) Service and office building. The service and office building has been eliminated.

(6) Contingencies. Contingencies were increased from 12 percent in the PB-3 estimate to 20 percent for the DM estimate which is consistent with the contingencies currently being used on all hurricane projects in the New Orleans vicinity.

b. Engineering and design. Engineering and design costs were raised from 10 percent (PB-3) to 11.4 percent (DM) which is consistent with the most recent design cost used on similar type structures. The total DM cost for E&D is $285,000 which is $85,000 above the PB-3 estimate of $200,000.

c. Supervision and administration. Supervision and administration costs were increased from 7 percent to 10.2 percent to agree with our most recent estimates. The total DM cost for S&A is $255,000 which is $115,000 over the PB-3 estimate of $140,000.

4. Comparison to previously-furnished cost (8th Ind. "DM No. 1, New Orleans to Venice"). The total estimated cost of $3,040,000 for the floodgate structure represents an increase of $1,120,000 from the estimate furnished in the 8th Indorsement to "Design Memorandum No. 1, New Orleans to Venice." This increase in the estimated cost is due principally to the reasons outlined in paragraph 3 and the escalation of unit prices.

5. Comparison to other plans investigated. Although the estimate for the floodgate has been increased considerably the "bottom hinged" type gate used is still the most economical of the three type gates investigated in the 8th Indorsement to "Design Memorandum No. 1, New Orleans to Venice." With the exception of the increase in operating machinery cost, all of the increases in cost resulted from revised foundation requirements, revised unit prices, revised contingencies, and revised E&D and S&A percentages. Similar increases in cost would have been required for the other two type gate structures ("Horizontal-Rolling Type" and "Sector Type") studied. The revised costs for the "Horizontal-Rolling Type Gated" structure and the "Sector Type" structure are $3,060,000 and $3,840,000, respectively.

6. Schedule for design and construction.

Table 4
Schedule for Design and Construction

Design Plans and Specifications		Construction	
Start	Complete	Start	Complete
Feb 71	Nov 71	Feb 72	Feb 74

SECTION IX - RECOMMENDATIONS

Recommendation. The Empire Floodgate consists essentially of a concrete U-frame gate bay with a steel gate hinged at the bottom--all supported on timber piling, a control house, timber guide walls and fenders, a breakwater, an access road, and T-type and I-type floodwalls connecting the gate chamber to the earthen levee on each side of the floodgate. This plan is considered to be the best means of accomplishing the project objectives and is recommended for approval.

APPENDIX A

SALINITY DATA AND DESIGN CALCULATIONS

APPENDIX A
SALINITY DATA

LOCATION: Bay Pomme d'Or

1966

Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												
7												
8												25.2
9											24.5	
10												
11												
12												
13												
14												
15												
16												
17											22.6	
18												
19												
20										22.7		
21												
22												21.5
23												
24											23.3	
25												
26												
27										20.4		
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Bay Pomme d'Or

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4					13.7					24.7		
5												
6												
7												
8			19.6									
9		14.3						14.2				
10					14.1							
11												
12				14.9					13.3			
13												
14						9.1						
15											21.8	
16				14.8				16.3				
17										22.7		
18	14.5			10.0								
19												
20												
21			20.2									
22												
23												
24												
25	17.9						13.0					
26				9.6					13.1			
27												18.8
28						10.4					23.2	
29												
30												
31										21.0		

APPENDIX A
SALINITY DATA

LOCATION: Bay Pomme d'Or

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1					9.8			9.8				
2				16.1			8.2					
3												
4	15.0								18.3			21.0
5												
6		18.6	20.4									
7			19.9									
8										23.0		
9							9.7					
10	13.7			12.2								
11											28.8	
12			18.6						15.8		24.0	
13												
14												
15		23.3			10.3			12.7		23.1		
16				13.0								
17						9.3						
18							10.2					
19						8.8			17.7			
20												
21												
22										24.2		
23	20.5											
24	16.1						9.5					
25												
26												
27			17.9									
28		23.2	18.4					13.2				
29					9.5							
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Bay Pomme d'Or

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				17.5			14.7					
2					12.4							
3				19.2								
4		17.9	16.0									
5												
6												
7												
8					10.6							
9				15.5								
10				15.8		11.8						
11												
12		17.3										
13						12.3						
14	27.4											
15	22.7				10.1							
16												
17												
18				9.7								
19			16.1									
20		14.6										
21												
22												
23					8.7							
24												
25												
26												
27			15.9									
28												
29												
30					8.7							
31												

