U. S. ARMY, CORPS OF ENGINEERS

LAKE PONTCHARTRAIN, LA. AND VICINITY

LAKE PONTCHARTRAIN BARRIER PLAN

DETAIL DESIGN MEMORANDUM NO. 7

CHEF MENTEUR PASS CONTROL STRUCTURE AND CLOSURE DAM

prepared for

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
NEW ORLEANS, LOUISIANA

by

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PERTINENT DATA

Location of Project	Southeastern Louisiana Orleans Parish, Chef Menteur Pass
Datum Plane	mean sea level
Control Structure	
Pass Section	Reinforced concrete base slab and piers forming gated bays and supporting a roadway bridge and gantry runway beams
Barrier Section	Reinforced concrete footings and piers forming bays obstructed with a strut wall and supporting a roadway bridge and gantry runway beams
Gates	Fixed wheel vertical lift (2 leafs)
Dimensions	Feet
Total length of Central structure	1118 abutment to abutment
Center to center spacing of piers	51.0
Clear distance between pier (Pass Section)	46.0
Base width of Pass	60.0
Channel	700 x 6,650
Elevations	Feet
Pass Section floor	-25.0
Top of raodway parapet	16.25
Roadway bridge	14.0
Gantry runway	14.0

PERTINENT DATA (continued)

Hydrologic Data

Temperature:	Maximum monthly	87.1°F
	Minimum monthly	43.0°F
	Average annual	69.7°F

Annual Precipitation:	Maximum	85.74 inches
	Minimum	31.07 inches
	Average	60.58 inches

Hydraulic Design Criteria - Tidal

Design hurricane - standard project hurricane (SPH)

Frequency	1 in 200 years
Central pressure index (CPI)	27.6 inches in mercury
Maximum 5-min. average wind	100 m.p.h.

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SECTION I - GENERAL

1. Project authorization. Public Law 298, 89th Congress, 1st Session, approved 27 October 1965, authorized the "Lake Ponchartrain, Louisiana and Vicinity," hurricane protection project, substantially in accordance with the recommendations of the Chief of Engineers in House Document No. 231, 89th Congress, 1st Session, except that the recommendations of the Secretary of the Army in that document shall apply with respect to the Seabrook Lock feature of the project.

2. Purpose. This detailed design memorandum presents the essential data, assumptions, and criteria used in the design of the principal features of the Chef Menteur control structure, closure dam and appurtenant channels. It is prepared for the purpose of developing the detail design and for facilitating review of subsequent construction plans and specifications.

3. Project location. The Lake Ponchartrain Barrier Plan of which the Chef Menteur control structure forms a part of, is located in southeast Louisiana in the parish of Orleans approximately 4700 feet southeast of the Louisville and Nashville Railroad where it crosses the Chef Menteur Pass waterway. (Plate I-1).

4. Local cooperation. The conditions of local cooperation pertinent to the Lake Ponchartrain Barrier Plan, of which the Chef Menteur control structure, the appurtenant channels and closure dam covered in this design memorandum are a part of are specified in the report of the Board of Engineers of Rivers and Harbors. This report was concurred in by the report of the Chief of Engineers, are as follows:

"...That the barrier plan for protection from hurricane floods of the shores of Lake Pontchartrain...be authorized for construction... Provided that prior to construction of each separable independent feature local interests furnish assurances satisfactory to the Secretary of the Army that they will, without cost to the United States:

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

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

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

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

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

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

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

5. <u>Previous Reports</u>. General information and basic data on the entire project are included in Design Memorandum No. 2 - General Design, dated August, 1967 and General Design Supplement No. 3, dated May, 1969.

6. <u>Datum Plane</u>. All elevations are in feet and refer to mean sea level, unless otherwise note.

7. <u>Description.</u> The project will consist essentially of the following features:

a. A control structure.

b. A control channel.

c. A closure dam.

8. The general plan of this segment of the project includes the construction of a 400 foot wide by 25 foot deep control structure with approach channels flaring at a 12.5 degree angle horizontally from the 400-foot width at the structure to a width of 700 feet, from which point a constant width of 700 feet will be maintained. See plates I-2 and I-3. The channel bottom will slope 1 on 10 from elevation - 25.0 at the structure to elevation - 40.0 from which point a constant channel bottom elevation of -40.0 will be maintained. Details and limits of construction of the control structure and channel are shown on plates I-2, I-3, and I-4.

9. The closure dam shown on plates I-5 and IV-3 will be constructed after the control channel, control structure, and navigation

structure and channel are in place. The earthen dam will be constructed of hydraulic sand fill with an impervious clay blanket and rip-rap slope protection on each side.

SECTION II - HYDRAULIC DESIGN

1. General. The purpose of the Chef Menteur Pass Complex is to lower hurricane tides in Lake Pontchartrain by reducing inflows from Lake Borgne. The tidal hydraulic analysis and design for the Chef Menteur Pass Complex protective structures are presented in Design Memorandum No. 1, "Hydrology and Hydraulic Analysis, Part II-Barrier", approved 18 October, 1967 which contains descriptions of the methods used in the tidal hydraulic design and covers essential data, climatology, criteria, and the results of studies which provide the basis for determining surges, routings, wind tides, runup, overtopping, and frequencies. The location of the Chef Menteur Complex is shown on Plate I-1 and the general plan on Plate I-2.

2. Design criteria. The hydraulic design computations were based upon the following criteria:

a. Design hurricane. The design hurricane for Chef Menteur Pass Complex is the Standard Project Hurricane (SPH) having a frequency of about once in 200 years, a central pressure index of 27.6 inches of mercury, a maximum 5-minute average wind velocity of 100 m.p.h. at 30 feet above water surface and a radius of 30 nautical miles from the center, moving on a track critical to the Chef Menteur Pass Complex at a forward speed of 11 knots. * ?? Why
A hurricane of lesser intensity would permit a lower grade for the crest of the Complex but would expose the protected areas

to hazards of life and property that would be disastrous in the event of a hurricane with the intensity and destructive capability equal to or greater than the SPH occurred.

b. Tracks. Plate II-1 is a map presenting a plot of actual and synthetic hurricane tracks in relation to the project. Parameters for the SPH, a hurricane condition from the Lake Borgne side, which corresponds to track F and a hurricane from the Lake Pontchartrain side which is the maximum reverse design condition are shown in Table II-1.

TABLE II-1
SPH Parameters

		Lake Borgne side case 1 1/	Lake Pontchartrain side case 6 1/
F	- Length of fetch (mi.)	5	1.3
U	- Windspeed (m.p.h.)*	88	90
swl	- Stillwater level (ft.m.s.l.)	12.8	11.5
d	- Average depth of fetch (ft.)	19.4	51.5

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

1/ See Table II-2

3. <u>Differential heads</u>. For a 10.5-foot stage in lake Borgne, a coincidental - 4.0-foot stage was determined on the Lake Pontchartrain side and for a 12.8-foot stage on the Lake Borgne side, a coincidental 4.0-foot stage was determined on the Lake Pontchartrain side of the gates. These stages correspond, respectively, to Moderate and Standard Project Hurricanes on moderate and severe hurricanes, was derived in order to determine the differential heads for any hurricane likely to occur. The minimum stages on the Lake Pontchartrain side, coincidental to maximum stages on the Lake Borgne side, were plotted at the frequency positions corresponding to the different hydothetical hurricanes. This plot provided a lower limit of points through which an envelope curve of minimum stages could be drawn. The maximum and minimum stage-frequency curves thus provided a means of determining coincident stages for any hurricane of an intensity equal to or less than the SPH. A study of these curves indicated that differential heads which fell between those actually computed were more critical than the less frequent differential caused by the SPH, and should be used for design of certain features. This procedure as illustrated on Plate II-2 was used to determine the differentials in both directions across the gates. Plates II-3 and II-4, respectively, illustrate stage-frequency curves for hurricanes following track C, and for hurricanes on any track producing higher stages on the Lake Pontchartrain side equal to or less than the SPH. Differentials produced by hurricanes which generate stages equal to or

greater than 9.0 feet may prevail for 15 to 20 hours. Two differential conditions obtained from Plates II-3 and II-4 were the minimum Lake Borgne level and the corresponding Lake Ponchartrain level (-4.0 and 9.0) and the minimum Lake Pontchartrain level and the corresponding Lake Borgne level (-5.0 and 9.0). These differentials have been used without considering the wave effect and as such are considered normal design cases. Table II-2 presents a tabulation of the water level conditions to be used in design.

TABLE II-2

DIFFERENTIAL HEAD CONDITIONS

	Water Elevation		Design Condition
Case	Gulfside	Lakeside	
1	+ 12.8	+ 4.0	Hurricane condition
2	+ 11.8	- 2.0	apply wave
3	+ 10.5	- 4.0	effect
4	+ 9.0	- 5.0	Hurricane condition
5	+ 5.0	+ 2.5	Omit wave effect
6	- 3.0	+ 11.5	Hurricane condition apply wave effect
7	- 4.0	+ 9.0	Hurricane condition
8	- 3.0	+ 2.5	omit wave effect

4. Hydraulic design designations. The differential head conditions shown in Table II-2 are developed from the stage frequency curves shown on Plates II-2, 3 and 4.

a. Direct hurricane. Cases 1, 2 and 3 are hurricane conditions providing increased levels in Lake Borgne, the hurricane wind effects are considered as acting and hence dynamic wave loadings are also included.

b. Reverse hurricane. Case 6 is a hurricane condition producing an increased level in Lake Pontchartrain, the hurricane wind effects are considered as acting and hence dynamic wave loading are also included.

c. Normal condition. Cases 4, 5 and 8 are hurricane conditions which produce an increased level in Lake Borgne. The wave effect is omitted. Cases 5 and 8 are equipment operation conditions and therefore are not applied as design loadings for the Complex structure.

d. Reverse normal condition. Case 7 is a hurricane condition producing an increased level in Lake Pontchartrain, the wave effect is omitted.

5. Waves. Differential head conditions as listed in Table II-2 present the hydrostatic loading condition that the Chef Menteur Comples will be subjected to. Cases 1, 2, 3 and 6 are hurricane conditions that include in addition to the loading due static differential head also a hydrodynamic loading due to wind generated waves.

a. Parameters. The parameters which determine wave characteristics are fetch length, windspeed, duration of wind, and the average depth of water over the fetch. In determining the design wave characteristics, it was assumed that steady state conditions prevail; i.e., the windspeed is constant in one direction over the fetch and blows long enough to develop a fully risen sea. The windspeed U is an average velocity over the fetch length F and was obtained from the isovel patterns for the synthetic hurricane chosen as being critical to the location of interest. The average depth of fetch d is the average depth of water as shown by the charts and maps for the area, plus the increase in water elevation caused by wind. The data used to determine the design hurricane wave characteristics for cases 1 and 6 is in Table II-1.

The wave height parameter (H_s) and the wave period (T) were determined from the above data using curves which are found in Coastal Engineering Research Center Technical Report No. 4, June 1966.

b. Characteristics. The deepwater wave length L_o was determined from the equation: $L_o = 5.12\ T^2$. The equivalent deepwater wave height H_o^1 was determined from table D-1 of the above reference, which relates the relative depth d/L_o to H_s/H_o^1. Wave characteristics for the hurricanes which are pertinent to the design of the structures are shown in Table II-3. It is to be noted that the wave characteristics for waves approaching the control structure in the channel differ from those approaching the control structure outside of the deep channel section. This latter zone has been designated as over the marsh.

c. Classification. For a specific set of parameters, waves attacking a structure may be classified in one of three categories; a) nonbreaking or standing wave, b) breaking wave and c) broken wave. The hydrodynamic effects differ in each wave classification. A wave will be classified as nonbreaking or standing if the still water depth in the zone under consideration exceeds a determined value designated as the breaking depth. The breaking depth is unique for each design wave. A wave begins to break when the still water depth becomes less than the breaking depth. The wave continues to break over a distance equivalent to 6 or 7 times the wave height. The latter figure has been used in this memorandum. Beyond this distance the wave is considered broken. Plate II-5 shows a half plan of the control structure and three profiles through the barrier section for which wave classifications for each of the design cases have been determined. The still water depth in the channel exceeds the breaking depth in each design case hence the pass section will be subjected to standing waves for all cases.

6. Hydraulic loading conditions. Plates II-6 through II-9 inclusive show the hydraulic loading conditions for cases 1, 2, 3, 4, 6 and 7 on the pass section and at various locations along the barrier section. The loading diagrams on both sides of the structure are shown as well as the net unbalance force diagram.

a. Hydrostatic loads. Cases 4 and 7 are design conditions where waves are not considered therefore loadings on both sides of the structure are hydrostatic. The hydrostatic pressure varies uniformly

(64 lbs/ft.2/ft.) from the still water level elevation to the base of the structure at the section under consideration. In cases 1, 2, 3 and 6 on the downwind side of the structure, a hydrostatic loading is applied as stated for cases 4 and 7.

b. Hydrodynamic loads. Cases 1, 2, 3 and 6 are design conditions in which waves are considered, therefore, on the windward side of the structure a wave pressure is additive to the hydrostatic pressure caused by the still water level. The severest loading occurs when the crest of the clapots strikes the structure. This is an instantaneous loading and is shown on Plates II-6 through II-9 inclussive. Standing waves produce a loading that is additive to the hydrostatic loading over the entire height of the structure. Broken waves produce a loading that is additive to the hydrostatic loading over a limited height of the structure.

c. Transverse loads on piers. A transverse loading on the pass section and barrier section piers i.e., a loading normal to the loading discussed in a and b can occur when the waves on each side of the pier are not in phase. In consideration of this occurrence a net unbalance loading equivalent to the wave pressure is applied transverse to the piers. This loading is applicable in Cases 1, 2, 3 and 6.

TABLE II-3

WAVE CHARACTERISTICS - LAKE PONTCHARTRAIN CONTROL STRUCTURE

		In Channel			
		Gulf Side			Lakeside
		Case 1	Case 2	Case 3	Case 6
swl	Still water level (ft.)	12.8	11.8	10.5	11.5
H_s	Significant wave height (ft.)	7.32	6.68	5.86	6.00
T_s	Wave period (sec.)	6.70	6.4	6.1	9.60
L_o	Deep water wave length (ft.)	229.84	209.72	190.52	471.86
d/L_o	Relative depth	0.08419	0.08774	0.08976	0.1091
H_s/H'_o	Shoaling	0.9491	0.9448	0.9425	0.9263
H'_o	Deep water wave height (ft.)	7.71	7.07	6.22	6.48
d_b^o	H'_o breaking depth (ft.)	9.30	8.50	7.56	10.51
H_{10}	Average of highest 10% of all waves (ft.)	9.30	8.48	7.44	7.62
d_b10	H_{10} breaking depth (ft.)	10.48	9.60	8.52	11.67
H_1	Average of highest 1% of all waves (ft.)	12.22	11.16	9.79	10.02
d_{b1}	H_1 breaking depth (ft.)	12.58	11.53	10.23	14.00

		Over Marsh			
		Gulf Side			Lakeside
		Case 1	Case 2	Case 3	Case 6
swl	Still water level (ft.)	12.8	11.8	10.5	11.5
H_s	Significant wave height (ft.)	5.3	4.67	4.1	4.65
T_s	Wave period (sec.)	5.6	5.2	4.9	5.25
L_o	Deep water wave length (ft.)	161	138.45	122.93	141.11
d/L_o	Relative depth	0.07640	0.08162	0.08135	0.0744
H_s/H'_o	Shoaling	0.9601	0.9525	0.9529	0.9634
H'_o	Deep water wave height (ft.)	5.5	4.90	4.30	4.83
d_b	H'_o breaking depth (ft.)	6.55	5.80	5.11	5.79
H_{10}	Average of highest 10% of all waves (ft.)	7.00	5.93	5.21	5.91
d_b10	H_{10} breaking depth (ft.)	7.70	6.59	5.81	6.59
H_1	Average of highest 1% of all waves (ft.)	9.20	7.80	6.85	7.77
d_{b1}	H_1 breaking depth (ft.)	9.24	7.91	6.97	7.90

7. Sequence of gate operation.

a. Closing. Two lifting hooks are provided at the top of each gate section for handling with the lifting blocks. To close the gate the following procedure should be followed:

(1) Connect the lifting blocks to the top section first, liftup from storage slot and attach to the bottom section by an air pressure operated connecting device (Plate V-9).

(2) Lift the gate as one piece and remove the dogging devices;

(3) Lower the gate into slot;

(4) Disconnect lifting blocks;

Closure of the control structure will begin with gate number 8 (located on the opposite end of the structure from where the gantry crane will be berthed) and proceed sequentially from gate number 8 to gate number 1 (immediately adjacent to the gantry berthing location). Differential heads during the closure operation will generally not be so cirtical as those for opening.

b. Reopening. To reopen the gate the following procedure should be followed:

(1) Connect the lifting blocks to the top section and lift the gate as one piece until the bottom of the first wheel of the bottom section lines-up with the deck;

(2) Close the dogging devices;

(3) Lower-down and set the gate on the dogs;

(4) Disengage the top section from the bottom section;

(5) Lift the top section and place into storage slot;

(6) Disconnect lifting blocks;

Control structure opening will begin at gate number 3 and proceed sequentially from gate number 3 to gate number 8. Gates 2 and 1, in that order, will then be opened. This sequence is important to the safety of the structure and rip-rap protection.

8. Diversion of flow during construction of closure dam. In order to maintain the natural processes of Lake Pontchartrain and navigation between Lake Pontchartrain and Lake Borgne, the closure dam in the Chef Menteur Pass cannot be constructed until the control structure, navigation structure and approach channels for both structures have been completed. While the closure is being made the flow between the juncture of the three channels (Chef Menteur Pass, control channel and navigation channel) and Lake Borgne will be distributed between the three channels dependent upon the relative water level in the two lakes and the relative flow resistance of the channels. It is necessary that construction of the closure dam be scheduled for completion during a time of year in which the chances of hurricane occurrence is slight or nil. Should a hurricane occur during the closure operation, it is probable that any completed embankment would be washed out. On the premise that no hurricane will occur, it is assumed that both the control structure and the navigation structure will remain open during the closure operation.

9. Hydraulic Computations. The direction and quantity of flow through the Chef Menteur Pass is principally influenced by tides in the Gulf of Mexico and wind on Lake Pontchartrain, Lake Borgne and shallow coastal waters. In the design of the closure dam, it is important to derive design criteria for the expected velocity through the Chef Menteur Pass while the closure is being made. To accomplish this objective, data collected for a prototype study of Lake Pontchartrain, Louisiana, and vicinity has been used. This data was collected by the U.S. Army Engineer District, New Orleans, Louisiana, and published in the report, "Hurricane Study, Lake Pontchartrain, Louisiana and Vicinity - Prototype Data Collection Program for Model Study of Lake Pontchartrain, Louisiana and Vicinity" dated July, 1962. Interpreted prototype data published in the report "Effects on Lake Pontchartrain, Louisiana of Hurricane Surge Control Structures and Mississippi River Gulf Outlet Channel - Hydraulic Model Investigation", Technical Report No. 2-636, U.S. Army Engineer Waterway Experiment Station, Corps of Engineers, Vicksburg, Mississippi, dated November, 1963, has also been used in this investigation. The measurements taken at station M-1 located approximately two and one-quarter miles from U.S. Highway 90 toward Lake Pontchartrain (measured along course of the waterway) have been used for this study.

10. From plates 7 and 10 of the model study report the maximum velocity during a typical spring tide is 3.5 f.p.s. and during a typical

neap tide 1.9 f.p.s. The maximum velocity for a mean tide is assumed to be 2.7 f.p.s. and a mean velocity for the mean tide of 1.73 f.p.s. This is reasonably substantiated by the mean of random measurements taken by prototype data collection program, 1.59 f.p.s. To allow for reasonable deviation from the typical tide conditions an average design velocity of 2.0 f.p.s. has been adopted. Since the rate of change of velocity with time is normally low, it is considered reasonable to approximate the flow condition using an average velocity which is assumed to be constant with respect to time. Based on the average velocity at station M-1, the average discharge through the existing Chef Menteur Pass has been calculated as 83,400 c.f.s. To investigate flow conditions under extreme conditions (not hurricane condition) another analysis was made assuming 4.0 f.p.s. through section M-1. See plates II-10 and II-11.

11. Case 1 assumes a water level at Lake Borgne of 0.94 feet with 83,400 c.f.s. flow from Lake Pontchartrain to Lake Borgne. By the step method of non-uniform flow analysis, the water level at Lake Pontchartrain corresponding to this flow was calculated to be 1.738 feet. These lake levels were held constant and a series of non-uniform flow calculations were made to determine for each channel the water level at their junction for various quantities of flow. For the Lower Pass (Chef Menteur Pass from the junction to Lake Borgne), similar calculations were made for various conditions of closure as indicated by the

elevation of top of closure. The results of these calculations have been plotted graphically on plate II-10. Althrough a sign convention was used designating flow away from the junction (+) and flow toward the junction (-), both directions of flow are plotted on the same side of the zero axis. Before any construction only two channels meet at the junction and the water level and flow is indicated by the intersection of the lower pass (LP) and the upper pass (UP) since the water level at the junction must be equal. By plotting a curve whose abscissa represents the horizontal algebraic summation of the flow in the UP, control channel and navigation channel, its intersection with LP at any stage of closure indicates the flow through LP and the water level at the junction. At this water level the flow in all other channels can be obtained from their respective curves.

12. Case 2 assumes a water level at Lake Borgne of 1.500 feet with 81,500 c.f.s. flow from Lake Borgne to Lake Pontchartrain. The water level at Lake Pontchartrain corresponding to this flow was calculated to be 0.94 feet. The curves for various channels and conditions were calculated and plotted similar to the curves for Case 1.

13. Case 3 assumes a water level at Lake Borgne of 0.94 feet with 164,500 c.f.s. flowing from Lake Pontchartrain to Lake Borgne. The water level at Lake Pontchartrain corresponding to this flow was calculated to 3.935 feet. Case 4 assumes a water level at Lake Borgne of 4,000 feet with 165,000 c.f.s. flowing from Lake Borgne to Lake

Pontchartrain. The water level at Lake Pontchartrain corresponding to this flow was calculated to 1.0211 feet.

14. Case 1 and 2 have been used for evaluating the flow conditions affecting the placement of closure fill; however, Case 3 and 4 present extreme conditions which could effect closure construction and indicate maximum flow which might be expected through the control structure after the closure is completed. The graphical solution of flow calculations for Case 1 and 2 are presented on plate II-10 and Case 3 and 4 are presented on plate II-11.

15. Velocities and scour protection. Computed velocities in the control channel are less than 2.0 f.p.s. except in the area immediately adjacent to the control structure with the gates open. Riprap protection on both sides of and adjacent to the structure is designed for a differential head condition represented by a stage of +2.5 feet M.S.L. on the Lake Pontchartrain side of the structure A and -3.0 feet M.S.L. on the Lake Borgne side, the maximum reverse head under which the gates operate for structural and mechanical design. This differential head will produce a maximum velocity at the structure of 13.4 f.p.s.; derived from the equation V = 0.7 (2 gH) 1/2, where V = velocity and H = head (5.5 feet). Riprap protection designed in accordance with the procedures outlined in EM 1110-2-1601 as supplemented by ETL 1110-2-120 will require the

specification of a quarry stone with a weight of average size stone (W50) of 600 pounds for protection of the approach channel bottom and slopes (See appendix III, page III 31). The U.S.B.R. curve presented on the above reference was used to determine the average stone weight.

16. Approach channel design. The approach channel will be 700 feet wide with a bottom elevation of 40.0 and with 1 and 3 side slopes and will extend approximately 2/3 of a mile northward of the control structure to gain the Chef Menteur Pass and approximately 1/2 of a mile southward to Lake Borgne. The chanel transitions at an angle of 12.5 degrees to a bottom width of about 403 feet at the control structure. The approach to the control structure on each side slopes from elevation - 40.0 to -25.0 feet msl in a distance of 150 feet adjacent to the structure. The general design procedures and recommendations for transition design outlined in EM 1110-2-1601 as supplemented by ETL 1110-2-120 are incorporated in the approach channel design.

17. Hydraulic instrumentation. (Hydraulic Instrumentation will be furnished by New Orleans District).

18. Model studies. In order to determine the effects on the salinity and hydraulic regimens in Lake Pontchartrain and adjoining lakes of hurricane barriers in Chef Menteur and Rigolets and of the Mississippi River - Gulf Outlet Channel, a comprehensive model study was conducted.

19. The Lake Pontchartrain model was a fixed-bed model, constructed to scales of 1:2000 horizontally and 1:100 vertically, in which were reproduced the western portion of Mississippi Sound (beginning at Pass Marianne), Lake Borgne, Lake Pontchartrain, and Lake Maurepas, with the connecting waterways of Chef Menteur, Rigolets, and Pass Manchac. Also included with the Mississippi River-Gulf Outlet Channel extending from Lake Pontchartrain, through the Inner Harbor Navigation Canal and a portion of the Intracoastal Waterway, and thence southeast through the marshes into Breton Sound past Gardner Island and Grace Point. The model was equipped with necessary appurtenances for the accurate reproduction and measurement of tides, tidal currents, salinity intrusion, fresh water inflow, and other significant prototype phenomena.

20. Model verification tests indicated that the model hydraulic and salinity regimens were in satisfactory agreement with those of the prototype for comparable conditions. It, therefore, can be assumed that

model provided quantitative answers concerning the effects of the proposed structures on the hydraulic and salinity regimens of the lake system.

21. The main conclusions drawn from an analysis of the results of tests were:

(1) Construction of gated structures in Chef Menteur and Rigolets, which would reduce the cross-sectional area to 25 percent of the original cross-sectional area, would cause no appreciable change in the salinity of Lake Pontchartrain. Further, construction of the proposed structures would raise the average water-surface elevation in Lake Pontchartrain 0.1 feet with normal freshwater inflow into the lake; with the Bonnet Carre Spillway discharging the design flow of 250,000 c.f.s. into the lake, the average water-surface elevation would be raised 0.4 feet.

(2) Tests of complete closure of all structures (in Chef Menteur and Rigolets Passes and in the Gulf Outlet Channel) for a 2 week period simulating that between May 23 and June 5, 1959 (the time of passage of a hurricane over the area) indicated a relatively minor reduction of about 500 ppm in the average salinity of Lake Pontchartrain. Upon reopening of the structures, return to normal salinity was fairly rapid (approximately 11 weeks.) The maximum increase in water-surface

elevations in Lake Pontchartrain (at West End) resulting from complete closure was 1.2 feet, and this maximum increase was attained just 1 day before the reopening of the structures.

(3) Operation of the Bonnet Carre Spillway discharging as much as the design flow with both the Gulf Outlet Channel connected and hurricane surge control structures installed in Chef Menteur and Rigolets would raise the high-water elevation in Lake Pontchartrain to a maximum of 1.4 feet m.s.l.

SECTION III - FOUNDATION DESIGN

1. General. The Chef Menteur Complex control structure will be constructed on the alignment of the barrier embankment between stations 131 + 59.83 and 143 + 59.83. The control structure and facilities will consist of eight concrete gate bays supported on untreated timber piling; approach channels flaring at 12.5 degrees angle horizontally from 408 feet width at the structure to a bottom width of 700 feet, the channel bottom will slope 1 on 10 from elevation -25.0 at the structure to elevation -40.0 from which a constant channel bottom elevation of -40.0 will be maintained; the 1 on 10 approach slopes will be protected with rip-rap on 12-inch shall blanket; connection to barrier embankment shall be made with concrete bulkhead wall supported by concrete, pile supported piers and a transition section to the embankment of cantiliver I-type flood wall. For the general control structure plan see plate I-3, and for ele-vations and typical sections see plate I-4. After completion of construction of the control structure and the navigation structure (not included in this detail design memorandum), the existing Chef Menteur Pass will be closed with an earthen dam; the dam to be constructed of hydraulically placed sand fill with an impervious clay blanket. The embankment will be protected against wave action by rip-rap on a 9" shell.