APPENDIX A
SALINITY DATA

LOCATION: Bayou Maringouin

1966
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												24.2
7												
8											20.3	
9												
10												
11												
12												
13												
14												
15											22.8	
16												
17												
18										16.6		
19												
20												
21												
22											24.0	
23												
24												
25										17.3		
26												
27												
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Bayou Maringouin

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1			18.2								16.4	
2					15.0							
3										19.6		
4												
5												
6												
7							13.7					
8		13.4						12.4				
9					15.1							
10												
11				19.8								
12									15.1			
13						15.2				21.6		
14												
15				20.0								
16			21.0					12.9			20.0	
17												
18				16.2			14.3			20.4		
19	14.4											
20												
21												
22												
23												
24												
25				14.1								
26							16.0					
27												19.1
28												
29						13.1					22.4	
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Bayou Maringouin

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1								12.6				
2				17.5								
3												
4									16.0			19.3
5	14.6					11.1						
6		19.1	21.1									
7												
8												
9												
10												
11	15.5						11.6				22.8	
12												
13		19.6	20.8									
14					13.2							
15								13.3		22.6		
16												
17				14.8								
18						11.9						
19						11.8						
20												
21												
22												
23	19.0											
24							12.0					
25	19.0											
26			19.4									
27												
28		22.0			10.1			13.2				
29			18.6									
30				13.0								
31												

APPENDIX A
SALINITY DATA

LOCATION: Bayou Maringouin

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				16.8								
2					17.1							
3						11.7						
4			20.2			12.6						
5												
6												
7												
8												
9												
10												
11												
12		20.5										
13						13.0						
14	23.5											
15	24.0				12.2							
16				16.9								
17												
18				14.9								
19												
20		20.0										
21												
22												
23					11.5							
24												
25												
26												
27												
28												
29												
30					11.2							
31												

APPENDIX A
SALINITY DATA

LOCATION: Grand Bayou

1966
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												28.0
7												
8											29.0	
9												
10												
11												
12												
13												
14												
15											25.5	
16												
17												
18										26.0		
19												
20												
21												
22											28.5	
23												
24												
25										26.7		
26												
27												
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Grand Bayou

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1			27.8								18.5	
2					16.6							
3										24.4		
4												
5												
6												
7												
8								20.7				
9		17.2			18.8		20.5					
10												
11				17.4								
12									19.6			
13						12.7				30.1		21.2
14												
15				16.3								
16			28.1					17.8			26.3	
17												
18				13.4			22.2			24.1		
19	14.1											
20												
21												
22												
23												
24												
25				13.7								
26							18.0		5.1			
27												24.5
28												
29						16.3					26.6	
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Grand Bayou

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1		12.8										
2				6.2								
3												
4									18.1			20.5
5	18.1											
6		20.4	25.6									
7												
8												
9												
10	23.2									25.3		
11									19.5		29.8	
12												
13		25.7	22.3									
14											30.0	
15								16.5		26.7		
16				18.0								
17												
18												
19						13.8			21.5			
20											27.5	
21												
22										27.7		
23	16.3											
24							11.5					
25	21.4											
26												
27												
28		26.6	22.9					18.0				
29												
30												
31							14.6					

APPENDIX A
SALINITY DATA

LOCATION: Grand Bayou

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				21.0								
2												
3												
4		18.6	19.0			16.4						
5												
6												
7					14.6							
8												
9												
10												
11				17.7								
12		20.8										
13						15.0						
14	27.0											
15	28.0				17.8							
16				15.8								
17												
18												
19			19.4									
20		26.5										
21												
22												
23					10.9							
24												
25												
26												
27			19.9									
28												
29												
30				22.5	13.0							
31												

APPENDIX A
SALINITY DATA

LOCATION: Halfmoon Bay

1966
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												28.7
7												
8											23.9	
9												
10												
11												
12												
13												
14												
15											23.8	
16												
17												
18										25.1		
19												
20												23.5
21												
22											25.7	
23												
24												
25										20.9		
26												
27												
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Halfmoon Bay