2. Previous investigations. Lake Pontchartrain, Louisiana and Vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 General Design, Supplement No. 3, Chef Menteur Complex, includes soil investigation and foundation studies, as well as a description of the geology of the area. General type borings 3M, 4M, 5M, 6M and undisturbed type borings 1MU, 2 MU, 10 MU, 11 MU were made in February and March of 1967, and April and May of 1968 for this report. The location of these borings, as well as subsequent borings, are shown on plate I-5. General type borings 30 M through 39 M were taken in May and June of 1968 in the borrow areas proposed for obtaining fill for the Chef Menteur Pass closure dam and other closures of the existing G.I.W.W. and Marque Canal. The location of these borings are shown on plates III-9 and III-25. Test data for boring 1 MU is shown on plate III-6; test data for borings 10 MU is and 11 MU is shown on plate III-7.

3. Field exploration. Additional soil borings were made at the structure site to complete the subsurface investigation for this report. Undisturbed boring no. 14 MU was taken on the center axis of the control structure approximately 400 feet east of the control channel centerline. Undisturbed boring no. 13 MU was taken approximately 400 feet west of the control channel centerline. Both undisturbed borings extended 100 feet to elevation -98 into the stiff clay and siltry sand stratas of the Pleistocene formation. Three

additional general type borings (nos. 42M through 44M) were made at the control structure site. The structure borings extended to a depth of 100 feet below ground surface to approximate elevation -98. Split-spoon driving resistances were obtained in the sand stratas in the two undisturbed borings. The logs of the soil borings are shown on plates III-4, III-5, and III-10.

4. Laboratory tests. Visual classification and water content determinations were made on the soil samples obtained from the borings. Consolidation (C) tests, inconsolidated-undrained (Q), consolidated-drained (S) shear tests were performed on representative soil samples encountered in the undisturbed borings. Liquid and plastic limits were determined for all cohesive samples on which consolidation and shear tests were performed. Grain size gradation tests were performed on representative foundation sand samples. Range of gradation sand in Chef Menteur borrow area is shown on plate III-9. The location of the undisturbed soil sample tested is shown adjacent to the boring logs on plates III-4, and III-5. Test data are shown on plates III-6 and III-8.

5. Foundation conditions.

a. Control structure. The subsurface at the control structure site consists of a deposit of recent soils approximately 45 feet thick overlaying Pleistocene soils. At the surface a layer of peat (Pt) and very soft, highly organic fat clay (CHO) approximately

11 feet thick exists with water content ranging from 177 to 748 percent. Beneath the (Pt) and (CHO) strata is a very soft to soft fat clay (CH) with 2 to 3 feet thick intermittent layers of silty sand (SM) and fine sand (SPF) with a total thickness of approximately 34 feet overlying stiff to very stiff clays of this Pleistocene formation. This upper clay layer of the Pleistocene formation varies considerably in thickness (7 feet to 24 feet) and contains intermittent thin layers of silt (ML), lean clay (CL), silty sand (SM) and fine sand (SPD). Beneath the upper clay layer the soil is predominately pervious material, silty sand (SM) and fine sand (SPF) with intermittent thin layers of stiff fat clay (CH), lean clay (CL) and silt (ML). For purpose of foundation design and slope stability analyses the composite soil profile and design strengths shown by plate III-10 has been selected based on boring logs and test data for all borings taken in vicinity of the control structure and is considered appropriate for design purposes. Because of the variation in the depth of the pervious strata, the design soil profile for stability and seepage analysis has been modified as shown by plate III-10.

b. Closure dam. A discussion of the geology, and foundation conditions for the Chef Menteur Pass closure dam is presented in the general design memorandum supplement. The locations of borings directly affecting the design of closure dam are shown on plate I-5 of this report and the results of soil tests of undisturbed borings on plates III-9 and III-25. The location of borrow area borings and their logs are presented on plate III-26 of this report and plate 33 of General Design Supplement No. 3. In the natural formation of the Chef Menteur Pass, the present geometry of the cross section was developed by the scouring action of the water caused by tidal fluctuations in the Gulf of Mexico. The waterway has cut through the upper stratas of Marsh formation and Intradelta Complex formation overlying the ancient Pleistocene formation. The thalweg of the waterway generally follows the top of the Pleistocene except in bends where additional depth of scouring has occurred. At the proposed dam location, the depth of the thalweg is nearly constant at an elevation of approximately -53.0 feet. In the vicinity of the thalweg, the proposed dam will rest on the Pleistocene formation which is composed of stiff to very stiff fat clay (CH) approximately 7 feet thick overlying a deep stratum of fine sand (SPF). As the closure dam approaches the waterway banks, the soft fat clay (CH) overlying the Pleistocene formation comes into the foundation profile. Along the banks of the waterway, the subsurface consists of 10 to 12 feet of very soft organic clay (CHO) and peat (Pt) overlying 35 to 40 feet of very soft to soft fat clay (CH) which are underlain by the Pleistocene formation.

6. Control structure.

a. Temporary protection and spoil retention dikes. Protection from flooding will be provided during construction for a maximum stage, outside of the protected area, of elevation 6.0. The structure excavation spoil area enclosed by retention dikes will provide protection along the west side of the structure site northward of the barrier embankment. Spoil sections with connecting temporary protection dikes, located as shown on plate I-2, will complete the required protection. The spoil retention dikes will be constructed with material cast from adjacent borrow pits located within the spoil area and from the structure excavation. The temporary protection dikes will be constructed with material obtained from the structure excavation.

b. Structure excavation. The structure excavation will be accomplished by hydraulic dredge to the plan and section shown on plates III-17 and III-18. The remaining excavation to final grade will be accomplished in dry after the excavation is dewatered.

(1) The access flotation channel into the gate area will be excavated to elevation -5.0, with bottom width of 40 feet, and 1 on 3 side slopes.

(2) After hydraulic dredge operations are completed, the access flotation channel will be closed with an earthen closure and the excavation unwatered.

(3) The approach channel excavation will be accomplished in the wet after the control structure is completed and water allowed to enter the excavation. The hydraulically excavated material for construction excavation and approaching channel above approximate elevation -10 ft. M.S.L. will be placed within the spoil area; material below -10 feet M.S.L. will be used for levee construction.

(4) The shell blanket and rip-rap protection for approach bottom slope as shown on plate I-2 will be placed under water on the bottom of the approaches.

c. Stability of slopes.

(1) During construction. The stability of the excavation retaining dikes and temporary protection dikes was determined by the method of planes based on the water conditions and (Q) design shear strengths shown on plate III-20. The stability was investigated for various depths of failure in the foundation and factors of safety with respect to shear strength were determined for various assumed failure planes. The critical failure surface and their corresponding data, including the critical vector analysis, are shown on plate III-20.

(2) Following construction. The stability of the final section of the channel and tie-in levee was determined by the method of planes based on critical water conditions and (C) design shear strenght, as shown on plate III-21. The critical failure surfaces and their corresponding data are shown on plate III-21.

d. Construction dewatering and hydrostatic pressure relief.

(1) Prior to dewatering the initial hydraulic dredge excavation, deep wells will be installed around the excavation inside the temporary protection and spoil retention dikes. The wells will be pumped so as to lower the piezometric head in the underlying pervious strata. The design and operation of the wells are described in subsequent paragraphs. The water in the initial excavation will be pumped out concurrently with the lowering of the piezometric head in the underlying pervious strata. The operation of the deep well system and the removal of water from the excavation will be controlled so that the piezometric head between wells will be at least five (5) feet below the water surface in the excavation. After the water is removed from the excavation, the piezometric head in the previous foundation stratum will be maintained at least five (5) feet below the bottom of the ultimate excavation.

(2) Several thin strata of impervious soil exist above the bottom of the excavation and will outcrop on the slopes; however, because the individual strata are relatively thin and not highly pervious, the quantity and rate of flow seeping out at the slope will not create a serious erosion or stability problem. It is not considered necessary to provide any special dewatering facilities for this condition.

(3) Pressure relief during construction. The soil borings taken at the site show considerable variation in depth to the top of the pervious strata and its continuity with the pervious strata underlying the Chef Menteur Pass is not clearly established; however, because of the depth of the channel and its close proximity to the control structure excavation, it is reasonable to assume that the hydrostatic pressure in the underlying pervious stratum closely reflects the water level in the Chef Menteur Pass. Although the bottom of the structure elevation (-33.0 MSL) does not extend into the pervious strata, the hydrostatic uplift pressure below the clay foundation would necessitate a pressure relief system to prevent heaving and blowouts at the bottom of the excavation during construction. It is considered necessary that the hydrostatic head in the pervious strata be kept at least five (5) feet below the bottom of the excavation. A reduction in hydrostatic head of 44 feet for a maximum water level in the Chef Pass of 6.0 MSL will be required to meet this criteria. The required capacity of the deep well pressure relief system required was estimated

based on a flow net drawn with the following assumptions:

(a) The thickness of the pervious strata are uniform throughout the area, from elevation -54.0 to 120.0.

(b) The coefficient of permeability, 109 x 10^{-4} CM per SEC was obtained from field pumping tests made in similar fine sand for the Rigolets Lock. The flow net is presented on plate III-19. It is estimated that the required total quantity to be pumped from the pressure relief system fully penetrating the aquifer will be about 15.3 gallons per foot of drawdown. On this basis, the total required pumping capacity of the system will be approximately 675 gpm.

(4) Relief wells will be located as shown on plate III-17. The well will consist of 10-inch ID commercial slotted stainless steel well screen extending from approximate elevation -50 to elevation -112 and a 10-inch ID galvanized iron riser pipe. Details of the well screen and gradation of the filter gravel around the screen section are shown on plate III-19.

(5) Piezometers will be located as shown on plate III-17 and will consist of 1-1/4 inch No. 18 slot brass commercial well point screen 2-feet long with 1-1/4 inch plastic riser pipe to be installed in the sand aquifer to provide data on drawdown during dewatering and construction. Details of piezometer installation are shown on plate III-19.

(6) After the excavation is dewatered, ditches, sumps, and pumps will be required in the lower part of the excavation to remove surface runoff and to provide surface drainage.

(7) Because of the close proximity of the site and the similarity of soil characteristics, the results of the field pumping tests taken at the proposed site of the Rigolets lock has been used for the design of the pressure relief system for the Chef Menteur project. The field pumping test was performed at the proposed lock site during the period 21-29 April, 1969 to determine insitu horizontal permeability of the foundation sands. The investigation consisted of installing a test well and ranges of open-type piezometers extending radially from the well. The test well extended in depth to elevation -64.4 feet MSL and the center of the piezometer screens were set a approximate elevation -35.0 MS. The field investigation consisted of pumping the well at two different drawdowns, and reading the piezometers.

e. Permanent seepage and hydrostatic uplift control. During a hurricane when the gates of the control structure are closed, a differential hydrostatic head will exist between the two sides of the control structure; from plate II-7 the maximum differential is 14.5 feet. From the soil borings, it appears likely that the hydrostatic head in the deep pervious strata below the proposed control structure and control channel will respond to the water level in the Chef Menteur

Pass, but not to the water level in the control channel since the control channel excavation does not extend into these pervious strata. On this premise and with the assumption described for pressure relief during construction, the flow net shown on plate III-19 was drawn to estimate the critical uplift pressure beneath the control structure and the control channel. The flow net indicates that the uplift pressure due to a differential head of 14.5 feet in the Chef Pass will be on the order of 312 p.s.f. The minimum depth of impervious clay between the top of the pervious strata is sufficient to resist this uplift pressure with a factor of safety not less than 1.5. Since uplift pressure below the control structure and channel is not critical, no permanent hydrostatic uplift control is recommended.

f. Pile foundation.

1) Capacities. Allowable compression and tension capacities versus pile tip elevations are shown on Plate III-11. The allowable loads were computed using both the (Q) and (S) shear strengths. The strength selected for each curve was the one requiring the greatest pile penetration for any given load. 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.

Plots for tension piles on this plate are for allowable loads determined by foundation soil strength. The actual allowable loads must be reduced so as not to exceed 5,000 pounds times the number of TECO connectors on each timber pile or so as not to exceed the capacity of the H-pile connectors.

(a) Timber piles. The allowable load in compression selected for the twelve inch average diameter piles to resist static external loadings is 40,000 pounds. The allowable loads in tension selected for the piles is 24,000 pounds for piles beneath the pass section pier strips and 10,000 pounds for piles beneath the pass section bay strips. This reduced load beneath the bay strips requires the use of only two TECO Ten-con tension pile connectors on each pile for purposes of anchoring them to the footing. Five such connectors are required on all pier strip piles. The allowable loads in tension were established by the number of tension connectors used rather than by the foundation soil strength. The number of connectors selected was established by the maximum tension load computed and by the 5,000 pounds allowable load per connector. A minimum of two connectors must be installed on every pile.

The allowable loads selected for transient loading cases (1, 2, 3, and 6) and for the construction case (9) were 33-1/3% greater than the corresponding allowable loads for static loading cases (4 and 7)

(b) Steel H-piles. The allowable loads selected for the twelve inch piles to resist static external loadings, including negative skin friction, are as shown below:

	Allowable Loads (in Kips)	
Piers	Compression	Tension
Abutment	220	80
Barrier Piers 1 thru 5	180	10
Barrier Pier 6	180	30

The allowable loads selected for transient loading cases (1, 2, 3, and 6) and for the construction case (9) were 33-1/3% greater than the corresponding allowable loads for static loading cases (4 and 7).

The levee fill and the backfill above the abutment and barrier pier footings causes a net increase in the unit load now existing at the elevation of the base of each of these footings. This increase in unit load will cause a settlement of the foundation material beneath these footings. Because the steel piles will be driven into a deep silty sand layer, they will settle much less than the upper foundation soil. Thus, the settling foundation material will cause a down drag force, or negative skin friction, on the steel piles. The negative skin friction, to be applied only to compression piles, was computed separately for each pier affected as follows:

(1) The net increase in unit loads at the base of each footing was computed using the long term static water level.

(2) The effect of this net increase in unit load was computed at various depths beneath the centerline of the footing by applying the Boussinesq theory adapted to distributed loads. The effect of the increase along the centerline of the footing was assumed to apply beneath all portions of the footing as well.

(3) The negative skin friction was computed using the method described by Terzaghi and Peck in the first edition of Soil Mechanics in Engineering Practice. In using this method, Q" was assumed equal to Q_{max}". Soil resistance along the perimeter of the pile cluster was computed using the (S) shear strengths.

(4) Foundation settlements to determine negative skin friction were computed by applying test results from the undisturbed borings to the usual settlement computation methods. All settlement was assumed to be caused only by consolidation of the clay strata; it was assumed the silty sand layers would not compress.

(5) All piles were assumed to settle 1/2 inch. Full negative skin friction was assumed to be developed everywhere above the elevation at which the settlement of the foundation soil relative to the pile was 1/2 inch (one inch total settlement). The negative

skin friction was assumed to vary linearly to zero at the elevation of zero relative displacement (1/2 inch total settlement). It was assumed that no foundation settlement occurred below elevation -70.0, the top of a deep silty sand layer.

(6) Positive skin friction was assumed to vary linearly from zero at the point of zero relative displacement to its full value of 100% at elevation -70 and below.

The negative skin friction is summarized below:

Pier	Elevation of 100% Neg.Skin Friction	Elevation of Zero Neg.Skin Friction	Negative Skin Friction/Pile
Abutment	-64.0	-67.0	93.0 (Kips
Barrier Piers 1 and 2	-64.0	-67.0	126.5
Barrier Piers 3 and 4	-64.0	-67.0	123.0
Barrier Pier 5	-64.0	-67.0	119.0
Barrier Pier 6	-61.0	-65.5	104.0

2) Subgrade moduli. The subgrade modulus is a required input for the method of pile analysis used. Its value was computed using the expressions shown on Plate III-11, and a plot of subgrade modulus versus foundation elevation is shown thereon. The subgrade modulus in clay strata was developed using the (Q) shear strength results; the modulus in silty sand strata was assumed to be equal to the value computed for the clay layer immediately above. The value selected for design was the average modulus in the five feet of foundation immediately beneath the base of the structure. These are summarized below:

Structure	Design Value of Subgrade Modulus (in psi)
Pass Section	80
Barrier Pier 6	90
Barrier Piers 3, 4 and 5	80
Abutment Pier and Barrier Piers 1 and 2	65

3) Field pile tests. Pile lengths to be used for construction will be determined from field pile tests performed within the control structure excavation. Vertical piles of three different lengths will driven for each combination of structure and pile as shown below. The pile of intermediate length will be tested first. If test results show that the pile can carry twice the design load, the pile will be tested in tension. If the intermediate length pile fails before the required capacity is attained in compression, the longest pile will be tested in compression and tension. If the intermediate length pile safely carries compression loads significantly in excess of that required, the shortest pile will be tested in compression and tension instead of the longest pi

			Test Load (in Kip	
Structure	Pile	Pile Lengths	C	T
Pass Section	12" Ø avg.	45, 50 55 feet	80	50
Barrier Pier 6	HP 12 x 53	80, 85, 90	360	60
Barrier Piers 1 thru 5	HP 12 x 53	90, 95, 100	360	20
Abutment	HP 12 x 74	100, 105, 110	440	140

g. Steel sheet pile cutoff. A steel sheet pile cutoff will be used beneath the control structure's pass and barrier sections to provide protection against piping caused by underseepage forces. The recommended tip elevations of the cutoffs beneath the pass and barrier bays are shown on Plate III-12. 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 recommended by Gregory P. Tschebotarioff in Chapter 5 of "Foundation Engineering," edited by G. A. Leonards, and dated 1962. The analysis was performed only for Loading Case 6. Case 6 was chosen because it was the loading case having the highest water surface elevations of the three loading cases having the greatest water level differential. The analysis was performed at 1-foot intervals with the active wedge located at the flood (lake) side edge of the structure and the passive wedge located at the protected (gulf) 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, $D = (D_A - D_P) - (R_A + R_B + R_P)$, 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) Using the method of planes stability analyses, D was determined by assuming failure at the bottom of the base of the structure and at each foot in depth thereafter. The algebraic difference in D for each 1-foot interval was used to develop the net pressure diagram. If the algebraic difference is negative, the pressure diagram indicates an available horizontal resistance in excess of that required, and if the algebraic 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-13 through III-16, 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.

7) No analysis was made for the abutment pier. Analysis of the adjacent piers resulted in a large net available resistance. It was concluded the result for the abutment pier would be the same.

8) Determination of the sheet pile tip elevations beneath the barrier section were based critical sections investigation between pier footings, where the neutral block and the resistance therefrom did not exist.

h. Structure backfill.

1) Shell. Clamshell will be used as backfill between the levee and abutment piers.

2) Sand. Sand will be used as a filter beneath the pass

section. Because of its limited availability in the dredgings at the construction site, the use of sand will be restricted. The most economical source will be determined during later investigations.

(3) Clay. Clay obtained from required excavation will be used as backfill for the control structure in areas other than those referred to above.

i. Settlement. The control structure and the adjacent piers are pile supported; therefore, there should be little or no settlement of these structures. The excavation of low density soils adjacent to these structures, and subsequent filling with heavier materials will result some settlement at and beneath the piers:

Structure	Total Settlement (feet)
Abutment Pier	3.7
Barrier Piers 1,2,3 & 4	3.2
Barrier Pier 5	2.6
Barrier Pier 6	1.5
Pass Section	0

Additional fill can be placed during construction to compensate for most of the anticipated settlement during construction.

j. Erosion protection. Protection against erosion of the sloped areas adjacent to the control structure will be provided by rip-rap on a shell blanket, as shown in plan on plate I-2. Details of design are described in Section II, paragraph 10 of this document.

k. Levee Tie-In. The levee tie-ins adjacent to the control structure will be constructed from materials excavated from the control structure excavation. Construction of these levees will coincide with the sequence of construction on the control structure. Typical sections through the tie-in levees are shown on plate III-23.

l. Approach channel. Excavation of the approach channel for a distance of approximately 1000 feet either side of the centerline of the control structure can be accomplished in the wet by dragline (5) after completion of the control structure. Material below elevation -10 ft. M.S.L. will be used for tie-in levee construction. Other excavated material will be placed within the spoil area.

m. Settlement reference markers. Settlement observations will be made by means of marked metal hubs embedded in the concrete superstructure. Such observations will be referenced to permanent bench marks installed in the deep underlying sand. In order to obtain the settlement profile along the axis of the structure, a hub will be installed at the top of each abutment and approach pier and atop three of the piers of the pass section. The three hubs on piers of the pass section will also be used to determine lateral movements of the structure during high-water periods. Provision will also be made to determine any settlement of the earth or drainage blankets away from the base of the structure.

Settlement observations will be made yearly on these marks, until settlement is essentially complete. Also observations for lateral movement of the floodwalls will be made yearly until it becomes apparent that there is no lateral movement or that movement has ceased.

n. Sequence of construction. The sequence of construction of the control structure is outlined under items 6a. and 6b. of this section.

7. Closure dam.

a. Sand section. The most economical source of suitable material for construction of the closure dam has been determined to be borrow areas within the channel of existing Chef Menteur Pass. Plate III-26 shows the location of these borrow areas and typical soil boring logs; additional boring logs are shown on plate III-9. These borings show the bulk of the Pleistocene material to be fine sand. This material is suitable for the dam construction; however, it is highly susceptable to lateral movement by the water flowing over the closure while it is being constructed. The loss of material has been observed in the construction of previous closure dams; however, with suitable allowances for bed load transport losses and adequate dredging equipment the closure of a waterway can be accomplished if the velocity of flow is not extreme.

(1) In the construction of the closure dam using hydraulic dredges, a mixture of fine sand is discharged into the moving water. The finer material will stay in suspension and be carried away by the receiving stream. The coarser material will settle out to furnish material for the closure dam. There will be a further loss of material for the closure dam due to lateral movement of the soil particles along the bottom of the section. This lateral movement is defined as bed load transport.

(2) Based on the theory of bed load transport developed by

H.A. Einstein, a critical velocity can be determined for various size material and various transport rates. Figure 1 (plate III-27) shows a series of curves giving the critical velocity as a function of the transport rate for various values of E, elevation of the top closure.

(3) From the hydraulic calculations described in section II and shown on plates II-6 and II-7 the discharge over the closure at various heights can be obtained. Based on these discharges and the water level profile the velocity over the closure was obtained. From figure 1, (plate III-27) the bedload transport rate correponding to the calculated velocity was determined. The design bedload transport rate was established as the average of case 1 and case 2. Table II-4 presents velocity over the closure and corresponding bedload transport rate for case 1 and case 2 and the mean bedload transport rate

TABLE III-1 BEDLOAD TRANSPORT RATES

El Top of Closure	Case 1		Case 2		Average Case 1 & 2
	Vc F.P.S.	Trans. Rate 1,000 cy/day	Vc F.P.S.	Trans Rate 1,000 cy/day	Trans. Rate 1,000 cy/day
-2	3.01	2.0	3.08	2.2	2.1
-6	3.28	1.5	3.41	1.5	1.5
-10	3.29	1.5	3.40	1.5	1.5
-12	3.24	1.5	3.37	1.5	1.5
-16	2.95	1.2	2.86	1.2	1.0
-18	2.78	1.0	2.76	1.0	1.0
-20	2.66	1.0	2.65	1.0	1.0
-23	2.41	1.0	2.41	1.0	1.0
-28	2.31	1.0	2.16	1.0	1.0

(4) The transport rates tabulated for case 1 and case 2 indicate a relatively low transport rate of 2,200 c.y. per day or slightly less than 10 percent of the capacity of a 25,000 c.y. per day dredge. Flood velocities for case 1 and case 2 peak at a relatively low value of 3.4 f.p.s. to provide a factor of safety to compensate for these relatively low values of transport rate and velocity. A contingency factor was applied to net fill to project gross fill requirements. The gross fill required for the closure using two, three or four dredges were calculated. The cost was determined based on the following assumptions: capacity per dredge of 25,000 c.y. per day, cost per dredge of $9,000 per day per dredge plus a fixed mobilization cost of $25,000 per dredge. The cost analysis indicates that the cost is optimized using two dredges; however, the time factor is weighted toward using three or four dredges to reduce the contingencies of weather and flow velocities to optimize transport rates:

No. of Dredges	Gross Fill (cu. yds.)	Days Required	Cost
2	3,897,000	78	1,454,000
3	3,897,000	52	1,479,000
4	3,897,000	39	1,504,000

(5) In order to minimize the scouring velocity, it is proposed to use a level fill closure. For this type of closure, it is necessary to discharge the fill material as uniformly as possible across the full width of the waterway cross section as the top of the closure is gradually raised. During the closure, the dredge piping should be located so as to discharge near the

center of the embankment.

(6) Figure 2 and 3 on plate III-27 show the fluid velocity as a function of closure elevation over the top of the embankment and at the toe of the embankment for cases 1 and 2. For case 1 the velocity at the toe and the top decreases slowly until the top of the closure reaches an elevation of approximately 28 feet. During this period, the rate of bed-load movement at the toe is close to that at the top which will result in a very flat slope from the line of placement to the toe. Above elevation -28 the rate of bedload movement at the toe begins to decrease rapidly as compared to the rate at the top which will cause the slope to steepen. For case 2, the rate of bedload movement starts relatively slow but picks up rapidly over the top while the rate at the toe decreases slowly to elevation -20 and then at a much faster rate. Since case 1 is moving the fill in the direction of Lake Borgne and alternates with Case 2 moving the fill toward Lake Pontchartrain, it is expected that embankment will be un-symmetrical with respect to the line of fill placement. The Lake Borgne side will have a much flatter slope than the Lake Pontchartrain side; however, as long as the section, it can be brought to the proper section after the flow has been stopped. Monitoring of the section during construction may indicate the desirability of shifting the line of fill placement in order to avoid filling beyond prescribed limits . To minimize the loss of material carried off in suspension, the dredged discharge lines should be provided with a tremie section so that the fill material can be placed as close to the embankment as possible.

(7) The elevation of the top of the sand section has been raised from that shown in the general design memorandum to reduce estimated settlement and facilitate keeping the closure dam to grade.

b. Clay cover. Under hurricane conditions described in section II, a differential water level on the two sides of the closure dam will exist. To prevent seepage above the water level on the low water side, an impervious clay blanket will be placed over the sand fill. The clay material shall be placed with hydraulic dredges immediately upon completion of the sand fill.

(1) Retaining dikes on each side of the sand embankment shall be constructed using stiff pleistocene clay. This material can be deposited in water and above water with an angle of repose of approximately 1 on 6. Since there may not be enough stiff clay in the borrow pits to construct the complete clay layer, soft clay may be used for the section between retaining dikes. Continuation of the clay blanket to the base of the dam is not recommended because of the limited stiff clay available in the borrow area; further, it would not materially affect the design of the closure dam cross section.