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1			23.3								24.6	
2					17.9							
3										24.1		
4												
5												
6												
7												
8		16.8						17.2				
9					16.4							
10												
11												
12									17.0			
13				13.6		13.3				25.8		
14												
15				18.5								
16								16.9			23.6	
17												
18							17.7			23.8		
19	17.8											
20												
21												
22												
23												
24												
25												
26				10.5								
27							16.7					22.5
28												
29						11.8			18.3		24.2	
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Halfmoon Bay

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1					13.0			13.4				
2				7.6			12.1					
3												
4									20.1			25.0
5	19.3											
6		19.6	22.3									
7												
8											28.0	
9										25.4		
10	23.2											
11							17.5		17.0			
12											25.0	
13			22.6									
14					12.5							
15		22.5						16.0		25.1		
16												
17				15.6		14.6						
18												
19						11.3						
20												
21					14.2							
22										26.1		
23	22.2											
24							14.0					
25	16.7											
26			22.3									
27												
28		24.7	24.7		11.9			15.6				
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Halfmoon Bay

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				21.5			19.2					
2					17.5							
3												
4			19.2									
5												
6												
7					13.9							
8												
9												
10						18.0						
11				18.0								
12		22.8										
13						14.4						
14	27.6											
15					14.8							
16				14.4								
17												
18				9.8								
19			19.7									
20		21.9										
21												
22												
23					10.1							
24												
25												
26												
27												
28												
29												
30					12.5							
31												

APPENDIX A
SALINITY DATA

LOCATION: Sandy Point

1966
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												
7												16.8
8												
9											22.1	
10												
11												
12												
13												
14												
15												
16											24.6	
17												
18												
19										22.2		
20												
21												25.7
22												
23											23.8	
24												
25												
26										29.2		
27												
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Sandy Point

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4										26.9		
5												
6												
7												
8		16.5	18.1									
9								8.1				
10												
11												
12									11.6	28.7		
13				3.4								
14						8.0						
15											23.8	
16				6.7				10.2				
17										22.2		
18	12.5											
19	16.2											
20				11.7								
21			16.5									
22												
23												
24												
25							8.4					
26				5.4								
27					7.5			21.5				
28											24.1	20.8
29												
30												
31										21.0		

APPENDIX A
SALINITY DATA

LOCATION: Sandy Point

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1		6.1										
2				1.8	10.0		9.3					
3												
4	7.5								16.1			21.8
5												
6		19.8	21.1									
7			14.8									
8												
9							12.2			22.3		
10	17.5			12.8					17.7			
11											27.8	
12											26.5	
13			13.7									
14												
15		22.9			5.7					24.0		
16								8.0				
17						3.5						
18							8.0					
19						3.9			24.9			
20												
21												
22										29.4		
23	6.6							12.0		29.1		
24	17.1						8.0					
25												
26												
27			16.0									
28		10.7	14.7					17.6				
29					19.8							
30					7.5							
31							10.3					

APPENDIX A
SALINITY DATA

LOCATION: Sandy Point

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				25.5								
2												
3				10.8		18.7						
4		11.1	12.2									
5												
6												
7												
8					4.2							
9												
10				5.3		14.4						
11												
12		2.8										
13						8.4						
14	28.7											
15	27.0				6.6							
16												
17						24.2						
18												
19												
20		19.0	18.4									
21												
22												
23	7.1											
24												
25												
26												
27												
28												
29												
30					7.7							
31												

APPENDIX A
SALINITY DATA

LOCATION: Scofield Bayou

1966
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4												
5												
6												
7												17.9
8												
9											24.5	
10												
11												
12												
13												
14												
15												
16											26.5	
17												
18												
19										24.7		
20												
21												21.1
22												
23											26.9	
24												
25												
26										27.7		
27												
28												
29												
30												
31												

APPENDIX A
SALINITY DATA

LOCATION: Scofield Bayou

1967
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1												
2												
3												
4					23.4					25.7		
5												
6												
7												
8		20.9	19.3					10.2				
9												
10												
11												
12									16.0	28.9		
13				2.7								
14						10.2						
15											24.8	
16			15.6	10.0				11.8				
17										23.5		
18	20.5											
19												
20				9.8								
21												
22												
23												
24												
25	18.5						19.9					
26				5.8								
27												
28						7.9			22.2		24.2	20.8
29												
30												
31										22.3		