(2) The first lift has been planned so that it can be reshaped after completion of primary settlement with sufficient material existing to provide the final design section including a two foot overfill to provide for ultimate settlement.

c. Source of borrow. Both sand and clay material shall be obtained from borrow pits in the existing Chef Menteur Pass and extending into Lake

Borgne. The location of borrow pits and boring logs are shown on plate III-26 with additional soil logs on plate III-9. Additional borings will be taken in the borrow area extension into Lake Borgne prior to submittal of construction plans and specifications.

d. Seepage and hydrostatic uplift. The design section for the closure dam requires an impervious blanket over the sandy portion of the embankment to restrict seepage above water level. Seepage will, however, occur through the exposed sand below elevation - 10.0 MSL as shown by the flow net on plate IV-2. The flow net was based on a coefficient of permeability of 500 x 10-4 cm/sec. This coefficient was determined from figure 17 of volume I, WESTM No. 3-424, "Investigation of Under Seepage and Its Control, Lower Mississippi", October, 1956. The quantity of seepage that may occur, as disclosed by the flow net calculations will be about 61.4 cu.ft.per day per linear foot of embankment. This flow will only occur during a relatively short time interval and at infrequent occurrences. The thickness of the impervious blanket has been checked to assure that it can resist the maximum uplift pressure with a factor of safety not less than 1.5. No further seepage or hydrostatic uplift controls are required.

e. Stability. Using sections representative of varying foundation conditions below the proposed closure dam, the slopes and berm distances were designed to assure a factor of safety against hear failure not less than 1.3. The stability was determined by the method of planes using the design (Q) shear straight (shown on the stability plates); water levels were based on the maximum differential to be expected from project hurricane. The stability of the section where underlain with soft clay was found to be satisfactory, therefore, the excavation of the soft clay as recommended in the general design memorandum is considered unnecessary. The results of stability analysis for the closure dam are shown on plates III-25. The results for the levee tie-in are shown on plate III-25.

f. Settlement.

(1) Closure dam. It is estimated, based on consolidation curves shown on plates III-4 and III-5, that settlements varying from 1.1 ft. to 2.8 ft. can be expected due to consolidation of the underlying soil stratus and a further settlement of 2.6 ft. due to consolidation of the clay fill used to construct the impervious blanket. Most of the settlement (90%) will occur during the first three years after first lift construction. Final shaping of the closure dam will provide for a 2-foot over-build to accommodate any further settlement.

(2) Tie-in levee. The estimated settlement for the tie-in levee is 7.5 ft. for the first lift and an ultimate settlement for the final section of approximately 10.0 ft. If the second lift is placed 2 years after the first lift approximately 6 feet of the settlement will have occurred and 2 years later the estimated total settlement will be 9 feet or 90% of the ultimate settlement. The 2-foot over-build in final shaping will accommodate the remaining anticipated settlement.

g. Erosion protection. During hurricane periods, the windward slope of the closure dam will be subjected to significant wave action. To protect the slopes against erosion from the waves, riprap protection will be provided on both sides of the earthern embankment. From table II-3 of this report, the maximum predicted wave height is approximately 12 feet. The maximum wave height on the Lake Pontchartrain side is somewhat less than on the Gulf side; however, the difference is not sufficient to justify a different design.

(1) The design of riprap is based on the procedure outlined in EM 1110-2-2300. The result of this analysis indicate that a two (2) foot layer of riprap with an average weight of rock equal to 200 pounds is required. The layer of riprap will be placed on a 9 inch layer of clam shell and the protection will extend from the crown of the closure dam along the slope and berm to elevation -5.0 MSL. The crown of the levee will be protected against erosion by the compacted shell roadway extending across the barrier system.

h. Levee tie-in. Typical sections through and stability analysis of the closure dam levee tie-in are illustrated on plate III-25. Settlement is discussed in the preceeding paragraph f. (2). The levee tie-in between the closure dam and the protection levee will be constructed for a length of 190 feet beyond the "limits of closure dam". (See plate I-5).

i. <u>Sequence of construction</u>.

(1) After the control structure, navigation structure, control channel, and navigation channel are completed, closure of the existing Chef Menteur Pass will be started. Material shall be dredged from both borrow pits designated within the existing Chef Menteur Pass and the extension into Lake Borgne. Because of the importance of bedload transport losses, the dredging must be continuous and at maximum efficiency.

(2) Immediately upon completion of sand fill to the specified cross section, the placement of impervious clay layer shall be started. Retaining dikes will be constructed on each side of the sand embankment as shown on plate IV-3. The material for the retaining dike shall be stiff Pleistocene clay which can be placed by hydraulic dredge and maintain a reasonably steep angle of repose below water (estimated 1 on 6). The clay material between the retaining dikes may be soft to stiff clay depending upon availability in the borrow area. Hydraulically placed clay shall be brought to the section on plate IV-3.

(3) After the 90% of the ultimate settlement has taken place (estimated at 3 years) the section will be shaped to the final section except that an overbuild of the crown shall be provided to accommodate any further settlement.

(4) Upon completion of the shaping, shell bedding and riprap shall be placed to protect the slopes against wave erosion.

(5) The tie-in shall be constructed concurrently with the closure dam except that due to the substantially greater settlement which is expected outside of the Chef Menteur channel additional material will be required before shaping operation can be made. Because of the relatively small amount of material that would be required, it would be more economical to provide the additional fill and shaping of the levee along with the final shaping of the protection embankment.

SECTION IV - CLOSURE DAM DESIGN

1. General. The Chef Menteur Pass is one of the two natural outlets of Lake Pontchartrain. This pass, which connects Lakes Pontchartrain and Borgne, is naturally developed and is approximately seven (7) miles long, 1,000 feet wide, and has a nominal depth of 43 feet. The gated control structure and control channel presented in this design memorandum has been designed to replace approximately 2-1/2 miles of the Pass adjacent to Lake Borgne and will maintain substantially the same hydraulic and ecological regimens during normal conditions. A navigation channel and gated navigation structure, to be presented in a subsequent detail design memorandum, will provide the necessary navigational connection. After these new facilities have been completed, the existing Chef Menteur Pass will be closed with an earthen dam. The selection of the type of dam is discussed in previous report; Lake Pontchartrain, Louisiana and vicinity, Lake Pontchartrain Barrier Plan, Design Memorandum No. 2 - General Design, Supplement No. 3, Chef Menteur Complex.

2. Method of construction. Based on hydraulic design criteria developed in section II of this report and soil strength characteristics presented in section III, the closure dam cross section has been designed to provide necessary stability and also to minimize filling costs in consideration of bedload transport losses.

3. Based on a modified Einstein procedure for estimating the bedload transport losses, it was determined that the optimum filling operation would be accomplished using two dredges each of 27-inch size or larger. With this equipment it has been determined, at least theoretically, that the closure could be made in 78 days with a total bedload transport loss of only 2,200 cubic yards. Because investigators have found considerable variation between theoretical losses and actual measured losses, at the estimate was increased to provide a contingency factor.

4. The cost of making the closure will depend to a large measure on the efficiency and skill of the contractor undertaking the work. The contractor must keep his equipment operating continuously at maximum capacity and make careful observations to assure that dredge discharge lines are located to keep the embankment within or as close as possible to the required section. In addition, it will be difficult to measure the actual quantity of fill material pumped.

5. For these reasons, it is recommended that the contract for the closure dam embankment be bid on a lump sum basis with full responsibility for performance given to the contractor. Two local contractors have been consulted on the advisability of this type of contract. Both contractors have extensive experience in large dredging operations including closures made in flowing water. Both contractors agree that

a lump sum performance bid would be the most practical method of accomplishing this work. Because of the relatively small amount of settlement anticipated or the closure dam, the initial section has been designed so that there will be sufficient material for final shaping without having to place a second lift.

6. The construction of the barrier embankment which is adjacent to and is on alignment with the closure dam will be substantially completed before the closure dam is constructed. The barrier embankment will terminate at station 175+10 on base line "A" (approximately 885 feet west of the centerline of Chef Menteur Pass) and begin again at station 192+81 (approximately 885 feet east of the centerline of the Pass). The first lift of the tie-in levee can be constructed concurrently with the closure; however, because of highly compressible nature of soils in the upper strata of the levee foundation, a second lift will be required to bring the levee in this area back to the design grade.

7. Erosion protection. After final shaping of the earthen embankment the slopes and beams on both sides of the embankment will be protected against erosion due to wave action by the placement of a two (2) foot layer of riprap on a nine (9) inch bed of clam shell. The protection will extend from the crown of the levee to elevation -5.0 MSL.

SECTION V-STRUCTURAL DESIGN

1. General. This section presents the basic criteria, assumptions methods of analysis and results of computations for the design of the principal features of the control structure including the gantry crane and vertical lift gate. Structural design is made in accordance with standard engineering practice, with criteria set forth in the Engineering Manual for Civil Works construction published by the Office, Chief of Engineers, and Standard Specifications for Highway Bridges, published by the American Association of State Highway Officials. The general features of the structure, (Plates V1 thru V-3) structural concepts and methods of design follow the preliminary design presented in Design Memorandum No. 2 - General Design Supplyment No. 3 Chef Menteur Pass Complex, dated May 1969.

2. Pertinent data. Elevations, dimensions and lake stages governing the layout and design of the control structure are given below:

Pass section

Elevations	Ft. msl
Bottom of channel	El. - 40.0
Sill	El. - 25.0
Bridge deck	El. - 14.0
Top of gantry beams	El. - 14.0
Structure foundation	El. - 31.0

Dimensions	Feet
Channel width at base	700.0
Gross pass section width	408.0
Clear bay width	46.0
Bay opening height	27.0
Bay slab length	60.0
Pier thickness	5.0

Barrier section

Elevations	Ft. msl
Base of footing abutment,	
Piers 1 and 2	El. - 8.0
Piers 3, 4 and 5	El. - 11.0
Pier 6	El. - 21.0
Bridge deck	El. - 14.0
Top of gantry crane beams	El. - 14.0

Dimensions	Feet
Bay width	51.0
Pier 6 footing thickness	4.0
Pier 6 footing length	60.0
Pier 6 footing width	24.0
Other pier footing thickness	4.0
Other pier footing length	55.0
Other pier footing width	16.0
Abutment footing thickness	4.0
Abutment footing length	55.0
Abutment footing width	16.0

Water levels

	Gulf side El.	Lake side El.
Case 1*	+ 12.8	+ 4.0
Case 2*	+ 11.8	- 2.0
Case 3*	+ 10.5	- 4.0
Case 4	+ 9.0	- 5.0
Case 5+	+ 5.0	+ 2.5
Case 6*	- 3.0	+ 11.5
Case 7	- 4.0	+ 9.0
Case 8+	- 3.0	+ 2.5

* Still water levels does not include wave effects.

+ Still water levels for gate operation.

3. Unit weights. The following values of unit weights of materials were used in the design:

a. Material	Wt. - lb per cu. ft.
Water	64
Concrete	150
Earth backfill (dry)	100
Earth backfill submerged	60
Earth backfill moist	122

4. Loadings.

a. Water. The horizontal water loadings acting on the pass and barrier sections of the structure and their determination has been discussed in Section II of this memorandum. The design loading conditions consist of a hydrostatic loading for cases 4 and 7 and in cases 1, 2, 3, and 6 a hydrodynamic loading due to the hurricane wave effect these loadings have been indicated on the Pile Foundation Analysis exhibits in this section.

b. Wind. In computing the stability and foundation analysis, the structure shall be subjected to a wind load of 30 lbs/sq.ft. on the vertical projection of the exposed structure. The direction of the hurricane in the cases analyzed dictated the direction of the wind load application. In the construction condition, (no hurricane) designated case 9 the indexing of the case i.e., 9.1 and 9.2 indicates reverse wind directions. Wind load in the design of the gantry beams will be based upon 50 lbs/sq.ft. acting on the gantry crane and supported gate leaf.

c. Earthquake. The Chef Menteur Complex is located in an area which is designated by the Uniform Building Code (International Conference of Building Officials, 1961) as Zone O. Structures in this zone are classified as not being subject to damage from earthquakes and therefore the coefficient multiplier for vertical loads to obtain the lateral seismic load effect is zero. During a conference at the New Orleans District Office it was decided that seismic effects be considered and a multiplier of 0.05 be used. Seismic hydrodynamic loading on Lake Borgne and Pontchartrain levels will be in accordance with the general Westergaard theory. Seismic effects will be investigated with the conditions stated for cases 4 and 7. Wind load will be excluded.

d. Uplift. Uplift loads on the control structure shall be computed over 100% of the base area. Two uplift loading conditions were investigated in the stability analysis of the pass section. The case numbers indexed .1 assumed the uplift pressure distribution to vary uniformly across the base with the pressure at each end being equivalent to that caused by the design water level for the case under consideration. The case numbers indexed .2 assumed the uplift pressure distribution to be uniform on each side of the steel sheet pile cut-off. The uplift pressure is equivalent to that caused by the design water level for the case under consideration. The selection of these two uplift conditions was based on the fact that the pass section would be founded on a placed granular material. Under optimum conditions the .2 case would exist, but over the life of the project the efficiency of the granular material would decrease, thereby causing an uplift condition that would approximate .1 case. The barrier piers and abutment are founded on the natural foundation, but the bases are narrow, and with an uplift equivalent to the design water level acting along the periphery, the uniform uplift of the .2 case was selected for analysis.

e. Access roadway bridge. The access roadway bridge dead load reaction shall be applied to the supporting piers for all cases analysed. In the construction case, (case 9) both dead load and live load are applied. The design rating of the access roadway bridge is HS20-44.

f. Gantry crane. In the stability analysis of the pass section the gantry crane was only considered in the construction case (case 9). All other cases are hurricane cases hence the gantry crane would not be on the pass section. The gantry crane was considered in all cases for stability analysis of the barrier section piers and abutment.

5. Lateral earth pressures. Lateral earth pressures shall be based upon the following equations

Active pressure intensity

$$= W\ h\ Tan^2\ (45-\phi/2) - 2C$$

"At rest" pressure intensity

$$= 0.5\ Wh$$

Passive pressure intensity

$$= W\ h\ Tan^2\ (45+\phi/2) + 2C$$

W = unit weight of material

h = height of backfill above plane of analysis

ϕ = angle of internal friction

C = unit cohesive force

6. Allowable working stresses. The allowable working stresses for structural steel and concrete are in accordance with those recommended in "Working Stresses for Structural Design," EM1110-1-2101 of 1 November 1963. For convenient reference, allowable stresses are tabulated as follows:

a. 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	$0.83\ K_1F_Y$	$1.11\ K_1F_Y$

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

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 then C_c

$$\frac{124{,}000{,}000}{\left(\frac{KL}{r}\right)^2} \qquad \frac{165{,}000{,}000}{\left(\frac{KL}{r}\right)^2}$$

On secondary member when $L/r > 120$, modify the above values by multiplying by the following factor: $\frac{1}{1.6-L/200r}$ $\frac{1}{1.7-L/200r}$*

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
Tension on extreme fibers of other rolled shapes, built-up members and plate girders	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
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)	$0.50\ K_2F_Y$	$0.67\ K_2F_Y$

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

$C_b = 1.75\text{-}1.05\ (M_1/M_2)+0.3(M_1/M_2)^2$, 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.

	Group 1 loading psi	Group 2 loading psi
(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.

Application	Group 1 loading psi	Group 2 loading psi
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.

6) Bolts (tension)		
A307 bolts	11,500	15,500
A325 bolts	33,500	44,500
A354 bolts (grade BC)	41,500	55,500

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
A354 bolts (Grade BC) when threading is not excluded from shear planes	16,500	22,000
A354 bolts (Grade BC) when threading is excluded from shear planes	20,000	26,500

Application	Group 1 loading psi	Group 2 loading psi
8) Bolts (shear) (friction type connections)		
A325 bolts	12,500	16,500
A354 bolts (Grade BC)	16,500	22,000
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 $f_a/F_a \geqq 0.15$, F_e = Euler stresses divided by factor of safety

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

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

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

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 porportioned so that the tension stress from the force applied to the connected part does not exceed the following:

For A307 bolts	$F_t = 15{,}000 - 1.6\ f_v \leqq 10{,}500$
For A325 bolts in bearing type joints	$F_t = 37{,}500 - 1.6\ f_v \leqq 30{,}000$
For A354 bolts (Grade BC) in bearing type joints	$F_t = 45{,}000 - 1.6\ f_v \leqq 37{,}500$

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	$F_v \leqq 11{,}000\ (1 - f_t A_b/T_b)$
For A354 bolts	$F_v \leqq 15{,}000\ (1 - f_t A_b/T_b)$

T_b = the proof load of the bolt.

b. 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.

c. 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

d. 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. Timber piles subject to both axial and bending stresses shall meet the following requirements:

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

In compression the allowable stresses shall be:

F_a = 1200 psi

F_b = 1550 psi

In tension the allowable stresses shall be:

$F_a = F_b$ = 1550 psi

The TECO Ten-con pile tension connectors shall have an allowable load of 5,000 pounds per connector.

e. Steel piles. Steel piles subject to both axial and bending stresses shall meet the following requirements:

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

where: F_a = 12,000 psi

F_b = 18,000 psi

These piles have not been designed as columns, because the computed allowable stress in compression for the actual (Kl/r) ratios would be extremely low and would not agree with the allowable stresses that actual experience reveals to be acceptable.

Beneath the abutment piers the steel piles are subjected to loads in orthogonal directions. In these cases the stresses were computed neglecting biaxial bending.

f. Steel and concrete sheet piling. The sheet pile cutoff beneath the control structure and extending into the abutting levee shall be of carbon steel. The sheet piling in the levee transition at the control structure abutment which is expanded to view will be precast concrete sections. The precast concrete sheet piling will be of the tongue and groove type. Concrete shall have a 28 day ultimate strength of 4,000 psi. Design stresses shall be in accordance with the applicable provisions of paragraph 6. Allowable working stresses.

g. Roadway bridge bearing. Elastomeric pads shall be used for the roadway bridge bearings. To provide for expansion and contraction the thickness of the bearing pad shall not be less than 0.001 times the bridge span in feet.

Allowable bearing pad compression stresses for the following loading conditions shall not exceed:

Bridge dead load only	500 psi
Bridge dead + live load	800 psi

The bearing pad shall have a hardness as measured by durometer no greater than 60.

7. Modulus of elasticity. The modulus of elasticity of concrete and steel is assumed as follows:

Concrete	3,000,000 psi
Steel	30,000,000 psi

8. Coefficient of expansion. The coefficient of expansion of concrete and steel is assumed as follows:

Concrete	0.000005/°Fahrenheit
Steel	0.0000065/°Fahrenheit

9. Control structure.

a. General. The Chef Menteur Control Structure will be a gated overflow structure constructed across an excavated channel which joins Lake Borgne with the Chef Menteur Pass. The overflow section or pass section as it is referred to in this memorandum consists of 8 gate bays 46 feet wide separated by piers 5 feet thick. The piers support a roadway access bridge which crosses the control structure and joins the roadway atop the abutting levee sections. The piers also support the gantry crane beams which run continuously from abutment to abutment of the control structure. The gantry crane is provided to handle sectional vertical lift gates. Two gate leafs are provided for each gate bay. The gate sill is at El. - 25.0, the bottom of the gate head beam is at El. 2.0. The gate head beam also functions as one of the gantry crane beams. The other gantry crane beam is a tee beam section and maintains this shape across the control structure. The gantry crane rail gage is 19'. The tee flange of the gantry crane beams provides a walkway for maintenance personnel.

The portion of the control structure joining the pass section with levee consists of a series of piers on independent slab footings. This section of the control structure is called the barrier section. The piers are connected by a barrier wall which lies beneath the gantry crane beam. This gantry crane beam is a continuation of the beam that also served as the gate head beam in the pass section. A steel sheet pile cutoff is embedded in the base of the barrier wall. This cutoff is also continuous beneath the pass section and extends into the levee fill. Details of the control structure are shown on Plates V-1 through V-3.

b. Net overflow area. The net overflow area required in the pass section was determined as 9,200 square feet below El. 0.0. Studies were made during the preparation of Design Memorandum No. 2, Supplement No. 3 on the various types of gates, methods of gate operation and corresponding structures. These studies culminated in the selection of an 8 bay pass section utilizing sectionalized vertical lift gates.

c. Dewatering of gate slots. Pass section gate slots will be dewatered for inspection and maintenance by lowering a steel bulkhead that will span the gate slot. Removing the water from the gate slot will cause the differential hydrostatic pressure to seal the bulkhead against the pier surface adjacent to the gate slot.

10. Design of pile foundation. Design investigations showed that the control structure is subjected to large lateral loads and uplift forces. This requires a large structural base in order to maintain the resultant base pressure within the kern limit. The low shear strength of the foundation at the site does not allow the use of a spread footing, unless it is supported on bearing or friction piles. The subgrade material does not present a bearing stratum. Therefore, the control structure foundation was designed to be supported by friction piles.

The Hrennikoff Method of pile foundation design as formulated in the ASCE Transactions, Paper No. 2401, Analysis of Pile Foundations With Batter Piles, was used in this investigation.

Because of the large horizontal component of the external loadings, only batter piles were considered in the final designs. Preliminary studies indicated vertical piles would severely reduce the overall ability of the total pile arrangement to resist the horizontal loadings. A design requirement was that pile spacing was not to exceed ten feet, for timber piles and seven feet for steel piles, nor be less than two and one half feet.

a. Abutment piers.

1) Loading cases. The loading cases 1 through 4, 6, 7 and 9 as described below were analyzed. The hurricane wave characteristics are a function of the approach channel and are determined on Plate II-5 for the various sections. Graphic representations of the hydrostatic and wave loads are presented for the sections on Plates II-6, 7, and 8. Tabulation of the horizontal and vertical forces for each loading case is presented on Plates V-4 through V-11.

The cases of differential water levels and other loading used in determining pile reactions are as follows:

Case 1. Headwater on Gulf side El. 12.8. Lake Side El. 4.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 2. Headwater on Gulf side El. 11.8. Lake side El. -2.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 3. Headwater on Gulf side El. 10.5. Lake side El. -4.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 4. Headwater on Gulf side El. 9.0. Lake side El. -5.0. No gantry crane. Uplift. (Group 1 loading)

Case 6. Headwater on Lake side El. 11.5. Gulf side El. -3.0. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 7. Headwater on Lake side El. 9.0. Gulf side El. -4.0. No gantry crane. Uplift. (Group 1 loading)

Case 9. Construction Case. Structure completed in the dry. Gantry crane over the bridge with one gate leaf in lifted position. Wind of 30 psf to be assumed acting on structure, gantry crane and gate section. Uplift assumed to be zero. (Group 2 loading)

Uplift pressure distribution on the base slab was considered uniform, being equal to the design water level on either side of the sheet pile cut-off. An uplift pressure distribution varying uniformly between design water levels at each side was not considered possible because of the limited width of these pier footings.

Unbalanced lateral earth, water and wind loads from Lake side and Gulf side in addition to the transverse load produced by the levee fill will be resisted by piles battered at 1 horizontal to 2 vertical in a direction of 45° to the longitudinal and lateral (transverse) centerlines of the base slab.

2) Results of analyses. High negative skin friction on piles beneath the abutment piers did not allow an economic comparison of different pile types to be made. The negative skin friction force alone exceeded the allowable load of 40 Kips on a timber pile. While a precast concrete pile could have taken the required load, its length would have greatly exceeded the allowable maximum of about 80 feet. Therefore, only steel H-piles were considered.

Final axial pile loads for each case were obtained by combining the results of two separate analyses: one in the longitudinal direction and the other in the lateral direction. In the longitudinal direction, the full effect of all vertical and longitudinal horizontal forces was applied to a section on which the true pile batters were projected.

In the lateral direction, the full effect of all lateral horizontal forces, and only the moments caused by vertical forces, were applied to a section on which the true pile batters were projected. In each direction the projected pile batter used was 1 horizontal to 2.82 vertical. The true axial pile loads calculated were obtained by projecting the pile loads from the longitudinal and lateral direction into the true axial direction and summing them.

The critical pile load (highest ratio of actual to allowable axial load) for every pile is summarized on Plates V-12 through V-17. External forces for loading cases which were critical for at least one pile are summarized on Plates V-4 through V-11. All loads are shown for these cases, and axial loads obtained by graphical means are compared with those results, obtained from the Hrennikoff analysis. Plots of allowable and actual transverse pile loadings and deflections at various subgrade modulii are presented in Appendix B. The index .2 in the loading case number (as 4.2) indicates the uplift pressure was considered to be uniform.

Group action of the overall pile arrangement for the abutment was studied for possible reduction in the pile bearing values. This was done by considering the piles and footing as a deep footing and computing its bearing capacity. This study indicated no reduction for group action was necessary.

b. Barrier piers.

1) Loading cases. The loading cases 1 through 4, 6, 7 and 9 as described below were analyzed. The hurricane wave characteristics are a function of the approach channel and are determined on Plate II-5

for the various sections. Graphic representations of the hydrostatic and wave loads are presented for the sections on Plates II6, 7 and 8. Tabulation of the horizontal and vertical forces for each section and loading case is presented on Plates V-18 through V-24. All forces acting directly on the pier or on the barrier wall 25.5 feet on either side of the pier centerline were included in the pier's foundation analysis.

The cases of differential water levels and other loading used in determining pile reactions are as follows:

Case 1. Headwater on Gulf side El. 12.8. Lake side El. 4.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added Uplift. (Group 2 loading)

Case 2. Headwater on Gulf side El. 11.8. Lake side El. -2.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 3. Headwater on Gulf side El. 10.5. Lake side El. -4.0. Gantry crane is in place. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 4. Headwater on Gulf side El. 9.0. Lake side El. -5.0. No gantry crane. Uplift. (Group 1 loading)

Case 6. Headwater on Lake side El. 11.5. Gulf side El. -3.0. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading)

Case 7. Headwater on Lake side El. 9.0. Gulf side El. -4.0. No gantry crane. Uplift. (Group 1 loading)

Case 9. Construction Case. Structure completed in the dry. Gantry crane over the bridge with one gate leaf in lifted position Wind of 30 psf to be assumed acting on structure, gantry crane and gate section. Uplift assumed to be zero. (Group 2 loading)

Uplift pressure distribution on the base slab was considered uniform, being equal to the design water level on either side of the sheet pile cut-off. An uplift pressure distribution varying uniformly between design water levels at each side was not considered possible because of the limited width of these pier footings.

Unbalanced lateral earth water and wind loads from Lake side and Gulf side will be resisted by battering piles in both directions. All piles will have a batter of 1 horizontal to 2 vertical.

(2) Results of analyses. As with the abutment piers, the high negative skin friction beneath the barrier piers did not allow an economic comparison of different pile types to be made. Only steel H-piles were considered. The critical pile load (highest ratio of actual to allowable axial load) for every pile is summarized for the barrier piers on the Plates listed below:

Barrier pier	Plates
1, 2	V-25 and V-26
3, 4	V-34 and V-35
5	V-43 and V-44
6	V-52 and V-53

External forces for loading cases which were critical for at least one pile are summarized for the barrier piers on the Plates listed below:

Barrier pier	Plates
1, 2	V-18 through V-24
3, 4	V-27 through V-33
5	V-36 through V-42
6	V-45 through V-51

All loads are shown for these cases, and axial loads obtained by graphical means are compared with those obtained from the Hrennikoff analysis. Plots of actual and allowable transverse pile loadings and deflections at various subgrade modulii are presented in Appendix B. The index .2 in the loading case number (as 4.2) indicates the uplift pressure was considered to be uniform.

Group action of the overall pile arrangement for each pier was studied for possible reduction in the pile bearing values. This was done by considering the piles and footing as a deep footing and computing its bearing capacity. This study indicated no reduction for group action was necessary.

c. Pass section monoliths.

(1) Loading Cases. The loading cases 1 through 4, 6, 7 and 9 as described below were analyzed. The hurricane wave characteristics are a function of the approach channel and are determined on Plate II-5. Graphic representations of the hydrostatic and wave loads are presented for the pass section on Plate II-9. Tabulation of the horizontal and vertical forces for each loading case on the pass

section is presented on Plates V-57 through V-67. The foundation was designed as two separate components: one for a 27 foot wide pier strip centered about each of the nine piers and the second for the eight 24-foot wide bays between the pier strips. The loads transmitted to the pass structure piers by the gates, gantry crane beams and bridge were assumed to be taken by the pier strips. The loads taken by the bay strips arise from water loads acting directly on these strips, plus the weight of the strip itself. Bay strip loads are shown on Plates V-70 through V-73.

The cases of differential water levels and other loadings used in determining pile reactions are as follows:

Case 1. Head water on Gulf side El. 12.8. Lake side El. 4.0. Gates are closed. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading).

Case 2. Headwater on Gulf side El. 11.8. Lake side El. -2.0. Gates are closed. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading).

Case 3. Headwater on Gulf side El. 10.5. Lake side El. -4.0. Gates are closed. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. UPlift. (Group 2 loading).

Case 4. Headwater on Gulf side El. 9.0. Lake side El. -5.0. Gates are closed. No gantry crane. Uplift. (Group 1 loading).

Case 6. Headwater on Lake side El. 11.5. Gulf side El. -3.0. Gates are closed. No gantry crane. Wind of 30 psf. Dynamic wave force to be added. Uplift. (Group 2 loading).

Case 7. Headwater on Lake side El. 9.0. Gulf side El. -4.0. Gates are closed. No gantry crane. Uplift. (Group 1 loading)

Case 9. Construction Case. Structure complete in the dry. Gantry crane is in place with one gate section in lifted position. Second gate section is stored. Roadway bridge live load. Wind of 30 psf. to be assumed on structure, gantry crane and gate sections. Uplift assumed to be zero.

In Cases 1, 2, 3, 4, 6, and 7 two types of uplift pressures distributions were considered.

Case numbers indexed .1 considered uplift as varying uniformly across the base slab from a Gulf side design water level to Lake side design water level.

Case numbers indexed .2 considered uplift as being uniform and being equal to the design water level on either side of the sheet pile cut-off.

For Case 9 the index .1 indicates wind load from the gulf side and .2 indicates wind load from the lake side.

Unbalanced water and wind loads from the Lake side and Gulf side will be resisted by battering piles in both directions. All piles will have a batter of 1 horizontal to 2 vertical.

(2) Results of analyses. Preliminary control structure pass section base widths of 60, 67.5 and 75 feet were selected for study with timber and prestressed concrete friction piles. A more detailed description of the analyses appears in Appendix A. The analyses indicated that the timber pile scheme was lower in cost and that the 60 foot base width was the least costly.

The critical pile load (highest ratio of actual to allowable axial load) for every pile is summarized on Plates V-68, V-69 and V-74. External forces for loading cases which were critical for at least one pile are summarized on Plates V-54 through V-67 and V-70 through V-73. All loads are shown for these cases, and axial loads obtained by graphical means are compared with those obtained from the Hrennikoff analysis. Plots of actual and allowable transverse pile loadings and deflections at various subgrade modukii are presented in Appendix A.

Group action of the overall pile arrangement for the pass section was studied for possible reduction in the pile bearing values. This was done by considering the piles and footing as a deep footing and computing its bearing capacity. This study indicated no reduction for group action was necessary.

11. Masonry design.

a. General. The half-plan and typical sections through the control structure are shown on Plate I-3. Analysis and design of control structure is divided into six items

1) Abutment piers
2) Barrier piers
3) Pass section
4) Barrier walls
5) Gantry crane beams
6) Access roadway bridge

b. Abutment piers. The principal structural function of the abutment piers is to retain the levee fill, support the gantry crane beams, access roadway bridge, support and offer lateral rest-raint to the barrier wall and transmit these loads to the pile foundation. The design of the pile foundation was discussed in paragraph 10 a. These vertical and horizontal loads are transmitted through the stem of the piers to the pier footing and thence to the pile foundation.

The abutment pier is 18 feet high above the base slab and is 55 feet wide. Wing walls monolithic with the abutment pier at each end of the pier extend back into the levee where they are joined at the face of the base slab with the precast concrete sheet piling. The pier is designed as a retaining wall as well as for the vertical loads of the roadway bridge, gantry crane beams, back fill and barrier wall. Plates V-4 through V-11 show the loads imposed upon the abutment for the design loading cases, the resultant forces and pile vector polygon diagrams for the critical cases.

The abutment pier footing was porportioned to provide space for the supporting piles, allow for normal penetration of the piles into the base slab, and to provide ample thickness to resist the maximum pile loads on the footing. A summary of the pile loadings for the critical cases and the pile layout are shown on Plates V-12 to V-17. The base slab analysis showing required reinforcement at the critical section is shown on Plate V-75.

c. Barrier piers. The principal structural function of the barrier piers is to support the gantry crane beams, access roadway bridge, support backfill, support and offer lateral restraint to the loads imposed on the barrier wall and transmit these loads to the pile foundation. The design of the pile foundation was discussed in paragraph 10 b. These vertical and horizontal loads are transmitted through the stem of the piers to the pier footing and thence to the pile foundation.

The foundation grade varies across the barrier section (See Plate I-3) consequently the physical dimensions of the piers varied. The barrier pier sections analysed with their pier height and base dimensions are listed below.

Barrier Pier	Height	Base
1, 2	18'	55 x 16
3, 4	21'	55 x 16
5	21'	55 x 16
6	31'	60 x 24

The piers are designed primarily to resist the lateral load imposed upon them in shear at any elevation. Under static conditions hydrostatic and earth loading transverse to the pier are balanced. Transverse loads from the gantry crane and roadway bridge beams due to thermal effects or live load longitudinal thrusts are of little consequence because of the magnitude and the low frictional resistance of the support bearings. An unbalanced conditions producing a transverse loading on the pier may be obtained by considering a hurricane condition where waves striking the barrier wall on each side of the pier are assumed as being out of phase. This condition would cause an unbalanced hydrodynamic loading on the pier.

This condition was analyzed for case 1 on one side of the pier and a hydrostatic loading due to still water elevation for case 1 on the other side. This is a group 2 loading condition and the reinforcement required for this condition will govern.

The design loading cases showing the loads imposed upon the barrier piers the resultant force and pile force vector ploygons for the critical cases are indicated on the plates listed below for the corresponding piers.

Barrier pier	Plates
1, 2	V-18 through V-24
3, 4	V-27 through V-33
5	V-36 through V-42
6	V-45 through V-51

The pier footings were proportioned to provide space for the supporting piles, allow for normal penetration of the piles into the base slab, and to provide ample thickness to resist the maximum pile loads on the footings. For plans of footings and elevations of barrier piers see Plate III-12. A summary of the pile loadings for the critical cases and the pile layouts for the barrier piers are shown on the Plates listed below.

Barrier pier	Plates
1, 2	V-25 and V-26
3, 4	V-34 and V-35
5	V-43 and V-44
6	V-52 and V-53

The base slab analysis showing required reinforcement at the critical sections are shown for the barrier piers on Plate V-77.

d. Pass section.

1) General. The principal structural function of the pass section is to support the vertical lift gates, gantry crane beams, access roadway bridge and offer lateral restraint to the water loads imposed on the gates and transmit these loads to the pile foundation. The design of the pile foundation was discussed in paragraph 10 c. These vertical and horizontal loads are transmitted through the stem of the piers to the pier footing and thence to the pile foundation.

The pass section pier is 39 feet high above the base slab and is 50 feet wide at the base tapering to approximately 43 feet in width at the top (El. 14). The pier is 5' thick at the gate slot this thickness is reduced to 1'-10". The width of the base slab is 60 feet; causing it to extend 5 feet beyond the pier on each side. The pass section was analyzed as two interacting structures, identified as the pass pier and the pass bay (see Plates I-3 and V-1).

2) Pass pier. The pass pier is a monolithic structure composed of one pier and a 27 foot by 60 foot base slab. The pass pier is designed primarily to resist the lateral load imposed on it in shear at any elevation. These lateral loads plus the vertical loads of the gantry crane beams and access roadway bridge are transmitted to the base slab and thence the foundation.

Under static conditions the hydrostatic loading transverse to the pier is balanced. Transverse loads from the gantry crane and roadway bridge beams due to thermal effects or live load longitudinal thrusts are of little consequence because of the magnitude and the low frictional resistance of the support bearings. An unbalanced condition producing a transverse loading on the pier may be obtained by considering a hurricane condition where waves striking the lowered gate leafs on each side of the pier are assumed as being out of phase. This condition would cause an unbalanced hydrodynamic loading on the pier. This condition was analyzed for case 1 with wave effect on one side of the pier and a hydrostatic loading due to still water elevation for case 1 on the other side. This transverse loading condition is shown on Plate V-79. This is a group 2 loading condition and reinforcement for this condition will govern.

The design loading cases, showing the loads imposed upon the pass pier section, the resultant force and pile force vector polygons for the critical cases are indicated on Plates V-54 through V-67. The 27 foot long footing was selected because the pile arrangement required to satisfy all loading cases could be accomodated without placing piles at a spacing less than the minimum specified. This compact arrangement reduced the base slab design moments so that bending in the base slab was not the governing design requirement. A summary of the pile loadings for the critical case and the pile layout are shown on Plates V-68 and V-69. The base slab analysis showing required reinforcement at the critical sections is shown on Plates V-77 and V-78.

3) Pass bay. The pass bay consists of a base slab section which adjoins the pass pier base slabs. The base slabs are not monolithic but transmit shear load through a key, (see Plates I-3 and V-1). The pass bay is designed to resist its own dead load, the gate reaction on the sill beam, uplift, vertical water load and the lateral earth and water loads acting on the base slab's vertical face.

The design loading cases, showing the loads imposed upon the pass bay section, the resultant force and pile force vector polygons for the critical cases are indicated on Plates V-70 through V-73. A summary of the pile loadings for the critical case and the pile layout are shown on Plate V-74. The 24 foot long pass bay when subjected to the above analyses was found to require piles at an arrangement that approached the maximum allowable spacing. The base slab analysis showing required reinforcement at the critical section is shown on Plate V-79.

e. Barrier walls. Concrete barrier walls will span between piers and form the water barrier connection to the abutment pier and end pass pier. These walls will rest in slots in the piers, the detail of which will permit minor moments due to temperature changes and/or differential settlement. The barrier walls have been investigated for the water level cases listed under paragraph 2. Pertinent data. Cases 1, 2, 3, and 6 are cases in which the hydrodynamic loading effect of hurricane wind induced waves are included. Cases 4 and 7 are hurricane cases in which hydrodynamic effects are not included and the cases are classified as group 1 loading condition. The wind load during the construction condition, applied to the vertical surface of the barrier walls is not a critical condition. Seismic lateral loads due to the wall mass and the seismic effect applied to the water levels for cases 4 and 7 also proved not critical. The effect of earth pressure against the buried portion of the barrier walls has been neglected in the design conditions listed above. A barrier wall section analysis for critical loading conditions producing maximum reinforcement requirements in each face is shown on Plate V-82.

f. Gantry crane beams. The gantry crane beams will consist of two reinforced concrete beams running the length of the control structure. The gantry crane rails which the beams support are spaced 19 feet center to center. The beams have been designed to support the following estimated live loads:

Crane + counterweight	265,000 lbs.
Trolley weight	70,000 lbs.
Total gate weight	100,000 lbs.

For the purpose of design, the center to center spacing of the gantry crane carriage trucks was assumed to be 53 feet, with two wheels per truck spaced 2.5 feet apart. The gantry crane beams are of two sections; a tee beam and the deeper section which also functions as the pass bay head beam (see Plate V-3 section B-B).

1) Gantry-head beam. The overall depth of this beam is 10 feet. The beam has a tee flange at the deck El. 14.0 and an L flange at the bottom El. 2.0. The depth of this member requires that it be de-

signed for the lateral loads of the static differential water levels, hurricane wave loads, seismic effect on mass and hydrodynamic loads plus any gate loads on the head beam. Design for these lateral loads assumed that the tee flange and L flange acted as beams, in supporting laterally the wall (gantry crane beam stem). In the vertical direction the beam was designed for the loading conditions listed in 3), below. This beam's action will be monolithic with the pass pier. Alternate bay concrete placement for this beam during construction is advocated so as to reduce shrinkage induced moments in the pass piers.

2) Gantry crane tee beam. The overall depth of the beam is 5 feet, the tee flange is 8 feet 3 inches wide. A niche is provided in the piers so that the beam may be laterally restrained and to afford vertical support.

The beam is anchored but not fixed against rotation at one support and at the other support rests on a teflon surfaced stainless steel bearing plate. This bearing minimizes the lateral friction force possible on piers and also minimizes the vertical offset at a joint between adjacent loaded and unloaded beams.

3) Design conditions. Maximum vertical gantry wheel loads were computed for the following conditions:

Condition A: Dead load of crane plus rated live load capacity and 30 lbs/sq.ft. wind load (Crane in static hoisting position). Group 2 loading.

Condition B: Dead load of crane, no live load and 50 lbs/sq.ft. wind load. Crane in static position, located on beam to produce maximum loading conditions on beams. Group 2 loading.

Condition C: Dead load of moving crane plus lower gate leaf weight and 30 lbs/sq.ft. wind load. Crane positioned on beam to produce maximum loading conditions on beams.

An impact allowance of ten percent shall be added to the wheel loads. Lateral and longitudinal loads applicable to Condition C only are ar follows:

Lateral thrust at the top of the rail shall be taken as ten percent of sum of the trolley weight and lifted load, with 3/4 of this amount distributed equally among the wheels at either side of the runway.

Lateral thrust shall be considered as acting in either direction normal to the runway rail but shall not be combined with impact.

Longitudinal thrust shall be taken as ten percent of the maximum vertical wheel loads.

The longitudinal and lateral thrust shall not be assumed to act simultaneously.

Condition C above was found to be the governing condition. The resulting loading, shear, moment and required reinforcement are shown for the tee gantry crane beam are shown on Plate V-83.

g. Roadway bridge bearings. The roadway bridge is a single lane reinforced concrete slab and stringer bridge, designed for a HS20-44 loading. There are three simply supported stringers, therefore six bearings, which results in stringer reactions of moderate intensity.

The computed change in length in an individual bridge span for a 100°F temperature change is approximately 0.3 inches. The above, incon- junction with the hostile environment offer justification for the utilization of elastomeric bridge bearings.

The roadway bridge is subject to being struck by waves during hurricanes. The stringers at the supporting piers have been placed in pockets in order to restrict the longitudinal and lateral movement. Waves striking the roadway bridge in addition to producing lateral loads will also produce a vertical force acting upward. This force has been conservatively estimated at 600 lbs/ft.^{2} The area over which this force or fraction thereof extends is debatable. Fortunately in a slab and stringer bridge, the stringers break the action of the wave and therefore reduce the extent over which the wave's upward force acts. As an example, assume a wave striking the side of the bridge, (for bridge details see V-85). One can visualize the wave force acting upward on the underside of the bridge slab outside of the first stringer and also on the soffit of the stringer. However, it seems very unlikely that the wave force would also act simultaneously in the area between the two stringers. Using the above wave force value over the surface area indicated above shows that a sufficient factor of safety exists against the possibility of lifting the access roadway bridge off its supports.

The design loading, shear, moments and maximum steel requirements for the interior stringer are shown on Plate V-84.

12. Vertical lift gates.

a. General. Eight sets of vertical lift gates of the fixed wheel, welded structural steel type are to be provided. To simplify the storage each gate is to be divided into two sections or leafs. Each leaf has four wheels with self-aligning spherical bearings. All gates are to have a clear width of 46'-0" and an overall height of 27'-6 1/4" (2 leafs of 13'-9 1/8" each). The leafs will be removed by means of a gantry crane and stored in slots at the top of the pier. The top leaf will be stored in the storage slot on the Gulf side with bottom elevation 3.0 and the bottom leaf will be dogged in the gate slots. Plates V-86 and V-87 show plan, elevation, and sections of the top and bottom leafs. Plate V-88 shows the installation assembly and seal details. On Plate V-89 the dogging device and on Plate V-90 the connecting device of the leafs are detailed. Plate V-91 shows the gate guide frame and storage assemblies.

b. Elements. Structurally each section of gate consists of the following elements:

1) skin plate

2) series of horizontal girders

3) vertical end girders

4) axles

Other items related to the gates include gate tracks, embedded metal and miscellaneous details. The design of these principal structural parts is discussed in the following paragraphs. For basic dimensions used in the design see calculations pg. 1

c. Loading conditions. Loading conditions investigated for the design of the gate are as follows:

1) Cases 1, 2, 3, and 6 for dynamic effect.

2) Cases 1, 2, 3, 4, 6, and 7 for maximum wheel load.

3) Case 6 for skin plate and beam design.

4) Case 8 for gate operation.

Cases 1 through 8 were investigated as shown in calculations pages 2 through 10 and are tabulated in Plate II-1. The maximum leaf loads and wheel reactions are shown in Figure 1.

d. General framing The skin plate is welded to the web of the girder and acts as a flange of the girder. Girder supporting the skin plate, frames into the end posts. The gate body is stiffened vertically with five continuous diaphragms.

In design of the girder, the width of the skin plate acting as effective flange area was determined by using the width - thickness ratio of 32. No allowance has been made for corrosion in the thickness of skin plate or other structural steel members of the gates, since adequate maintenance of the gates will be provided.

e. Skin plate. The skin plate was designed as a member spanning in the vertical direction across horizontal girders. The combined stresses of the skin plate and horizontal girder are shown in calculations on page 21.

f. Horizontal girders. The horizontal girders will support the skin plate directly as described in paragraph d. Web stiffners will be provided in pairs on the webs of the girders at intermediate points between diaphragms. Three-inch diameter drainage holes will be provided in the web of all girders.

g. Vertical end girders. The end girders are subjected to the following loads.

1) Reaction from horizontal girders.

2) Cantilevered axle reactions.

(The end girder together with the first interior diaphragm provide reactions resisting the cantilever action of the axles). Working stresses used in the design of these end girders were reduced to 50 % of normal to allow for overloading resulting from track irregularities. Loading diagrams (see figure 2, p. 25) shows loads, shears and moments used in the design of the end girder bottom leaf.

The detail design computations of the bottom leaf including analytical considerations, assumptions and maximum deflection is shown on pages 11 through 35.

h. Axles. Axles are designed for maximum computed radial wheel load acting simultaneously with an assumed side thrust equal to one third of the radial wheel load applied at the wheel tread. For analysis see Section VI Mechanical Design.

i. Tracks. Tracks will be provided in the gate slots on both upstream and downstream side of the gate. These tracks will consist of 171 lbs. "Bethlehem" rails mounted on structural steel beams embedded in concrete. Beam action of the structural steel, bearing on the relatively elastic concrete, is neglected. Compressive stresses are assumed distributed over areas bounded by

45 degree planes. The stresses in the rail and concrete are computed as shown in the computations on pages 39 through 44 and Figure 5 (page 45).

j. Miscellaneous details. Miscellaneous structural details include sills, guides, seals and lifting device. These details are discussed in the following paragraphs.

k. Sill. A stainless-clad steel plate welded to W6 x 25 structural steel shape and set in a block-out in the concrete of the control structure will act as the gate sill.

l. Guides. Horizontal movement of the gate parallel with the direction of flow must be small in order that the skin plates of the stacked gate leafs make suitable contact. The upstream and downstream tracks will be set to limit the total movement in this direction to 1/8" plus or minus 1/16" tolerance (see Figure 4, pg. 34). Sidewise movement of the gates will be limited to 1/2" plus or minus 1/4" tolerance for rails. This freedom of lateral movement will provide for thermal contraction or expansion and construction tolerances but will not cause excessive deflection of the rubber side seals.

m. Seals. The J-seals will be mounted on the Lake Pontchartram side of the gate along the sides and between the leafs. Along the sides, the seals will bear on stainless-clad steel plates welded to the edge angle. The side and top seals have an initial deflection of 1/16" minimum and 3/16" maximum.

n. Lifting device. The gate leafs will be raised to the required position by means of a gantry crane and lifting blocks which will engage hooks located in a vertical plane through the center of gravity at each end of the gate leafs. Design of the lifting hooks is shown on pages 54 and 55.

SECTION VI - MECHANICAL DESIGN

1. Gate wheel assembly.

a. Wheel. The wheels for the vertical lift gates will be of the double flanged type. Each wheel will be mounted on a self-aligning spherical roller bearing placed on a cantilever axle supported at the end girder of the gate. An end cap on the outer side of the wheel will serve as a seal to keep water out and to retain the bearing lubricant. The inner side of the wheel will have a shield containing a grease retainer serving a similar purpose.

1) Data for the wheel are as follows:

Load per wheel	205,000 lb.
Diameter of wheel	30 in.
Width of tread (Effective width)	3-1/2 in.
Thrust load (assumed)= 0.2 x radial load	41,000 lb.

Material: QQ-S-763a, Class 3 Brinell hardness (BHN) 325. For computations of the wheel see pages 35, 36, and 37 of computations in Section V.

b. Axle. The axle for the gate wheel will be tapered from the point of support at the end girder to the point of support at the diaphragm to reduce the weight. The diameter of the axle at the end girder will be made 10" to provide the required diameter of shoulder at the wheel for the roller bearing inner race. The material for the axle will be ASTM-A273-64. The load diagram is shown on figure 3 (p. 38 of the computation in Section V). The maximum bending moment occurs at the point of support at the end girder. The size of the axle is determined as shown in the computations pages 27 through 34 in Section V. The axle will be held in place at the diaphragm by a shoulder and a nut which will be placed on the threaded end of the shaft. The diameter of the threaded portion is found from the formula given in Leutwiler's "Machine Design", see computations p. 28 of Section V. The diameter of the thread will be made 4 5/8 inches. After the wheel has been adjusted to the proper position the axle will be locked in place by means of bars resting on flats milled on the axle and welded to the diaphragm. Then the nut will be drawn up tight and tack-welded in place.

c. Bearing. The bearings for the gate wheel will be self-aligning spherical roller bearings, as manufactured by, "Torrington Bearings" South Bend, Indiana 46621 or equal having a minimum radial static load capacity of 365,000 pounds and a lateral minimum thrust load 63,000 pounds.

1) The bearing inner race will be held in place by a shoulder on the axle and by a retainer plate bolted on the end of the axle. The bearing outer race will be held in place by a retainer on each side of the wheel.

2) The bearings will include provisions for lubrication and relubrication, as well space for reserve lubricant as recommended by bearing manufacturer.

d. Hoisting load. In analysis of the hoisting load a force of 11 lbs. per inch due to 3/16" seal deffection was assumed. A friction coefficient of 1.0 was used in computing the side and top seal friction forces. For complete analysis of the hoisting load see computations pages 51, 52, and 53 in Section V.

e. Dogging Device. A dogging device will be provided at each end of the gate, consisting of a horizontal beam being operated through gears with a handle at the top of the pier. This device is designed to withstand the actual load of two leafs, plus 50% impact. See computations pages 46, 47, 48, and 49, and Figure 6 in Section V.

2. Gantry Crane.

a. General.

1) Description. A self propelled gantry crane Plate VII will be provided to handle the gate with lifting blocks. The gantry crane will consist of a gantry structure, travelling along the control structure, a trolley travelling across the gantry structure and a double hoist mounted on the trolley.

2) Operation. To place a gate, the gantry crane will be positioned in the center of the span. The gate top section will be engaged with the lifting blocks, raised, transferred over and coupled to the lower gate section which will be dogged in the gate slot. The connected leaves will be raised slightly to permit disconnecting the dogging device, then lowered down to the sill beam. The lifting blocks will be disengaged and raised for handling another gate. Gate removal will be performed in reverse sequence. The gantry crane when not in use will be parked at the southwest end of the control structure next to the tool shed.

b. Speeds. Speeds were selected so that the net time required for hoisting and travelling of the gantry crane, from parking position to placement of the last of the eight gates, does not exceed 100 min. This is 1/3 of the 5 hours assigned for control structure closure, leaving 200 min. for positioning, engagement, coupling and miscellaneous operations. The gantry crane will be designed to provide operating speeds as follows:

1) Hoist.

(a) Raising one or two gate leaves simultaneously - 6 f.p.m.

(b) No-load raising and lowering lifting blocks - 18 f.p.m.

2) Trolley travel. The distance the trolley will travel will be about 18 feet, therefore, speed of 20 f.p.m. is selected.

3) Gantry travel. A speed of 45 f.p.m. is selected. For travelling against strong wind and easy exact spotting, a slow speed of 15 f.p.m. is selected.

c. Hoisting load.

See VI:A.7 and Figure 3.

d. Capacity.

1) Hoisting. Rated capacity at low hoisting speed, at the hooks of the lifting blocks will be approximately 5% higher than the weight of the two coupled gate leaves (62,000 lbs. each) or 65 tons. At high hoisting speed the crane will be required only to lift the unloaded lifting blocks.

2) Travel conditions. The driving mechanisms will be so designed and shall have such capacity that the movement will be steady while moving at high or low speed loaded with one gate

section against a wind load of 30 pounds as well as while moving at low speed unloaded against a wind load of 50 pounds.

e. Analytical Considerations and assupmtions

1) Loading cases. Structural parts will be designed for the following load combinations and unit stresses:

(a) Live load for trolley or gantry travel, impact, dead load, 30 pound per square foot wind load and tractive forces with resulting unit stresses not exceeding the basic unit stress.

(b) Live load for trolley or gantry travel, impact, dead load, 30 pound wind load and collision forces with resulting unit stresses not more than 25 percent in excess of the basic unit stress.

(c) Dead load, 30 pound wind load and the forces produced by the maximum torque of the hoist motor with the resulting unit stresses not exceeding 90 percent of the yield points of the materials involved.

(d) Live load for trolley travel, impact and 50 pound wind load with resulting unit stress not more than 25 percent in excess of the basic unit stress.

2) Wind loads. Wind loads will be applied to the horizontally projected area of the crane and gate. No shielding effect of one element by another will be considered where the distance between then exceeds four times the smaller projected dimension of the windward element. The projected area of the legs and sill beams will be increased by a factor of 60% to allow for shape conditions.

3) Impact. Impact will be taken at 10% of the live load.

4) Stability. The gantry crane will have a minimum factor of safety of 1.25 against overturning with the crane travelling unloaded and subject to a 50 pound wind load under each condition of loading stated above. The longitudinal sill beams will be filled with concrete to provide adequate counterweighting as required. The anchorage of the unloaded gantry crane in parking position will be designed for a minimum factor of safety of 1.75 both against overturning and sliding on the rails under a 50 pound wind load.

5) Stresses in structural members. Mechanical parts of the crane, including the lifting blocks and tractive drive, will be designed for rated loads, with a factor of safety of 5 based on the ultimate strength of the materials, provided that each part or component, including speed reducers but excluding wire rope, will be proportioned to withstand the stresses produced by maximum torque of the motors with resultant stresses not exceeding 75 percent of the yield point of the materials involved.

f. Components.