APPENDIX A
SALINITY DATA

LOCATION: Scofield Bayou

1968
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1		8.5										
2				3.1	14.3							
3												
4	10.5								16.2			
5												
6		19.5	25.1									
7			23.3									
8												
9							15.5			27.0		
10	19.2			12.9					20.9			
11											29.5	
12			14.6						20.2			
13												
14												
15		29.0			7.5					23.4		
16				12.5								
17						5.2						
18							11.5					
19						5.9			24.2			
20												
21												
22												
23	6.7							12.3		28.3		
24	17.1						10.7					
25												
26												
27												
28		15.0	17.7									
29												
30					20.0							
31												

APPENDIX A
SALINITY DATA

LOCATION: Scofield Bayou

1969
Salinity--in parts per thousand

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1				24.4								
2												
3				12.6		19.4						
4			15.1									
5												
6												
7												
8					4.6							
9												
10				8.5								
11												
12		7.9										
13	30.0					9.5						
14												
15												
16												
17						25.5						
18												
19												
20		29.9										
21												
22												
23	10.5											
24												
25												
26												
27												
28												
29												
30												
31												

Empire Floodgate
Flap Gate - Cathodic Protection

Calculations

1. The cathodic protection provided for the flap gate will be sacrifical metal type, as a supplement to 7.5 mils of vinyl paint, and is designed to protect both sides of the gate.

Protection Potential: - 0.85 volts measured to a copper - sulfate reference electrode

Anode: High Purity zinc, Hull type, anodes rated 335 amp. hrs./lb. @ 90% efficiency.

Solution Potential: -1.10 volts relative to a copper sulfate reference electrode.

Driving Potential: 0.25 volts relative to polarized cathode

Current Density: 0.0003 amps/sq. ft. for painted surfaces

FIG A-1

a. Skin Plate (Protected Side)

Gate Width = 84'
Gate Height = 29.5'
Protected Height = 29.5'

Surface Area = (84')(29.5')
= 2,480 sq. ft.

$I_{req'd}$ = (2,480 sq. ft.)(.0003 amps/sq. ft)

$I_{req'd}$ = 0.745 amps

$$\text{lbs of zn/yr.} = \frac{(.745\ \text{amps})(3760\ \text{hrs/yr})}{335\ \text{amp-hr/lb.}}$$

lbs of zn/yr. = 19.5 lbs/yr.

Total weight of anodes for 20 year protection of skin plate is 400 lbs.
To obtain good current distribution 40-12# anodes will be used. See plate VII-3.

FIG A-2

b. Structural (Flood Side)

Because of the symmetry of the gate one typical section, which is repeated nine times, shall be calculated. This typical section will be 9'-2 5/8" by 29.5', bordered by the center line of two vertical 36 WF 150's and the top and bottom of the gate.

See plate VII-3.

SKINPLATE

$A = (9.2')(29.5') \qquad = 272$ sq. ft

8-ST 8 WF 20's - 9.2' long

$A = 8(1.54 \text{ sq ft/ft})(9.2 \text{ ft}) \qquad = 113$ sq. ft.

FIG A-3

36 WF 150 - 45' long

$$A = (45\,ft)(8\,sq\,ft/ft) = 360\,sq.\,ft.$$

8 sq ft/ft => web and 1 flange

GIRDER WEB - 9.2' long

$$A = (9.2\,ft)(12\,sq\,ft/ft) = 110\,sq.\,ft.$$

GIRDER FLANGE - 9.2' long

$$A = (9.2\,ft)(4.7\,sq.\,ft./ft.) = 44\,sq.\,ft.$$

GIRDER STIFFENERS ETC.

$$A\ (\text{approximately}) = 75\,sq.\,ft.$$

***Total Area of Typical Section* = 974 sq. ft.**

$$I_{reqd} = (974\,sq.\,ft)(.0003\,amps/sq.\,ft)$$

$$= 0.29\ amps$$

FIG A-4

$$\text{lbs of zn/yr.} = \frac{(0.29 \text{ amps})(8{,}760 \text{ hrs/yr})}{335 \text{ amp-hr/lb}}$$

$$= 7.6 \text{ lbs zn/yr.}$$

Total for 20 yrs. = (7.6 lbs zn/yr)(20 yrs)

Total for 20 yrs. = 152 lbs

Due to the complexity of the structural members varied size anodes will be used. Approximately 198 lbs of zinc anodes will be mounted on each typical section as shown on the drawing.

FIG A-5