1) Gantry structure. The gantry will be constructed of built-up members fabricated by welding. Working area of the trolley will be covered by a weather-proof housing.

2) Hoist, drum and cables. The hoist shall be of the single motor, two drum type, connected through gearing and shafting. The hoist will provide for two speed operation of the raising and lowering motions. From each driven drum two strands of round wire rope will be carried over idler sheaves, reeved over the sheaves of the lifting block and inter-connected through a compensation sheave.

3) Trolley. The trolley will be driven by a motor mounted on the trolley frame connected to at least one driving wheel on each side of the trolley through gearing and shafting.

4) Gantry travel. The gantry will travel on eight wheels which will be installed on equalizing levers in protective housings. The gantry will be driven by one or more motors connected through gearing and shafting to not less than two wheels on each side of the crane.

g. Brakes.

1) General. Electric brakes for hoist and travel mechanism will be shoe type, spring set, with direct current magnet releases. The lifting link brakes shall be the disk type, spring set, with alternating current magnet release.

2) Hoist. The hoist will be provided with an electric brake. To prevent overhauling the hoist motor during lowering, positive means will be provided for energy absorption. This will consist of a mechanical load brake or a resistance bank which shall be connected to the power system during lowering load operation. If a mechanical load brake is provided it will have a

capacity to hold 1-1/2 times the rated hoist load, and will be designed to prevent the load from lowering unless the hoist motor is revolving under power in the lowering direction.

3) Trolley. The trolley will be provided with an electric brake.

4) Gantry. Each motor will be provided with both an electric brake and a hydraulically operated service brake which would be used to facilitate spotting of the crane. Sufficient braking capacity will be developed on the wheels to prevent movement along the track with a wind load of 50 pounds with a skidding coefficient of friction of 0.20 between the wheels and the track.

h. Lifting blocks. A pair of lifting blocks will be supported by the round wire rope for raising or lowering the gates. The blocks will travel on gate recesses, one at each end of the gate bay. Latching and unlatching will be accomplished by torque motors actuating small cable drum hoists. Each block will be supported by two parts of rope as the round rope will pass around sheaves on the block.

i. Mechanical accessories. Gantry and trolley will be provided with rail sweeps, safety lugs, and spring bumpers. For the parking position, a hand-operated anchorage will be provided. An air compressor will be installed on the platform of the engine-generators, for operation of coupling devices between the gate sections. Operator's cab will be of closed type for outdoor service, with good visibility to the work area and to gantry forward and backward travel directions. Access will be provided for all necessary areas by walkways, platforms, stairs and ladders. Handrailing will be provided where required.

SECTION VII - ELECTRICAL DESIGN

1. Gantry Crane.

a. General. The gantry crane will be self-powered by means of diesel-electric generating sets. Generators will be rated 480 volts 3-phase for power supply to motors. The lighting supply will be rated 208 Y/120 volts 3-phase obtained by means of a step-down transformer. A standby circuit will supply energy to equipment heaters, the engine battery chargers, and walkway and cab lights when the crane is in a standby condition. The power source for the standby circuit will also be used for illumination and heating of a small service building.

b. Motor sizes. Computations for the approximate sizes of the motors for the hoist, trolley travel and gantry travel are shown on figure 4.

c. Engine generators. Two diesel engine-generator sets will be provided, one for operating and one for standby service. The units shall be of adequate capacity for powering at one time any one of the motions for hoisting or travelling plus auxiliary equipment. They shall be installed in weatherproof cabinets on a platform arranged over the gulf side sill beam of the gantry. Each generator will be rated 250 kw, 80 percent pf., 480-volt 3-phase (figures 5, 6, and 7), and will be furnished with a static high speed excitation system.

d. Resistance bank. If a resistance bank is provided for positive control of lowering, the rating of this resistance bank will be sufficient to absorb the power generated by the hoist motor and at the same time provide a positive load on the generating unit equivalent to approximately 25 percent of the rating of the generator. The resistance bank will be rated for continuous operation.

e. Limit switches. Limit switches for both limits of hoisting will be of the travelling screw type, driven through gearing by the hoist. In addition, for controlling the upper limit of hoist motion, a normally closed contact, weighted level type limit switch will be provided, which will be actuated directly by the lifting blocks.

Limit switches for the trolley and gantry travel and interlockings will be of the lever actuated type, with spring return to the normal or unactuated position.

All limit switches will be reset by reversing the movement of the actuating device.

The following features for interlocking will be provided.

1) While any one of the motions are in use, other motions will be blocked.

2) High speed hoisting will be blocked when load exceeds limit set for this motion.

3) Gantry travel will be blocked until parking brake is fully released and until anchorage is disengaged.

Hoisting and travel motions will be provided with limit switches at both ends of travel.

f. Scheme of control. The scheme of control will be as discussed herein:

1) Gantry travel. The controls for the gantry travel motor will be the full magnetic type with two speed points and a drift point in each direction of travel. Drift positions will release the motor brake to permit travel control by means of the hydraulic foot brake.

2) Trolley travel. Due to the slow motion of the trolley a squirrel cage motor is considered adequate for this application. The master control switch will provide speed points and a drift point in each direction of operation.

3) Hoist. Due to the slow hoisting speeds either a multi-speed constant horsepower, squirrel cage motor or a wound rotor motor will be satisfactory.

4) Low speed hoisting, high speed hoisting, gantry travel and trolley travel will be controlled by separate operating handles or switches. They will provide spring return to the vertical "off" position. A thumb-operated off-position latch will be provided for each switch handle to prevent accidental operation. The switches will be mounted so as to require the handle to be moved in the direction of travel.

g. Equipment location.

1) Switchboard. A free-standing, metal-enclosed, switchboard will be provided. The switchboard will house the main breaker,

all 480-volt branch breakers, generator voltage regulator, exciter field rheostat, reverse and open phase relay, magnetic overload relays, and the crane master contractor.

2) Contractors. Directional and accelerating magnetic contractors and resistors will be mounted in suitable cabinets.

3) Lighting transformer. The lighting transformer will be mounted on the machinery deck.

4) Main control cabinet. A control cabinet will be provided on the wall in the operator's cab. Mounted on the panel of this cabinet will be the following:

(a) Line ammeter with 3-phase transfer switch;

(b) Line voltmeter with 3-phase transfer switch;

(c) Frequency meter;

(d) Ignition switch and starter button for the engine generator;

(e) Main contactor push button control station;

(f) Main contactor indicating light; and

(g) Parking brake indicating light.

5) Master control switches. The crane master control switches will be mounted on a control stand in the operator's cab.

6) Lighting panelboard. The lighting panelboard will be mounted on the wall in the operator's cab.

7) Automatic transfer switch. The automatic transfer switch will be wall mounted in the operator's cab above the lighting panel.

8) Battery charger. The battery charger will be wall mounted in the operator's cab.

h. Lighting.

1) Fixtures. Illumination will be by means of incandescent fixtures. Vapor proof type units will be furnished to light stairs and platforms; open reflector units mounted on swivels will be used for bridge lighting. Lighting under the bridge will be with high-bay units having glass reflectors and shock absorbers. At the base of the gantry, wide-beam units will be mounted to illuminate the roadway during gantry travel.

2) Receptacles. Weatherproof single devices with threaded caps will be furnished as convenience outlets along the crane bridge. Raintight receptacles will be furnished at the base. A reverse service raintight receptacle will be furnished for the standby power connection.

i. Figures 1 through 9.

Figure 1

Wheel bearing friction load.

$F_b = Pf_b \times \frac{R_2}{R_1}$ Where F_b = frictional load, lb.

f_b = coefficient of friction, bearing = 0.002

R_2 = radius of bearing roller race, in = 6.4 in.

R_1 = radius of wheel = 15 in.

P = total gate load, lb.

$$F_b = 200{,}000 \times 0.002 \times \frac{[6.4]}{[\ 15]} = 170 \text{ lb.}$$

Wheel rolling friction load.

$F_r = \frac{Pfr}{R_1}$; Where F_r = frictional load, lb.

f_r = coefficient of friction, rolling = 0.06

R_1 = radius of wheel, in. = 15 in.

$$F_r = \frac{400.000 \times 0.06}{15} = 800 \text{ lb.}$$

Figure 2

Gate seal friction load.

F_s = pressure x contact area x coefficient of friction, lb.

Head = 2.5 ft. = 1.08 psi

Contact area = 1-1/2 (assumed) x [(2 x 27.5 + 46.3) x 12]
= 1520 square in.

Coefficient of friction - 1.0 (assumed)

F_s = 1.08 x 1520 x 1.0 = 1640 lb.

Gate seal deflection load.

F_d = load x length seal
load = 11 lb/in to deflect seal 1/4 in.

Fd = 11 x 1220 = 13,400 lb.

Figure 2 (Continued)

Lifting blocks.

Estimated weight of 2 lifting blocks = 3500 lbs.

Submerged weight of gates.

$$130{,}000 \times \frac{(490 - 62.5)}{490} = 113{,}000 \text{ lbs.}$$

Figure 3

Hoisting load.

	Submerged	Dry
Wheel friction	800 lb.	-
Bearing friction	170 lb.	-
Gate seal friction	1,640 lb.	-
Gate seal deflection	13,400 lb.	13,400 lb.
	16,010 lb.	13,400 lb.
Gate weight - 2 leaves	113,000 lb.	130,000 lb.
Lifting blocks	3,500 lb.	3,500 lb.
Wire ropes	600 lb.	600 lb.
	133,100 lb.	147,500 lb.
Rated load (incl. allowance of approximately 5%)	140,000 lb.	155,000 lb.

Lower load.

Wheel friction	800 lb.	
Bearing friction	170 lb.	
Gate seal friction	1,640 lb.	
Gate seal deflection	13,400 lb.	
	16,010 lb.	upward
Gate weight - 2 leaves	113,000 lb.	
Lifting blocks	3,500 lb.	
	116,500 lb.	

$$\text{Ratio} = \frac{116{,}500}{16{,}010} = 7.25 \text{ which is ample for safe closing.}$$

Figure 4

GANTRY CRANE - MOTOR SIZE COMPUTATIONS

Crane hoist motor size:

Load = 155,000 lbs.
Speed = 6 fpm
Overall efficiency = 0.06

$$HP = \frac{155,000 \times 6}{33,000 \times 0.6} = 47.0$$

Trolley motor size:
Wind load on one gate
leaf of 30 psf = 21,000 lb.
Gate weight (one leaf) = 65,000 lb.
Lifting blocks - 3500 lb.
Wire rope = 600 lb.
Trolley weight = 75,000 lb.
Speed - 20 fpm
Overall efficiency = 0.70
Wheel friction = 0.005
Bearing friction = 0.025

$$HP = \frac{[21,000 + (144,100 \times 0.03)]}{33,000 \times 0.7} \times 20 = 21.9$$

Gantry motor size:

Case A; (Low speed travel; no load)

Wind load on gantry and
lifting blocks of 50 psf = 38,000 lb.
Trolley weight = 75,000 lb.
Gantry weight = 180,000 lb.
Lifting blocks - 3,500 lb.
Wire rope = 600 lb.
Speed = 15 fpm
Overall efficiency = 0.70
Wheel friction = 0.005
Bearing friction = 0.025

$$HP = \frac{38,000 + (259,100 \times 0.03)}{33,000 \times 0.7} \times 15 = 29.8$$

Figure 4 (Continued)

Case B: (Low speed travel; load)

Wind load on gate, gantry and
lifting blocks of 30 psf = 24,700 lb.
Trolley weight = 75,000 lb.
Gantry weight = 180,000 lb.
Lifting blocks = 3,500 lb.
Wire rope = 600 lb.
Gate weight (one leaf) = 65,000 lb.
Speed = 15 fpm
Overall efficiency = 0.70
Wheel friction = 0.005
Bearing friction = 0.025

$$HP = \frac{24,700 + (324,100 \times 0.03)}{33,000 \times 0.70} \times 15 = 22.4$$

Case C: (High speed travel; no load)

Wind load on gantry and
lifting blocks of 30 pst = 22,800 lb.
Trolley weight = 75,000 lb.
Gantry weight = 180,000 lb.
Lifting blocks - 3500 lb.
Wire rope - 600 lb.
Speed = 45 fpm
Overall efficiency = 0.70
Wheel friction = 0.005
Bearing friction = 0.025

$$HP = \frac{22,800 + (259,100 \times 0.03)}{33,000 \times 0.70} \times 45 = 59.5$$

Figure 5

CHEF MENTEUR PASS COMPLEX
GANTRY CRANE - ELECTRICAL CALCULATIONS

Voltage drop due to motor starting on limited generating systems.

1. The following method of determining the voltage drop is based on the following assumptions:

   (a) The power factor of the applied load will be 0.40 or less when starting the squirrel cage motor.

   (b) Prior to load application the generator is operating at rated voltage.

   (c) Starting current of the squirrel cage motor is approximately 5.5 times normal.

   (d) Generator transient reactance, X'_d = 25%

Figure 5 (Continued)

2. Maximum running load in Kva

2 Gantry motors	-	31.8 Kva ea	=	63.6 Kva
Hoist motor	-	52.0 Kva	=	52.0 Kva
Cab heater	-	4.0 Kva	=	4.0 Kva
Lighting transformer	-	9.0 Kva	=	9.0 Kva
			Total	128.6 Kva

Figure 6

CHEF MENTEUR PASS COMPLEX
GANTRY CRANE - ELECTRICAL CALCULATIONS

The trolley motor load is not included in the maximum load since this load operation will usually alternate with either the hoist or the gantry and sufficient power will be available to power this load.

3. Voltage drop-(no initial load)

(A) Generator rated: 312 Kva, 0.8 P.F., 480 volts, 3 Ø

Generator reactance = X'_d = Transient rectance - 25%

Assuming $Z_{Gen} = X'_d$

$$X'_d = \frac{10(\%X)(KV)^2}{\text{KVA base}} = \frac{10(25)(.231)}{312} = .185 \text{ ohms}$$

For the two 30 HP gantry motors operating at 460 volts
Locked rotor Kva = $\frac{5.5(40)\ (1.732)\ 460}{1000} \times 2 = 350$ Kva

$$\text{Start Kva} = \frac{(480)^2}{(460)^2} \times 350 = 381 \text{ Kva}$$

Assume equivalent motor x = z

$$X_{ms} = \frac{1000\ (KV)^2}{SKVA} = \frac{1000 \times (.480)^2}{381} = .605 \text{ ohms}$$

Figure 7

CHEF MENTEUR PASS COMPLEX
GANTRY CRANE - ELECTRICAL CALCULATIONS

Generator	X'_d = 0.185 phms
Motor	X_{ms} = .605
Total	X_t = 0.790

$$\% \text{ voltage drop} = 100 \left(1 - \frac{Xms}{Xt}\right) = \left(1 - \frac{.605}{.790}\right) = 23.5\%$$

(B) Alternative claculations. Starting Kva (SKVA) from (A) above = 381. SKVA in % of generator KVA rating $= 100 \times \frac{381}{312} = 122$. Voltage drop, figure 8 no initial

load = 33 x 0.6 (high speed excitation system) = 19.8%

4. Voltage drop (initial load)

Assuming a 12-1/2 % initial load on the generator when the gantry motors are started the voltage drops from figures 8 and 9 are as follows:

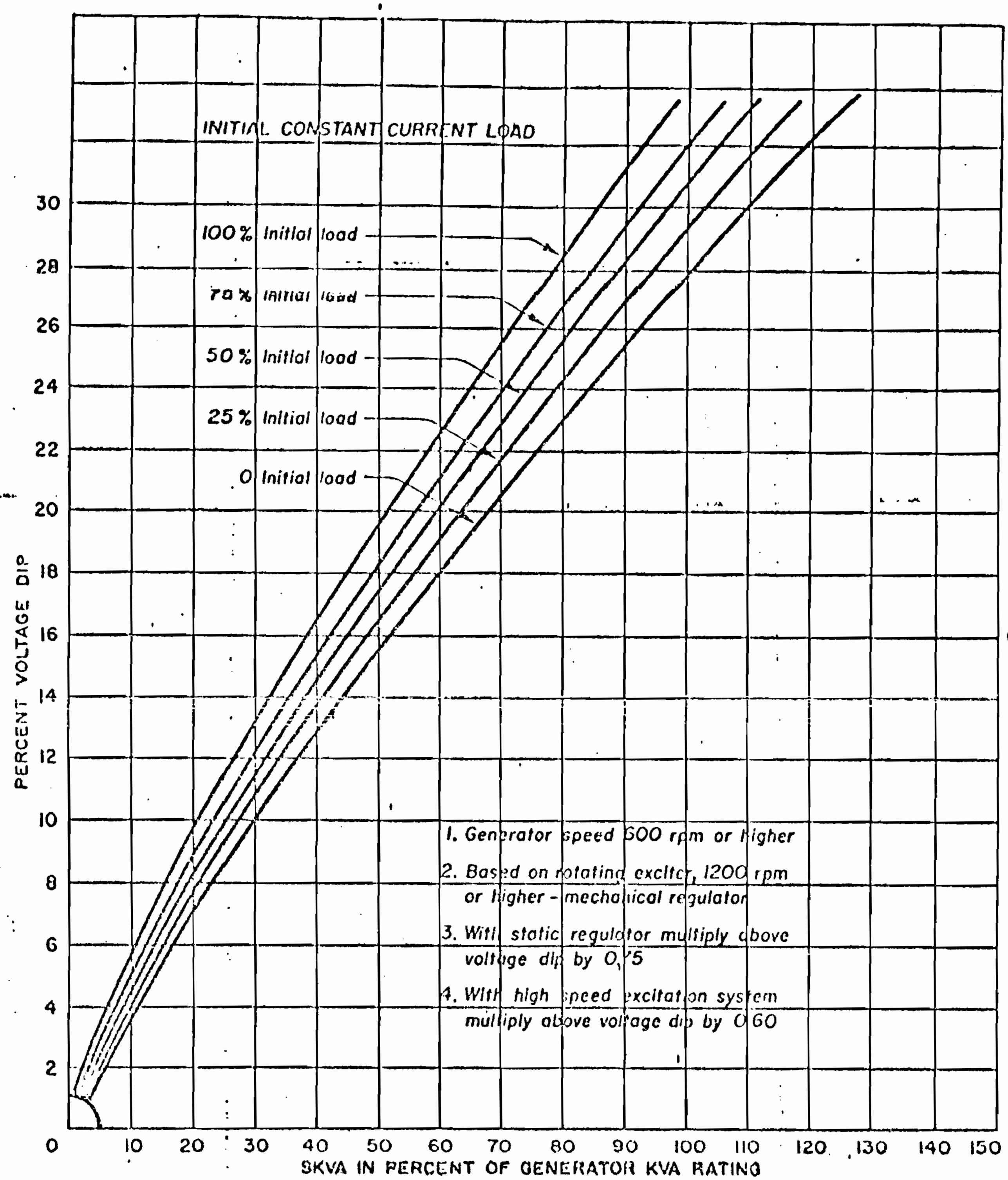
SKVA in % of generator Kva rating $= 100 \frac{(381)}{(312)} = 122\%$

For initial constant current load, voltage drop =
34 x 0.6 = 20.4

For initial load of loaded motors, voltage drop =
35 x 0. = 21.0

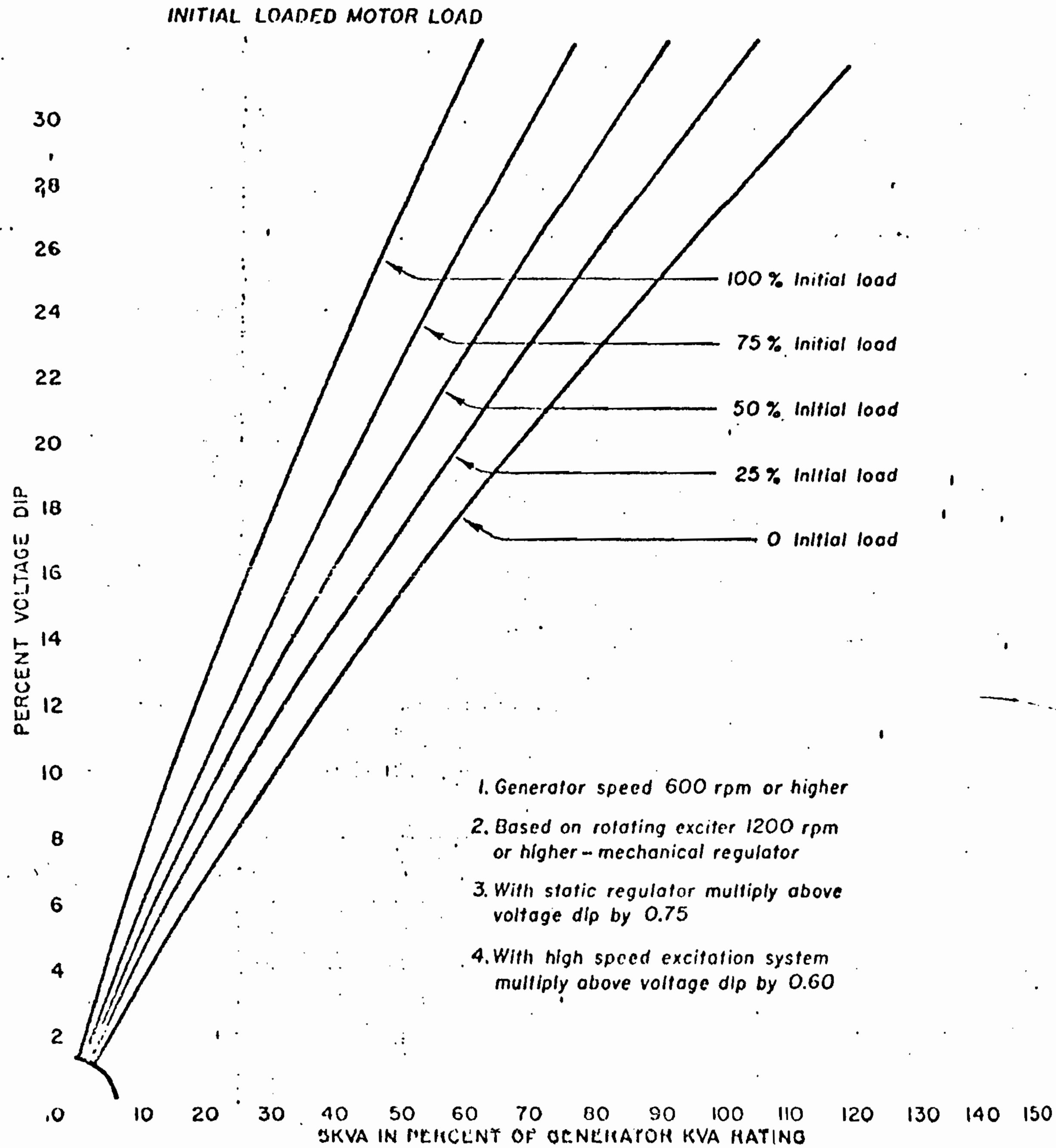
5. Generator rating

A rating 312 Kva 250 KW 0.8 PF, 480 volt, 3 Ø, 60 CPS will be used to prevent drop out of the contactors connected to the system, stalling of running motors and improve speed of response to the starting inrush and resultant voltage drop. The rating is standard with generator manufacturers.



Generator voltage dip with initial constant current load. *

* E-M Synchronizer. Special Issue. 200 SYN-59. Bulletin published by Electrical Machinery Mfg Co., 1961



Generator voltage dip with initial load of loaded motors.*

* E-M Synchronizer, Special Issue, 200 SYN-59, Bulletin published by Electrical Machinery Mfg. Co., 1961.

FIGURE 9

SECTION VIII—APPURTENANCES

1. Access roadway bridge. A bridge across the control structure on the Lake side of the control gates is provided to allow unimpeded vehicl traffic along the barrier (Plate V-85). The bridge is of a monolithic reinforced concrete beam and slab construction designed for an HS20-44 highway loading. Investigation indicated the reinforced concrete bridge had a lower cost than a prestressed concrete alternative.

The bridge is composed of 22 sections 51 feet long, simply supporte on the control structure piers. At the pier supports the beams are seated in pockets to develop a resistance to lateral loads imposed by wave action. Bridge bearings are 5/8" neoprene elastomer pads. The roadway is 12'-0" wide with a 1 3/4" crown, Precast concrete drain scuppers are provided on both sides of the roadway. The curbs on both sides of the roadway provide an 18" walkway. A 12 inch thick concrete parapet mounted by a 3 1/2 inch round aluminum pipe hand rail offers a safety barrier to both vehicles and pedestrians. The top of the parapet is at El. 16.25 and is the highest fixed element subject to wave loads on the control structure.

2. Walkways. The tee flanges of the gantry beams afford a continuous surface be used as walkways for the operation and maintenance personnel. The Gulf side gantry beam will have an aluminum handrail at the outer edge of its flange. The clear walking width between the handrail and gantry rail would be approximately 5 feet. The other gantry beam will have 3 feet clear, with the adjacent bridge parapet and rail providing a safety barrier.

3. Gantry rails. Gantry crane rails will be 105 lb. rail sections placed along the axis of the gantry crane beams. The gantry crane rail will be placed on 5/8" sole plates spaced 2'-0" on centers. Adjustment of the rail elevation will be by means of embedded anchor bolt sets with leveling nuts beneath the sole plate. The rail shall be fastened to the sole plate by standard rail clips. A nonshrink grout shall be placed beneath the sole plates and the rail along its entirety.

4. Fence. A six foot aluminum coated steel wire fabric fence with aluminum coated posts, braces and ties shall be provided at each end of the control structure with the alignment being as shown on Plate I-2. A two leaf gate of the same material as the fence shall be provided centered on the levee centerline at each end of the control structure. The gate shall provide a clear opening of 14 feet.

5. Storage building. A pre-engineered-prefabricated self supporting metal building complying with the requirements of the MBMA (Metal Building Manufacturers Association) will be located on the Northwest levee section abutting the control structure. The storage building will have a plan dimension approximately 10 feet x 10 feet. The floor of the building shall be a cast in place concrete slab monolithic with a continuous foundation wall.

SECTION IX - ESTIMATE OF COST

1. General. Based on December 1973 price levels, the estimated first cost of the control structure, control channel, Chef Menteur Pass Closure Dam, and tie-in levees is $19,442,000. Details of estimated first costs are shown in Table X-1.

2. Comparison of estimates. The current estimate of $19,442,000 represents an increase of $9,126,500 over the latest estimate presented in Design Memorandum No. 2 - General Design Supplement No. 3 - Chef Menteur Pass Complex dated May, 1969. The increase reflects the added cost for (1) the increases in price level between May, 1969 and December, 1973; and (2) variations of quantities of various items as a result of general refinements in the estimate based on the availability of more detailed information.

LAKE PONTCHARTRAIN BARRIER PLAN
DETAIL DESIGN MEMORANDUM NO. 7
CHEF MENTEUR PASS CONTROL STRUCTURE AND CLOSURE DAM

TABLE IX-1
COST EXTIMATE
December 1974 price levels

Cost Account No.	Item	Estimated quantity	Unit	Unit Price	Estimated amount
	Construction				
11	Levees and floodwalls				
	Levee fill (1st lift)				
	Chef Menteur Pass closure	3,616,000	c.y.	0.60	$ 2,169,600
	Levee fill (2d lift)				
	Chef Menteur Pass closure	234,900	c.y.	1.35	317,115
	Levee fill (3d lift)				
	Chef Menteur Pass closure	23,000	c.y.	0.75	17,250
	Fill shaping (4th lift)				
	Chef Menteur Pass closure	23,000	c.y.	0.75	17,250
	Shell (wave protection)				
	Chef Menteur Pass closure	24,400	c.y.	5.30	129,320
	Riprap				
	Chef Menteur Pass closure	114,240	tons	12.00	1,370,880
	Fertilizing and Seeding	9	acre	160.00	1,440
	Shell (in place for roadway)				
	Chef Menteur Pass closure	510	c.y.	7.50	3,825
	Subtotal, Levees and floodwalls				4,026,680
30	Engineering and design, 12%±				483,000
31	Supervision and administration, 7.2%±				289,320
	Total, Levees and floodwalls				$ 4,799,000

TABLE IX-1 (cont'd)

Cost Acct. No.	Item	Estimated quantity	Unit	Unit Price	Estimated Amount
15	Control Structure				
	Structure excavation	195,000	c.y.	2.30	448,500
	Backfill	30,000	c.y.	3.00	90,000
	Dewatering	1	job		664,400
	Filter gravel	1,570	c.y.	15.00	23,550
	Filter sand	1,710	c.y.	15.00	25,650
	Riprap in channel	60,000	tons	19.60	1,176,000
	Gravel	9,300	c.y.	15.00	139,500
	Sand	9,300	c.y.	15.00	139,500
	Timber piles 12"Ø - 50' long				
	Furnish and drive	104,250	L.F.	3.00	312,750
	Redrive	88,650	L.F.	1.50	132,975
	Teco tension connectors	9,500	each	6.00	57,000
	Steel H Riles				
	HP 12 @ 74#	23,520	L.F.	25.90	690,168
	HP 12 @ 53#	87,000	L.F.	18.55	1,613,850
	Steel sheetpiles				
	MP-113	4,400	S.F.	6.50	28,600
	MZ-27	13,500	S.F.	6.50	87,750
	Concrete				
	Bridge	1,510	c.y.	150.00	226,500
	Gantry beam	1,619	c.y.	150.00	242,850
	Pass section base slab	5,827	c.y.	53.00	308,831
	Barrier and abutment footings	1,991	c.y.	53.00	105,523
	Pass section piers	2,273	c.y.	75.00	170,475
	Barrier section piers and abutment	2,061	c.y.	75.00	154,575
	Barrier wall	1,625	c.y.	75.00	121,875
	Steel reinforcing	1,912,000	lbs	0.23	439,760
	Water stop	900	L.F.	9.80	8,820
	Pipe handrail 1 1/2" diameter	1,120	L.F.	15.00	16,800
	Parapet railing	2,240	L.F.	10.50	23,520
	Crane rails 105 lbs/ft	78,750	lbs	0.70	55,125
	Miscellaneous metals	45,000	lbs	0.90	40,500
	Gantry crane-furnished and erected	1	each		525,000
	Gates	875,000	lbs	1.30	1,137,500
	Gates guides, rails, dogging devices	289,000	lbs	1.00	289,000
	Lighting	1	lump sum		23,000

TABLE IX-1 (cont'd)

Cost acct. no.	Item	Estimated quantity	Unit	Unit Price	Estimated Amount
	Fences	1,900	L.F.	9.00	17,100
	Building	1	lump sum		3,025
	Subtotal, control structure				9,458,972
	Contingencies, 20%±				1,892,028
	Subtotal, control structure				11,351,000
30	Engineering and design, 12%±				1,362,100
31	Supervision and administration, 7.2%±				817,900
	Subtotal, control structure				13,531,000
09	Channels and Canals				
	Control channel excavation	1,872,800	c.y.	0.36 $	674,208
	Contingencies, 20%±				134,792
	Subtotal, control channel				809,000
30	Engineering and design, 12%±				97,000
31	Supervision and administration, 7.2%±				58,000
	Subtotal, control channel				964,000
	Total, control structure and channel				14,495,000
	Lands and Damages				
01	Control channel permanent right-of-way	20	acre	3,750	75,000
	Control channel permanent right-of-way	145	acre	300	43,500
	Subtotal, lands and damages				118,500
	Contingencies, 15%±				17,800
	Subtotal, lands and damages				136,300
	Acquisition cost by others				700
	Total, Lands and damages				137,000
02	Relocations				
	4" gas line	1	lump sum		5,100
	Telephone line	1	lump sum		2,500
	Subtotal, Relocations			$	7,500

TABLE IX-1 (cont'd)

Cost acct. no.	Item	Estimated quantity	Unit	Unit Price	Estimated Amount
	Contingencies, 20%±				1,500
	Subtotal, Relocations				$ 9,000
30	Engineering and design, 12%±				1,100
31	Supervision and administration, 7.2%±				900
	Total, Relocations				$ 11,000

TABLE IX-2

CHEF MENTEUR PASS COMPLEX
RECAPITULATION OF FIRST COST

Cost Account No.	Item	Estimated Amount
09	Channels and canals	$ 809,000
15	Control and structure	11,351,000
11	Levees and floodwalls	4,026,680
30	Engineering and design	1,943,200
31	Supervision and administration	1,166,120
01	Lands and damages	137,000
02	Relocations	9,000
	Total	19,442,000

TABLE IX-3

CHEF MENTEUR PASS COMPLEX
COMPARISON OF ESTIMATES

	Feature	GDM No. 2 Supp. No. 3 May 69	DDM No. 7 Dec. 73
15	Control structure	$ 5,336,000	$ 11,351,000
09	Channels and canals	540,000	809,000
11	Levees and floodwalls	2,683,700	4,026,680

TABLE IX-3 (cont'd)

30	Engineering & design	$ 1,027,200	$ 1,943,200
31	Supervision & administration	616,300	1,166,120
	Subtotal	10,203,200	19,296,000
01	Lands and damages	$ 101,300	137,000
02	Relocations	9,000	9,000
	Subtotal	$ 110,300	$ 146,000
	TOTAL	$10,313,500	$19,442,000

[illegible]

APPENDIX A

Detailed Description of Pass Section

Pile Foundation Analysis

COPY

HARZA ENGINEERING COMPANY
CONSULTING ENGINEERS
CHICAGO, ILLINOIS 60606

March 19, 1971

Air Mail - In Duplicate

District Engineer
U. S. Army Engineer District
Post Office Box 60267
New Orleans, Louisiana 70160

Attention: Mr. William B. Seale

Subject: Chef Menteur Pass Complex
Revised Control Structure Foundation
Design Contract No. DACW 29-68-C-0010

Gentlemen:

We are enclosing herewith, for your review and approval, results of two complete pile foundation designs for the control structure. One design is for 12-inch untreated timber piles, and the other is for 12 x 12 inch precast, prestressed concrete piles. This submission is in accordance with the request stated in your letter of February 9 and it supersedes and replaces our letters of December 23, 1970 and January 6, 1971.

The foundation was designed as two separate components: one for a 25 foot wide strip centered about each of the nine piers and the second for the eight 26-foot wide bays between the pier strips. The loads transmitted to the control structure by the gates and bridge were assumed to be taken by the pier strips. The loads taken by the bay strips arise from water loads directly on these strips, plus the weight of the strip itself. The design of both strips was based upon static, dynamic and construction condition loading cases. The loadings thus considered were Cases 4 and 7 (static), Cases 1, 2, 3 and 6 (dynamic) and 9 (construction). Uplift was alternatively considered as uniform and trapezoidal for each case except Case 9 which had no uplift at all. A summary of all cases is attached as Enclosure 1.

Following completion of the design, values of the subgrade modulus were assumed both higher and lower than the previously agreed upon value of 160 for concrete and 80 for timber piles. From these alternative modulus values, actual and allowable transverse loads and deflections were computed and plotted against the subgrade moduli. The computer printouts for all these runs are submitted as Enclosures 2 through 9. For convenience, Case 9 (construction) has been included with the dynamic case analyses, since a 33 1/3% overstress is permitted for both construction and dynamic loadings. Trapezoidal uplift is indexed by .1 and uniform by .2 following the case number (as Case 4.1 or 6.2). For Case 9 the index .1 indicates wind load from the gulf side and .2 indicates wind load from the lake side. Within each enclosure are five runs, one for each of the five modulus values. The plots for each combination of pier and bay strips and concrete and timber piles are submitted as Enclosures 10 through 17.

COPY

Prior to receipt of your letter of February 9, 1971, we had modified the computer program so that it exactly duplicated the results of the pile design submitted to us in Figure 4-8 through 4-12 of General Design Memorandum No. 2, Advance Supplement, Inner Harbor Navigation Canal West Levee, Florida Avenue to IHNC Lock. As a result of programming changes requested in your letter of February 9, 1971, results obtained from the computer program no longer duplicate those in Figure 4-12; this is described in more detail in the Appendix to this letter. The Appendix also describes other modifications made to the computer program DATA 2/K29HRN which you kindly furnished us.

The arrangements of piles in each design (wood and concrete) are shown on Enclosures 18 and 19. We have maintained center-to-center spacings, as previously approved by you, of 2.5 feet minimum, 10 feet maximum in bay strips and 7 feet maximum in pier strips.

To select a pile material, cost estimates were made for the nine pier and eight bay strips, excluding contingencies, for both pile types. The computation sheets for these estimates are submitted as Enclosure 20. * The cost of the 104,250 linear feet (2085 piles) of timber piles is $312,000 and of the 50,550 linear feet (837 piles) of concrete piles is $341,000. This result still supports our earlier recommendation which was based on a study made under a different set of assumptions.

Your early decision as to the pile type to be used is necessary to permit us to complete design work now many months behind schedule. Your cooperation will be appreciated.

Very truly yours,

HARZA ENGINEERING COMPANY

Andrew Eberhardt
Vice President

cc: Burk and Associates, Inc.

COPY

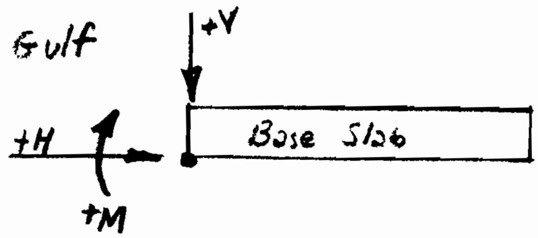
Enclosures:

1 - Loading Case Summary
2 - Computer Results - Concrete Piles - Pier Strips - Static Loads
3 - Computer Results - Concrete Piles - Pier Strips - Dynamic Loads
4 - Computer Results - Concrete Piles - Bay Strips - Static Loads
5 - Computer Results - Concrete Piles - Bay Strips - Dynamic Loads
6 - Computer Results - Timber Piles - Pier Strips - Static Loads
7 - Computer Results - Timber Piles - Pier Strips - Dynamic Loads
8 - Computer Results - Timber Piles - Bay Strips - Static Loads
9 - Computer Results - Timber Piles - Bay Strips - Dynamic Loads
10 - Allowable vs. Actual Transverse Loads and Deflections - Concrete Piles - Pier Strips - Static Loads
11 - Allowable vs. Actual Transverse Loads and Deflections - Concrete Piles - Pier Strips - Dynamic Loads
12 - Allowable vs. Actual Transverse Loads and Deflections - Concrete Piles - Bay Strips - Static Loads
13 - Allowable vs. Actual Transverse Loads and Deflections - Concrete Piles - Bay Strips - Dynamic Loads
14 - Allowable vs. Actual Transverse Loads and Deflections - Timber Piles - Pier Strips - Static Loads
15 - Allowable vs. Actual Transverse Loads and Deflections - Timber Piles - Pier Strips - Dynamic Loads
16 - Allowable vs. Actual Transverse Loads and Deflections - Timber Piles - Bay Strips - Static Loads
17 - Allowable vs. Actual Transverse Loads and Deflections - Timber Piles - Bay Strips - Dynamic Loads
18 - Concrete Pile Arrangement
19 - Timber Pile Arrangement
20 - Economic Pile Selection
Appendix - Modifications to Computer Program Data 2/K29HRN

FOR USE ON U.S. GOVERNMENT WORK ONLY ENCLOSURE 1

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT PILE DESIGN, CONTROL STRUCTURE	PROJECT CHEF MENTEUR
	COMPUTED EMC CHECKED	FILE NO. 453B1
		DATE MAR. 8, 1971 PAGE 1 OF 2 PAGES

LOADING CASE SUMMARY



Pier Strips

	Loading Case No.	ΣM (k-ft)	ΣV (k)	ΣH (k)
Static	4.1	100219	2591	1495
	4.2	102919	2555	1495
	7.1	52226	2293	-1410
	7.2	49136	2295	-1410
Dynamic (allows 33 1/3 % overstress)				
	1.1	131232	2203	2562
	1.2	132782	2186	2562
	2.1	135808	2263	2911
	2.2	138468	2241	2911
	3.1	130307	2295	2730
	3.2	133077	2272	2730
	6.1	25386	2181	-3025
	6.2	9716	2203	-3025
	9.1	118455	3522	83
	9.2	113684	3522	-83

Bay Strips

	Loading Case No.	ΣM (k-ft)	ΣV (k)	ΣH (k)
Static	4.1	20700	788	140
	4.2	23100	753	140
	7.1	26753	798	-129
	7.2	24344	839	-129

FOR USE ON U.S. GOVERNMENT WORK ONLY ENCLOSURE 10

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONCRETE PILE SUMMARY	PROJECT CHEF MENTEUR
	COMPUTED EMC CHECKED	FILE NO. 453B1
		DATE March 10 1971 PAGE 1 OF 3 PAGES

SUMMARY OF CRITICAL TRANSVERSE PILE LOADS

Critical is defined as the greatest ratio of actual to allowable

Design strip	Loadings	Subgrade Modulus (psi)	Tension All. load (k)	Tension Act. load (k)	Compression All. load (k)	Compression Act. load (k)
		50	2.897	.288	.823	.183
		100	3.477	.451	1.015	.305
PIER	STATIC	160	3.946	.680	1.183	.430
		200	4.195	.800	1.277	.506
		400	5.106	1.323	1.652	.830
		50	2.234	.503	.979	.337
		100	2.721	.841	1.176	.560
PIER	DYNAMIC	160	3.133	1.189	1.335	.787
		200	3.360	1.400	1.420	.923
		400	4.235	2.315	1.912	1.671
		50	No piles in Tension		4.022	.117
		100	"	"	4.782	.195
BAY	STATIC	160	"	"	5.377	.276
		200	"	"	5.685	.325
		400	"	"	6.157	.538
		50	4.338	.017	5.525	.148
		100	5.158	.028	6.569	.247
BAY	DYNAMIC	160	5.800	.037	7.387	.349
		200	6.132	.046	7.810	.411
		400	7.289	.075	9.283	.679

SUMMARY OF CRITICAL TRANSVERSE PILE DEFLECTIONS

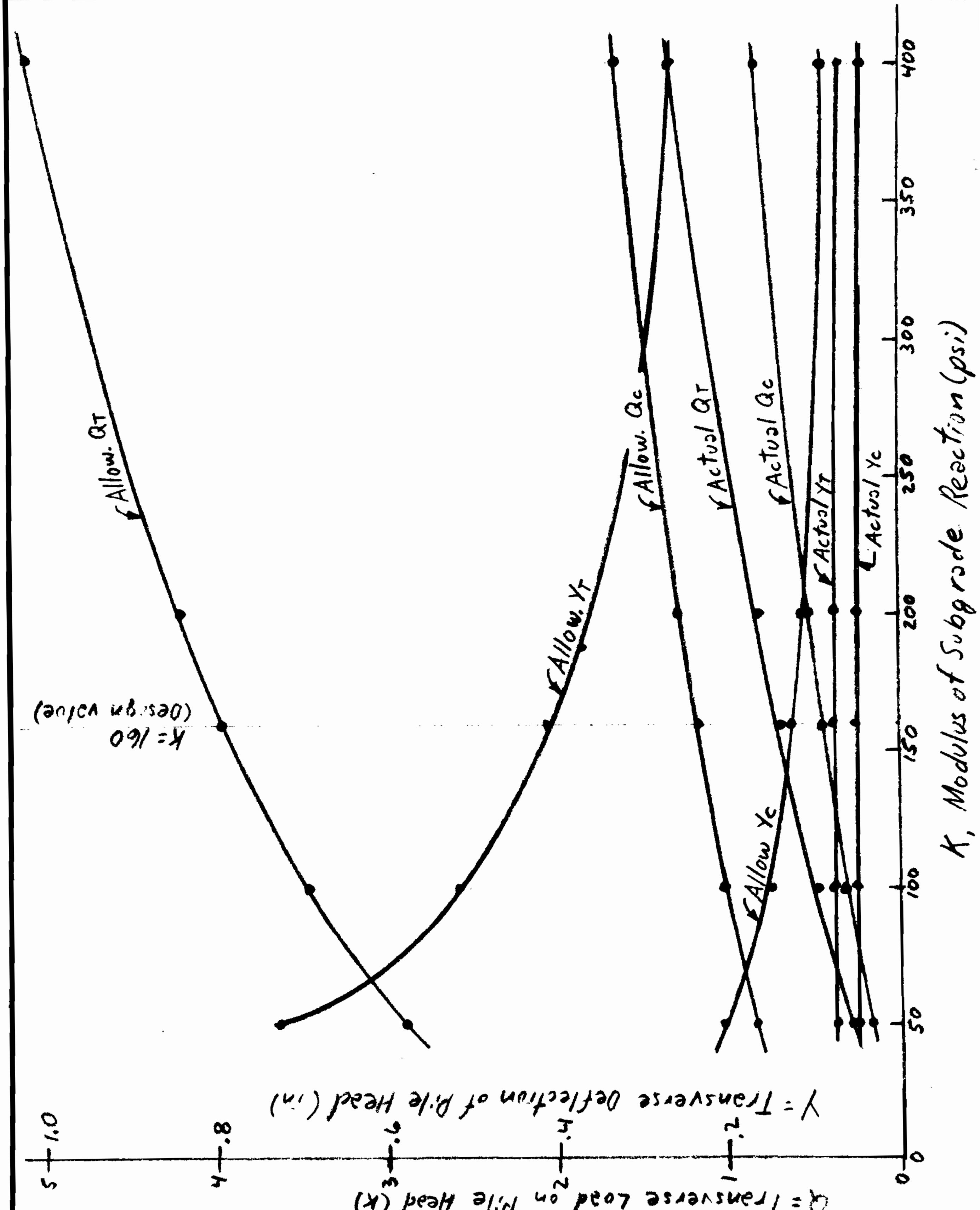
Critical is defined as the greatest ratio of actual to allowable

Design strip	Loadings	Subgrade Modulus (psi)	Tension All. defl. (in)	Tension Act. defl. (in)	Compression All. defl. (in)	Compression Act. defl. (in)
PIER	STATIC	50	.722	.074	.205	.047
		100	.515	.073	.150	.047
		160	.411	.073	.123	.046
		200	.370	.073	.113	.046
		400	.268	.071	.087	.045
PIER	DYNAMIC	50	.557	.129	.244	.086
		100	.403	.128	.174	.085
		160	.326	.127	.139	.084
		200	.296	.127	.125	.084
		400	.222	.125	.100	.090
BAY	STATIC	50	No piles in tension		1.003	.030
		100			.709	.030
		160			.560	.030
		200			.501	.029
		400			.354	.029
BAY	DYNAMIC	50	1.081	.004	1.377	.038
		100	.764	.004	.974	.038
		160	.604	.004	.770	.037
		200	.540	.004	.688	.037
		400	.382	.004	.486	.037

FOR USE ON U.S. GOVERNMENT WORK ONLY

ENCLOSURE 10

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y Vs. K - CONCRETE PIER STRIP - STATIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE No. 453B2 DATE Mar. 10, 1971 PAGE 3 OF 3 PAGES



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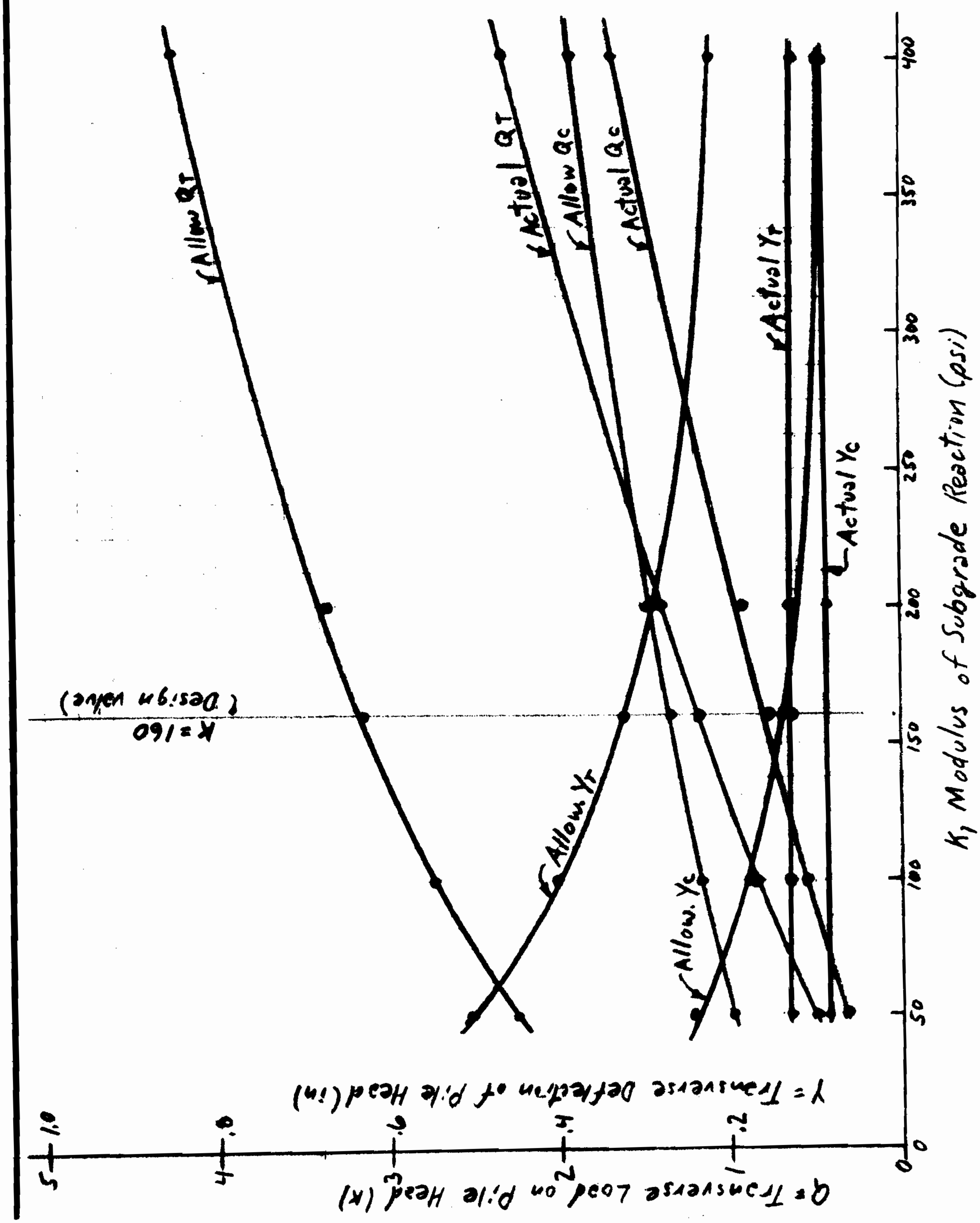
ENCLOSURE 11

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - CONCRETE PIER STRIP - DYNAMIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE No. 453B1 DATE Mar. 10, 1971 PAGE OF PAGES



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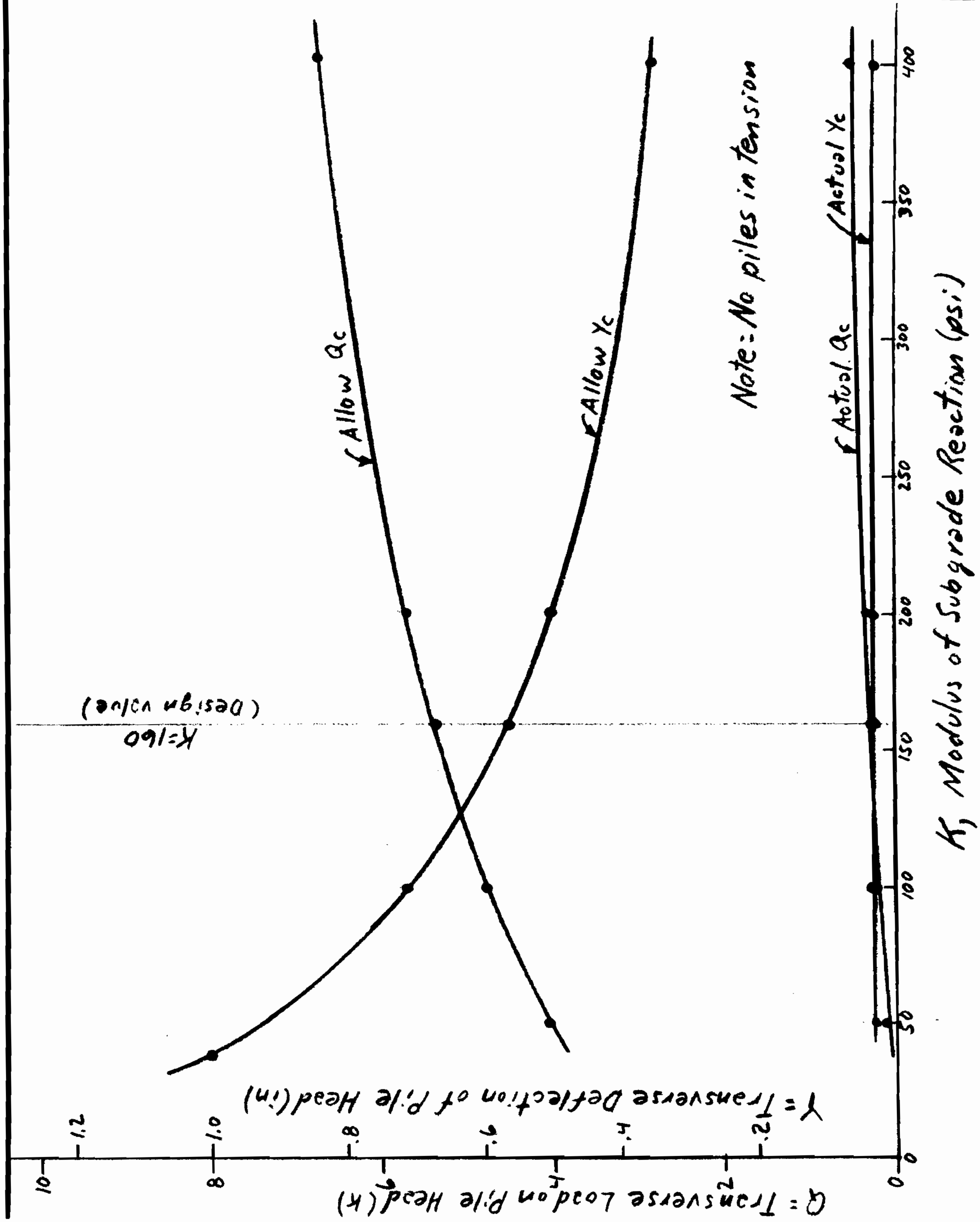
ENCLOSURE 12

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y VS. K - CONCRETE BAY STRIP - STATIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE NO. 453B2 DATE Mar. 10, 1971 PAGE OF PAGES



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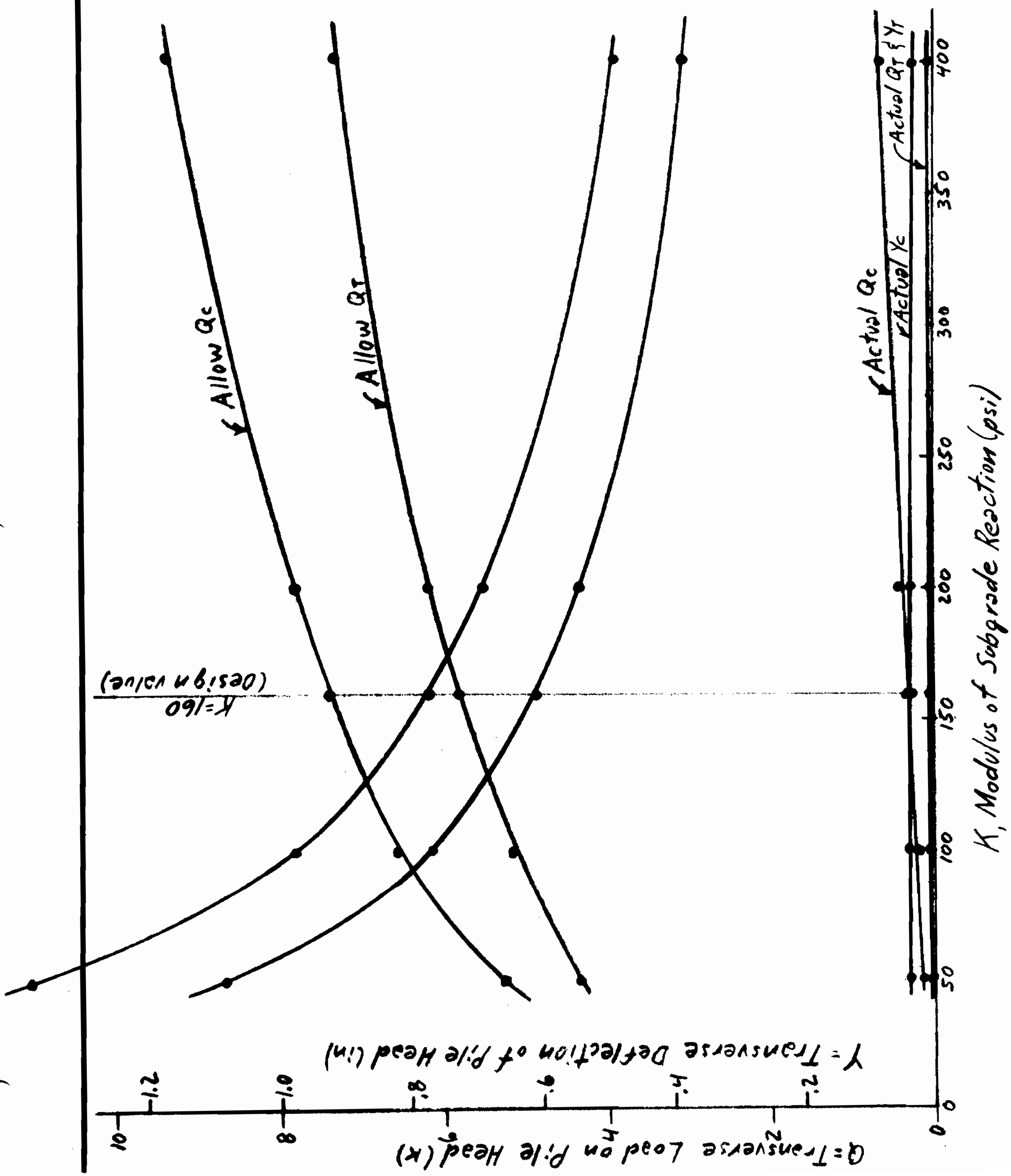
ENCLOSURE 13

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - CONCRETE BAY STRIP - DYNAMIC	PROJECT CHEF MENTEUR
		FILE NO. 453 B1
	COMPUTED EMC CHECKED	DATE MAR 11, 1971 PAGE ___ OF ___ PAGES



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ENCLOSURE 14

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT TIMBER PILE SUMMARY	PROJECT CHEF MENTEUR
	COMPUTED EMC CHECKED	FILE No. 453B1
		DATE March 17 1971 PAGE 1 OF 3 PAGES

SUMMARY OF CRITICAL TRANSVERSE PILE LOADS

Critical is defined as the greatest ratio of actual to allowable

Design strip	Loadings	Subgrade Modulus (psi)	Tension All. load (k)	Tension Act. load (k)	Compression All. load (k)	Compression Act. load (k)
		20	5.400	.091	4.301	.062
		40	6.423	.151	5.117	.103
PIER	STATIC	80	7.[illegible]	.252	7.642	.226
		120	[illegible]	.337	8.441	.305
		200	9.609	.486	9.558	.442
		20	6.673	.169	5.404	.140
		40	7.938	.282	6.430	.234
PIER	DYNAMIC	80	9.442	.468	7.654	.387
		120	10.452	.628	8.477	.517
		200	11.881	.903	9.645	.741
		20	No piles in tension		4.648	.071
		40	"	"	5.523	.119
BAY	STATIC	80	"	"	6.561	.198
		120	"	"	7.253	.267
		200	"	"	8.227	.389
		20	7.172	.018	6.264	.089
		40	8.529	.030	7.447	.148
BAY	DYNAMIC	80	10.143	.050	8.853	.248
		120	11.224	.068	9.794	.334
		200	12.753	.100	11.122	.485

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HARZA ENGINEERING COMPANY CHICAGO	SUBJECT TIMBER PILE SUMMARY	PROJECT CHET MU'ITEUR
	COMPUTED EMC CHECKED	FILE No. 45381
		DATE March 17, 1971 PAGE 2 OF 3 PAGES

SUMMARY OF CRITICAL TRANSVERSE PILE DEFLECTIONS

Critical is defined as the greatest ratio of actual to allowable

Design strip	Loadings	Subgrade Modulus (psi)	Tension All. defl (K)	Tension Act. defl. (K)	Compression All. defl. (K)	Compression Act. defl. (K)
		20	3.816	.066	3.040	.045
		40	2.699	.065	2.150	.045
PIER	STATIC	80	1.909	.065	1.909	.058
		120	1.559	.064	1.556	.058
		200	1.208	.063	1.201	.057
		20	4.716	.123	3.819	.102
		40	3.335	.122	2.702	.101
PIER	DYNAMIC	80	2.359	.120	1.912	.099
		120	1.927	.119	1.563	.098
		200	1.493	.117	1.212	.096
		20	No piles in tension		3.285	.051
		40	"	"	2.321	.051
BAY	STATIC	80	"	"	1.639	.051
		120	"	"	1.337	.051
		200	"	"	1.034	.050
		20	5.069	.013	4.426	.064
		40	3.584	.013	3.129	.064
BAY	STATIC	80	2.534	.013	2.212	.064
		120	2.069	.013	1.805	.063
		200	1.603	.013	1.398	.063

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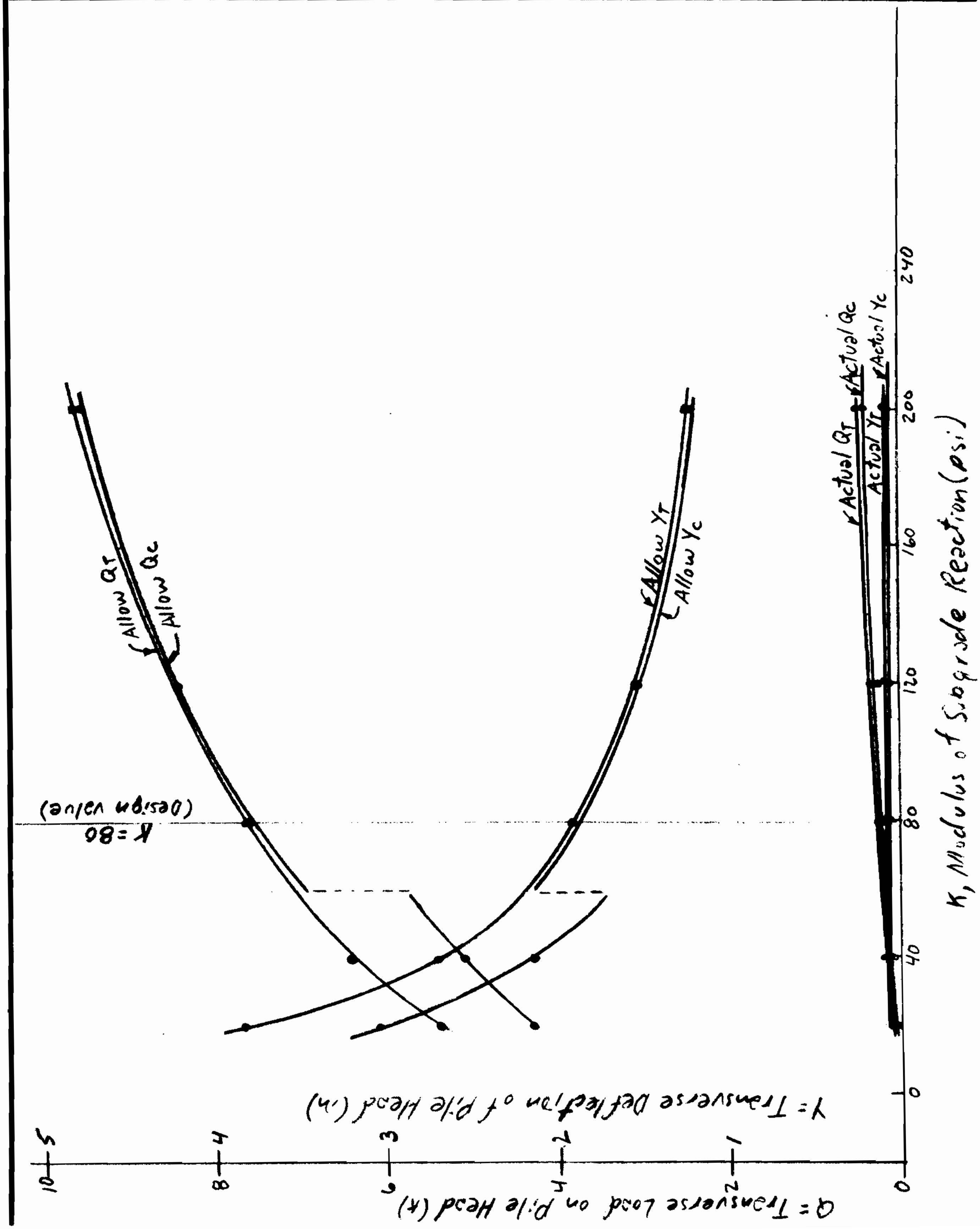
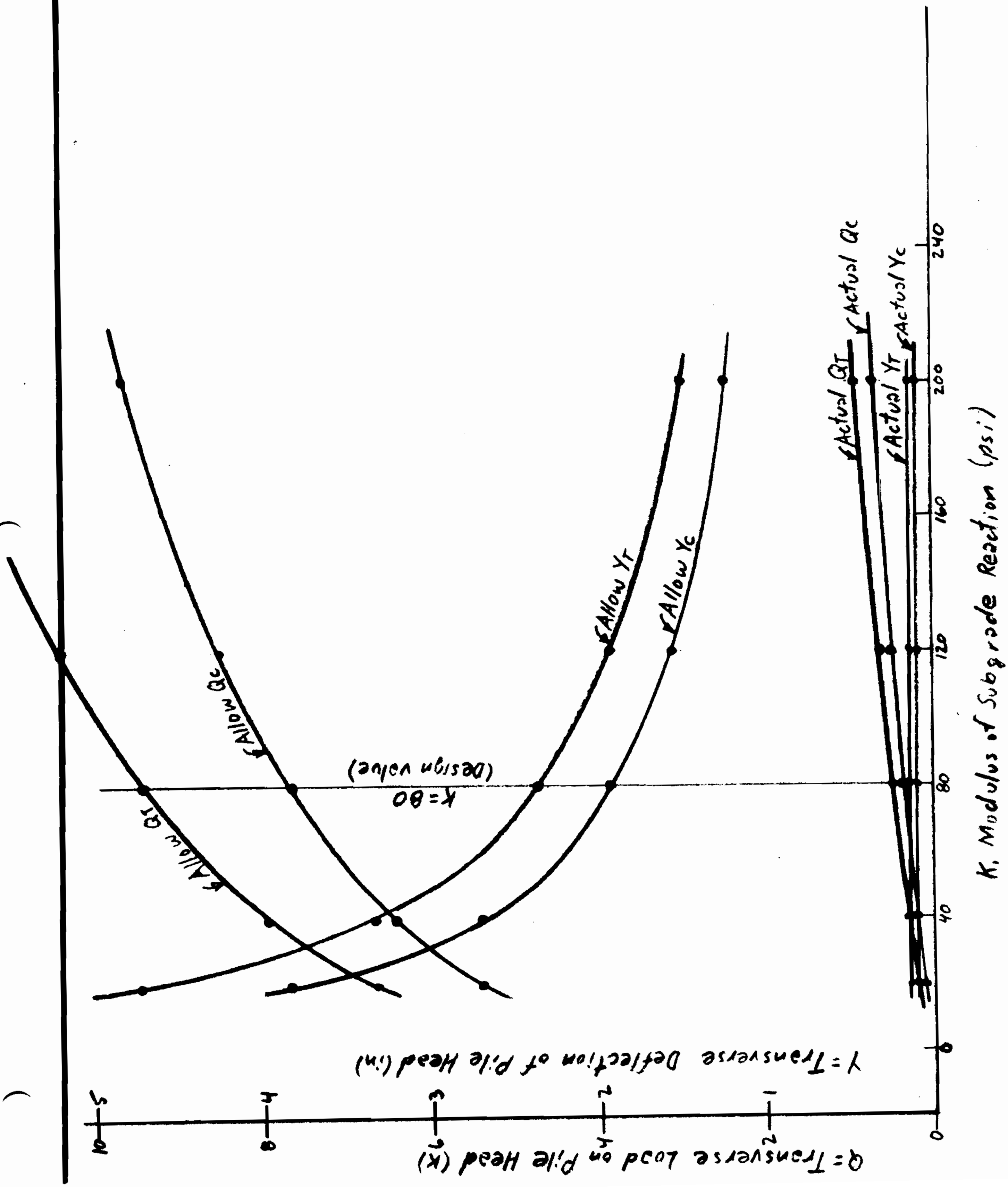
ENCLOSURE 14

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y VS. K - TIMBER PIER STRIP - STATIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE NO. 453B2 DATE Mar 17, 1971 PAGE 3 OF 3 PAGES



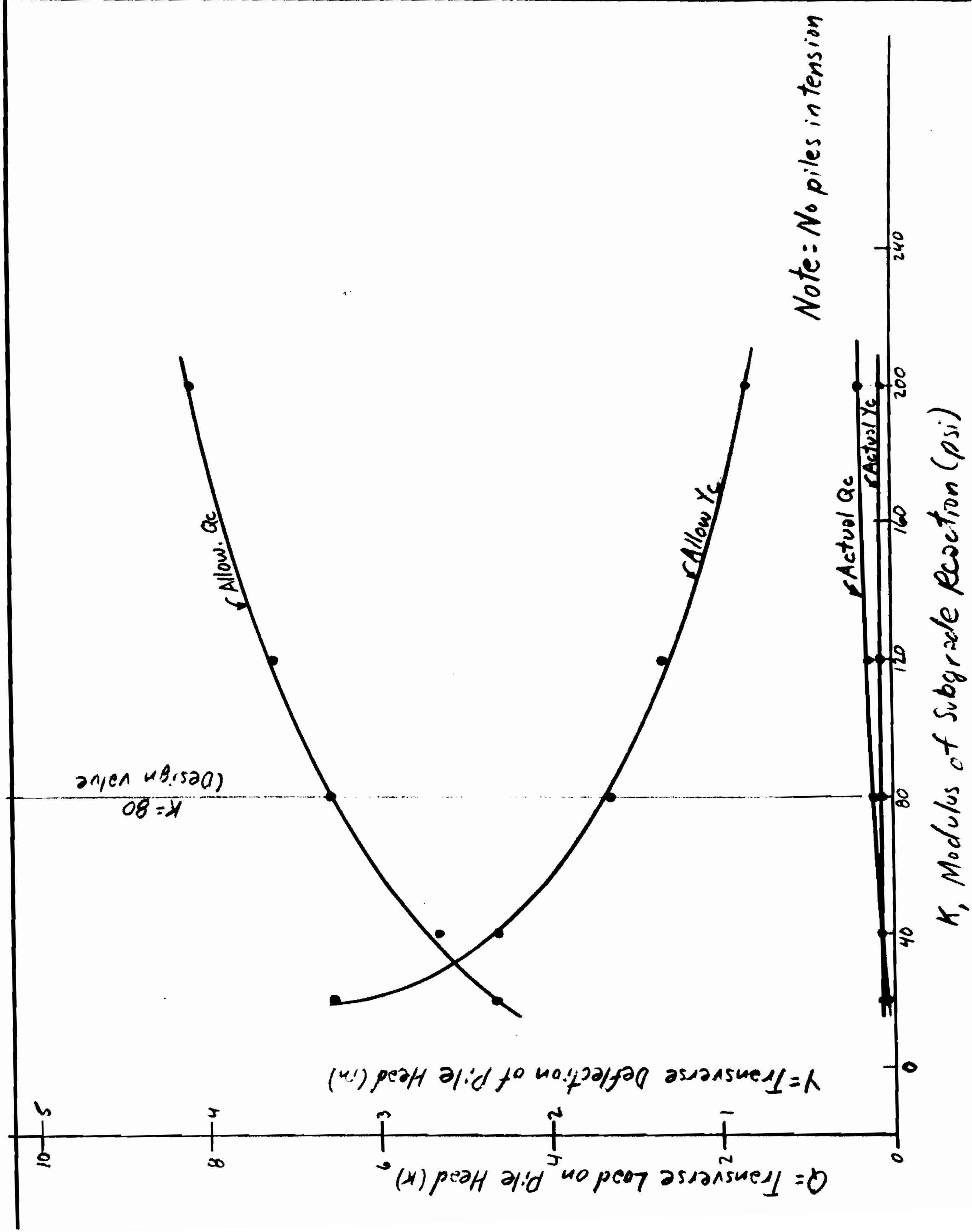
FOR USE ON U.S. GOVERNMENT WORK ONLY ENCLOSURE 15

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q&Y vs. K - TIMBER PIER STRIP - DYNAMIC	PROJECT CHEF MENTEUR
	COMPUTED EMC CHECKED	FILE NO. 453B1
		DATE March 18 1971 PAGE ___ OF ___ PAGES



FOR USE ON U.S. GOVERNMENT WORK ONLY ENCLOSURE 16

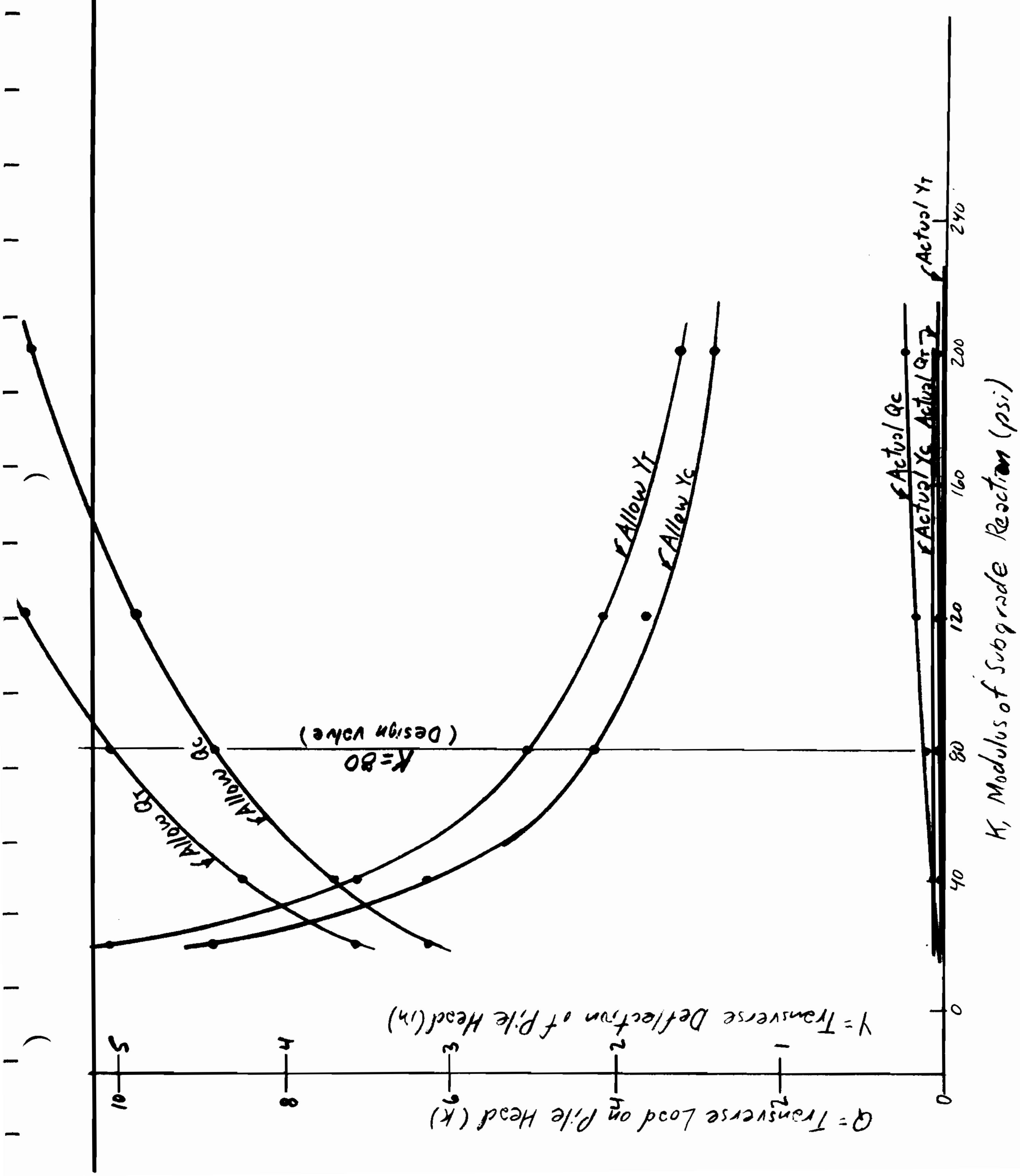
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y VS. K - TIMBER BAY STRIP - STATIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE No. 453 B2 DATE Mar 18, 1971 PAGE ___ OF ___ PAGES



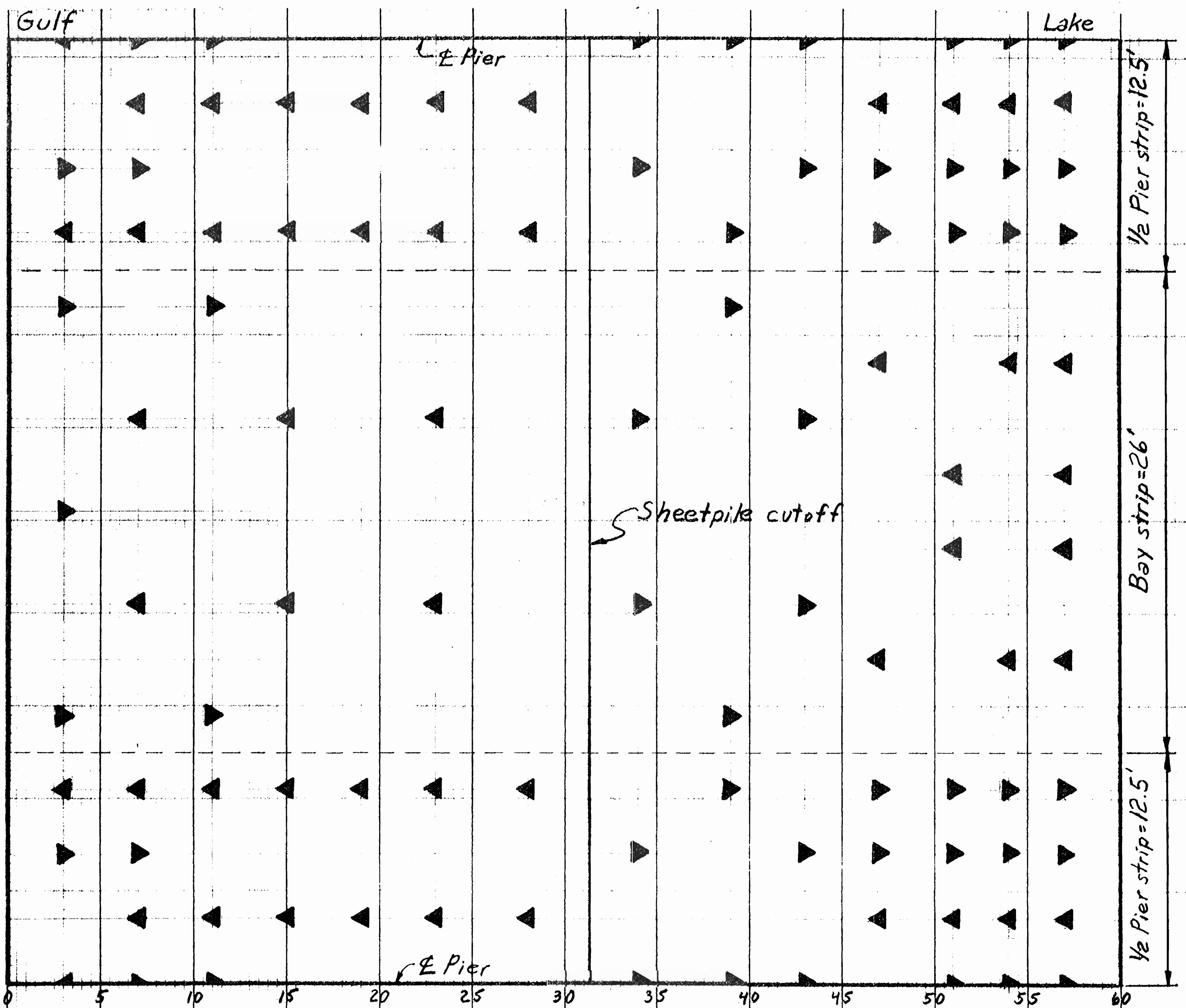
FOR USE ON U.S. GOVERNMENT WORK ONLY

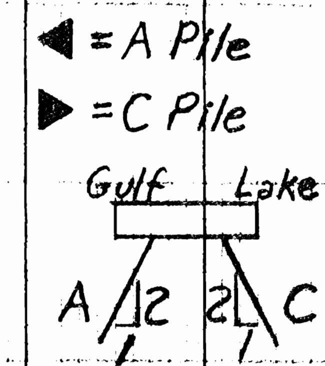
ENCLOSURE 17

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q&Y VS. K - TIMBER BAY STRIP - DYNAMIC COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE NO. 453B1 DATE Mar. 18, 197_ PAGE ___ OF ___ PAGES



ENCLOSURE 18



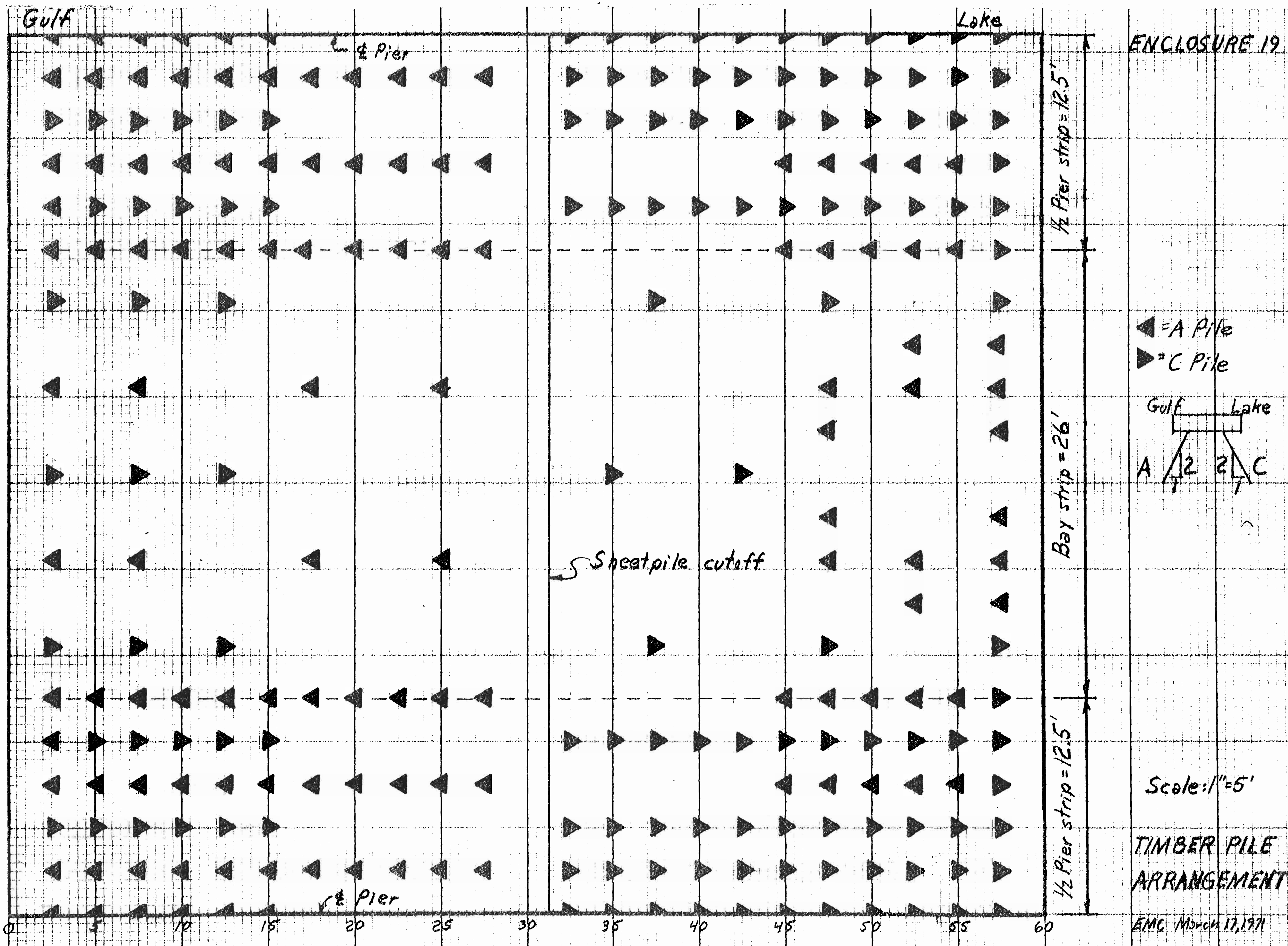


Scale: 1" = 5'

CONCRETE PILE ARRANGEMENT

EMC March 9, 1971

ENCLOSURE 19

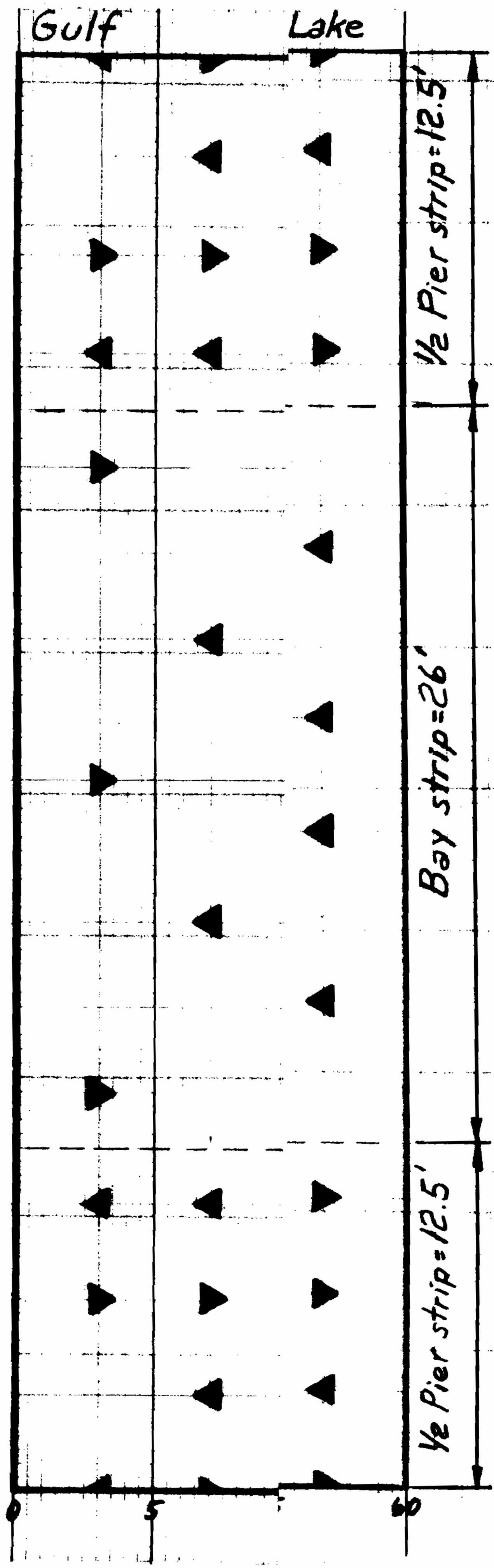


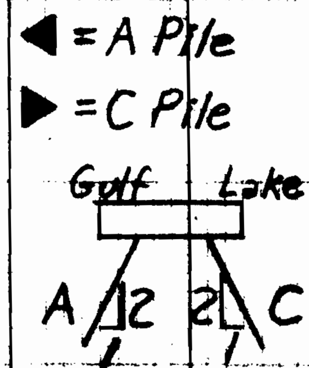
Scale: 1" = 5'

TIMBER PILE ARRANGEMENT

EMC March 17, 1971

ENCLOSURE 1B



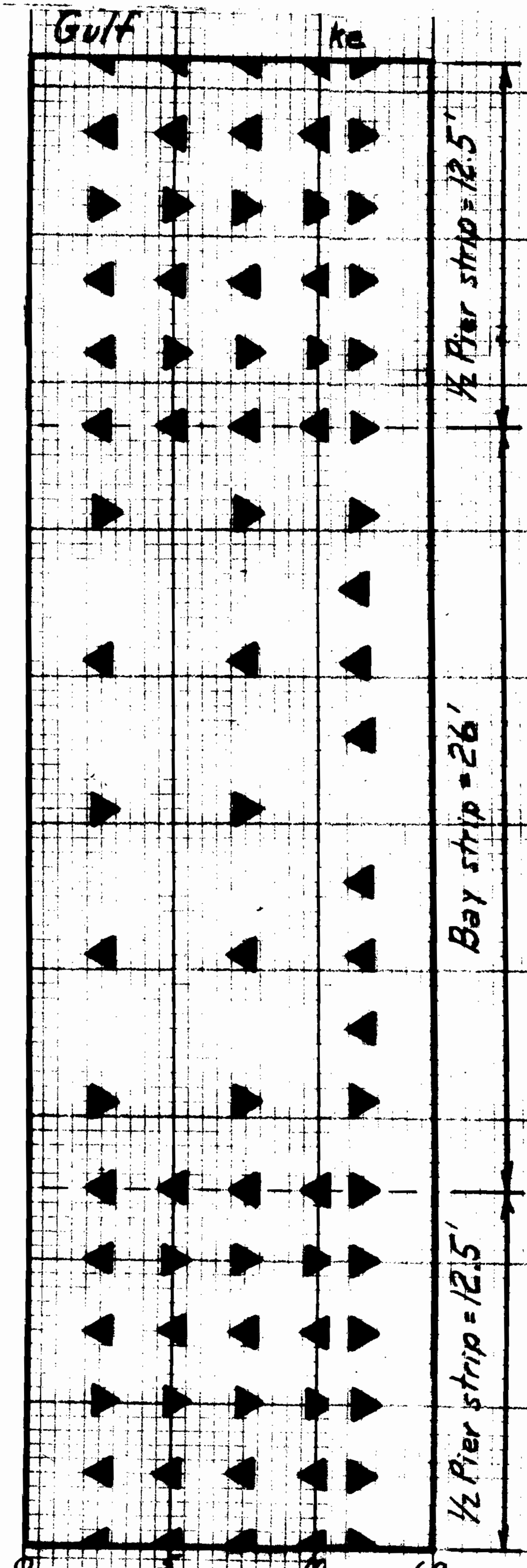


Scale: 1" = 5'

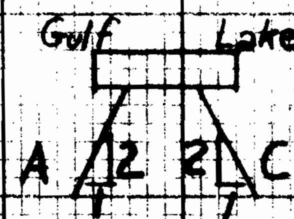
CONCRETE PILE ARRANGEMENT

EMC March 9, 1971

ENCLOSURE 19



◀ = A Pile
▶ = C Pile



Scale: 1" = 5'

TIMBER PILE
ARRANGEMENT

EMC March 17, 1971

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE ECONOMIC PILE SELECTION COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE No. 453B1 DATE Mar. 16 1971 PAGE 1 OF 3 PAGES

To select pile material compute cost of both timber and prestressed concrete piles and their installation for the entire control structure proper. All piles are driven on a 1h:2v batter.

TIMBER PILES

All piles are untreated dense southern pine, 50' long and 12" average diameter

Summary of Design

9 pier strips x 197 piles =	1773 piles x 50'	=	88,650 LF
8 bay strips x 39 piles =	312 piles x 50'	=	15,600 LF
TOTAL	2085 piles		104,250 LF

Costs

From the NOD, cost in place is \$2.25/LF

104,250 LF x \$2.25/LF = \$234,560

Assume all piles which are 2.5 ft. from the nearest pile will heave when adjacent piles are driven and will require redriving at a cost of \$.50/LF. This includes all piles in the pier strips

88,650 LF x \$.50/LF = 44,325

All tension piles require use of TECO, TENCON tension pile cap connectors. Piles in pier strip require 5 connectors each. Only a few rows in bay strip require tension connectors, but as a safety measure, say 2 connectors are applied to all bay strip piles.

Manufacturers cost data gives $4.60 installed cost per pair. Add 1/3 for contractors overhead and profit and something for manufacturers optimism, say $7/pair, or $3.50/connector.

Piers - 1773 piles x 5 connectors = 8865
Bays 312 piles x 2 connectors = 624
9489

Say 9500 connectors x $3.50/connector = $33,250

COST OF TIMBER FOUNDATION = $312,000

CONCRETE PILES

All piles are 12" square precast, prestressed concrete piles. Length = 63' beneath pier strips and 53 feet beneath bays.

Summary of Design

9 pier strips x 69 piles = 621 piles x 63' = 39,100 LF

8 bay strips x 27 piles = 216 piles x 53' = 11,450 LF

TOTAL 837 piles 50,550 LF

 ENCLOSURE 2

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE ECONOMIC PILE SELECTION	PROJECT CHEF MONTEUR
	COMPUTED EMC CHECKED	FILE NO. 453B1
		DATE Mar. 16, 1991 PAGE 3 OF 3 PAGES

Costs

From the NOD, cost in place is \$6.75/LF

50,550 LF x \$6.75/LF = \$341,000

COST OF CONCRETE FOUNDATION = \$341,000

No contingencies have been applied for comparative purposes because it would amount to the same percentage in each case

APPENDIX
(To letter of March 19, 1971 -
to NOD, District Engineer)

With regard to the design of the pile foundation, we are now using a modified version of the computer program submitted to us by the New Orleans District, U.S. Army Corps of Engineers (NOD) under cover of their letter of 18 September 1970. The purpose of this Appendix is to describe the modifications we have made to the basic program DATA 2/K29HRN.

The most basic change was to convert the program from a time-sharing to a batch-process mode, since the former is highly inconvenient for us. While making this change, we studied the program's operations in detail to determine its capabilities, especially with regard to utilizing the results of the computatinn to make a plot similar to Figure 4-12, Enclosure 7 to the NOD's letter of 19 June 1970. We understand that we will be required to prepare such a plot of transverse loads and deflections versus subgrade modulus as a part of our design of the control and navigation structure pile foundations.

Our review of the program revealed that it did not yield all information required for the plot; instead, much of the required information appeared to have been deferred to a hand computation. We saw value in causing the program to perform these calculations. Therefore, we changed the program to make it perform all the required computations. These changes, plus others that we made, are listed and described below:

1. Input data are now submitted on standard punched cards, rather than at a teletype terminal. We have made the program to read sequentially any number of data sets, so that a variety of pile layouts may be analyzed in one batch.

2. Allowable transverse and actual axial and transverse deflections are now automatically computed and recorded on the output printout.

3. The computation of "ACTUAL DEFL. IN." that appeared in the NOD's program appears to refer to the deflection of the top of a T-wall (not the pile tops); it has been deleted, as the present analysis is not for a T-wall.

4. The allowable axial and bending stress in tension for prestressed concrete piles, FAAT, was changed to 0.7 ksi in accordance with Enclosure 7 to the NOD's letter of 19 June 1970 and the references in that enclosure. Their program had used 0.35 ksi. Agreement with this change was confirmed in a telephone conversation between Tom Johnson of the NOD and Edward Cikanek of Harza.

5. We have added a column headed PILES REQD./ROW to the critical pile loadings summaries. This column shows the number of piles needed to make the computed axial pile load just less than the allowable axial pile load or to make the computed transverse load just less than the allowable transverse load. We believe the values in this column adequately replace the results of subroutine SUMRL1, which we have therefore deleted.

6. Through an iteration option, the program can utilize the results of one run as input for another to "zero in" on the necessary piles in each row without the need for additional intermediate manual inputs. If more than one iteration is specified, the program takes number of piles required in each row, rounds this value up to the nearest integral number, and recomputes all data with the new number of piles in each row. For transverse loads the program first checks whether the computed transverse load is less than 70 percent of the allowable transverse load. If it is not, the program adjusts the number of piles required so that it is less than 70 percent. This is done prior to the rounding up and recomputation for the

next iteration. This criterion for transverse loads was deduced from our previous contacts with the NOD, in which we understand they desire the computed transverse pile load to be significantly less than the allowable transverse pile load at the design value of the subgrade modulus. The number to the right of the second hyphen in the run number is the iteration number.

7. In conjunction with 6. above the printout options have been revised and expanded to call for printouts of either the last iteration, the first and last iterations, or all iterations.

8. The printout now automatically includes a summary of the maximum transverse loads and deflections for both tension and compression piles of all those computed. The allowable transverse loads and deflections corresponding to these maxima are also listed. The printout can also include a summary of the most critical transverse loads and deflections for both tension and compression piles of all those computed. Critical loads and deflections are defined as those having the greatest ratio of actual to allowable.

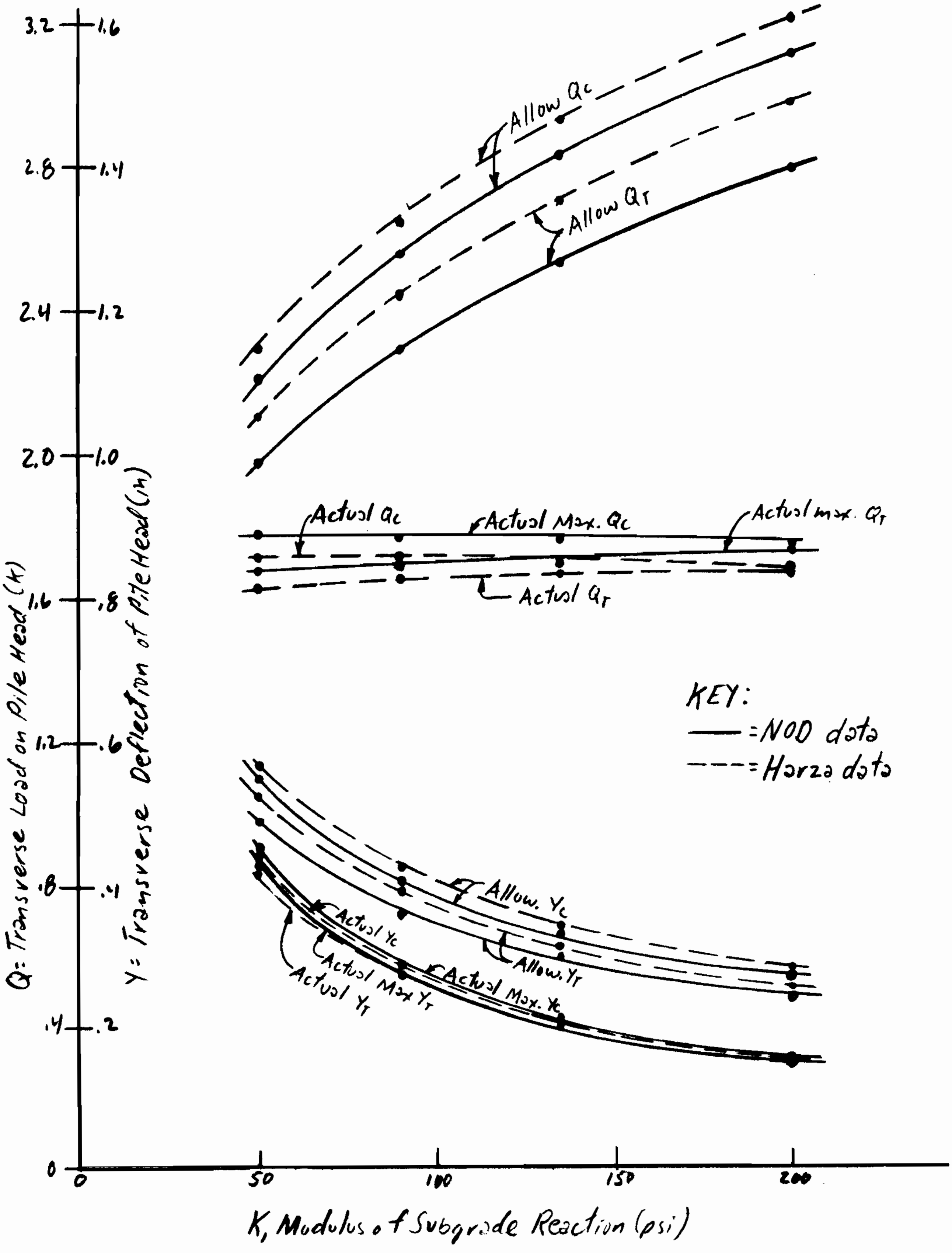
9. Allowable pile loads and stresses for transient loading cases may be increased by an appropriate input factor.

10. Any combination of pile group designations A, B, and C may now be used. The previous combinations were limited to ABC, AB, and A.

Samples of the output from our modified program are attached as Exhibit A-1. These results are for the same sample submitted as Enclosure 7 of the previously mentioned NOD letter of 19 June 1970, and they allow a plot similar to Figure 4-12 to be made. Prior to receiving the NOD's letter of 9 February 1971, the program had been modified by us to compute allowable transverse loads and deflections from <u>allowable axial</u> loads. The plot made from this program exactly duplicated Figure 4-12 and this was considered verification that the modified program was functioning correctly. Upon receipt of the aforementioned letter of 9 February 1971, the program was again modified to meet the letter requirement of the calculation of allowable transverse loads and deflections based on the computed axial loads. The plot made from the program thus modified does not now duplicate Figure 4-12. As was discussed in our letter to the NOD dated February 25, 1971, Figure 4-12 is a plot of maximum transverse loads and deflections and of allowable transverse loads and deflections versus subgrade modulus in which the allowable transverse values were based on the allowable axial loads rather than the computed axial loads. Also, the maximum values of transverse load and deflection plotted in Figure 4-12 were not the critical values. A plot of data shown on Figure 4-12 as sent to Harza, has been modified to show the critical results as obtained from our latest program modification and is also enclosed as Exhibit A-2.

FOR USE ON U.S. GOVERNMENT WORK ONLY APPENDIX EXHIBIT A-2

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT FIGURE 4-12 WITH CRITICAL RESULTS COMPUTED EMC CHECKED	PROJECT CHEF MENTEUR FILE NO. 453B1 DATE Mar. 18, 1971 PAGE __ OF __ PAGES



APPENDIX B

Actual and Allowable Transverse Loads and Deflections for Abutment and Barrier Piers

Enclosure 1	Abutment Pier - Static Loads
2	Abutment Pier - Dynamic Loads
3	Barrier Piers 1 & 2 - Static Loads
4	Barrier Piers 1 & 2 - Dynamic Loads
5	Barrier Piers 3 & 4 - Static Loads
6	Barrier Piers 3 & 4 - Dynamic Loads
7	Barrier Pier 5 - Static Loads
8	Barrier Pier 5 - Dynamic Loads
9	Barrier Pier 6 - Static Loads
10	Barrier Pier 6 - Dynamic Loads

ENCLOSURE I

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT [illegible] ABUTMENT PIER COMPUTED KWR CHECKED	PROJECT [illegible] FILE NO. [illegible] DATE 10/2/72 PAGE 1 OF 7 PAGES

SUMMARY OF CRITICAL DATA FROM OUTPUT

LOADING	SUBGRADE MODULUS psi	AXIAL LOAD TENSION K	AXIAL LOAD COMP K	TRANS. LOAD TENSION / COMP K	TRANS. DEFL TENSION / COMP in.
STATIC	20	-81.91	130.64	.619	.258
	40	-78.16	127.19	1.004	.250
	65	-74.83	123.74	1.393	.239
	125	-68.29	117.38	2.117	.223
	200	-62.29	111.48	2.806	.208
DYNAMIC	20	-89.39	143.10	.712	.297
	40	-85.76	139.38	1.244	.287
	65	-82.10	135.67	1.785	.278
	125	-75.35	128.82	2.634	.260
	200	-69.02	122.43	3.262	.243

ENCLOSURE I

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT: STEEL PILE DESIGN ABUTMENT PIER COMPUTED: RWR CHECKED:	PROJECT: Chef Menteur FILE No. 453121 DATE 10/27/72 PAGE 2 OF 7 PAGES

Since the longitudinal loadings and the lateral loadings had to be run separately and then the results added, the allowable transverse loads and deflections could not have been calculated by the computer. They will be calculated manually using the following equations from the Hrennikoff program.

Allow. Transverse Load in Compression

$$QC = \left(1 - \frac{P/AREA}{FAAC}\right) \times 2 \times FBAC \times \frac{SM}{R}$$

Allow. Transverse Load in Tension

$$QT = \left(1 - \frac{ABS(P)/AREA}{FAAT}\right) \times 2 \times FAAT \times \frac{SM}{R}$$

Allow. Transverse Deflection in Compression

$$YC \quad 1375 \times QC \times \frac{R^3}{E \times AI}$$

Allow. Transverse Deflection in Tension

$$YT \quad 1375 \times QT \times \frac{R^3}{E \times AI}$$

ENCLOSURE 1

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT: STEEL PILE DESIGN ABUTMENT PIER	PROJECT: Chef Menteur
	COMPUTED: RWR CHECKED:	FILE No. 453 B1
		DATE 10/27/72 PAGE 3 OF 7 PAGES

Constants

FAAC = 12 KSI
FBAC = 18 KSI
FAAT = 18 KSI
AREA = 21.8 in^2
SM = 93.4 in^3
E = 29000000 psi
AI = 566 in^4
AK = SUBGRADE MODULUS, varies

$$R = \sqrt[4]{\frac{E * AI}{AK}}$$

AK	R	QC	YC	QT	YT
20	169.2	19.87 - 0.07596P	0.4056 Q_c	19.87 - 0.05064P	0.4056 Q_T
40	143.2	23.46 - 0.08976P	0.2459 QC	23.46 - 0.05984P	0.2459 Q_T
65	126.1	26.66 - 0.1019P	0.1679 Q_c	26.66 - 0.06794P	0.1679 Q_T
125	107.0	31.42 - 0.1201P	0.1026 Q_c	31.42 - 0.08007P	0.1026 Q_T
200	95.9	35.06 - 0.1340P	0.0739 Q_c	35.06 - 0.08935P	0.0739 Q_T

ENCLOSURE I

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE DESIGN ABUTMENT PIER	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453B1
		DATE 10/27/72 PAGE 4 OF 7 PAGES

Compression Piles

LOADING	SUBGRADE MODULUS PSI	AXIAL LOAD KIPS	NEGATIVE SKIN FRICTION KIPS	P TOTAL AXIAL LOAD KIPS	fP	QC KIPS	YC in.
STATIC	20	130.64	92.87	223.51	16.98	2.89	1.172
	40	127.19		220.06	19.75	3.73	.917
	65	123.74		216.61	22.07	4.59	.771
	125	117.38		210.25	25.25	6.17	.633
	200	111.48		204.35	27.38	7.68	.568
DYNAMIC	20	143.10		235.97	17.92	1.95	0.791
	40	139.38		232.25	20.85	2.63	.647
	65	135.67		228.54	23.29	3.37	.566
	125	128.82		221.69	26.62	4.80	.492
	200	122.43		215.30	28.85	6.21	.459

Tension Piles

LOADING	SUBGRADE MODULUS PSI	P Axial Load KIPS	fABS(P)	QT	YT
STATIC	20	-81.91	4.15	15.72	6.376
	40	-78.16	4.68	18.80	4.623
	65	-74.83	5.08	21.58	3.622
	125	-68.29	5.47	25.95	2.662
	200	-62.29	5.57	29.49	2.179
DYNAMIC	20	-89.39	4.53	15.35	6.226
	40	-85.76	5.13	18.35	4.512
	65	-82.10	5.57	21.09	3.541
	125	-75.35	6.03	25.39	2.605
	200	69.02	6.17	28.89	.135

ENCLOSURE I

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE SUMMARY	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 1532 C 1
		DATE 10/29/72 PAGE 5 OF 7 PAGES

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. defl. (in)	Tension Act. defl (in)	Compression All. defl. (in)	Compression Act. defl. (in)
Abutment Pier	Static	20	6.376	.258	1.172	.258
		40	4.623	.250	.917	.250
		65	3.623	.239	.771	.239
		125	2.662	.223	.633	.223
		200	2.179	.208	.568	.208
	Dynamic	20	6.226	.297	.791	.297
		40	4.512	287	.647	.287
		65	3.541	.278	.566	.278
		125	2.605	.260	.492	.260
		200	2.135	.243	.459	.243

ENCLOSURE I

FOR USE ON U.S. GOVERNMENT WORK ONLY

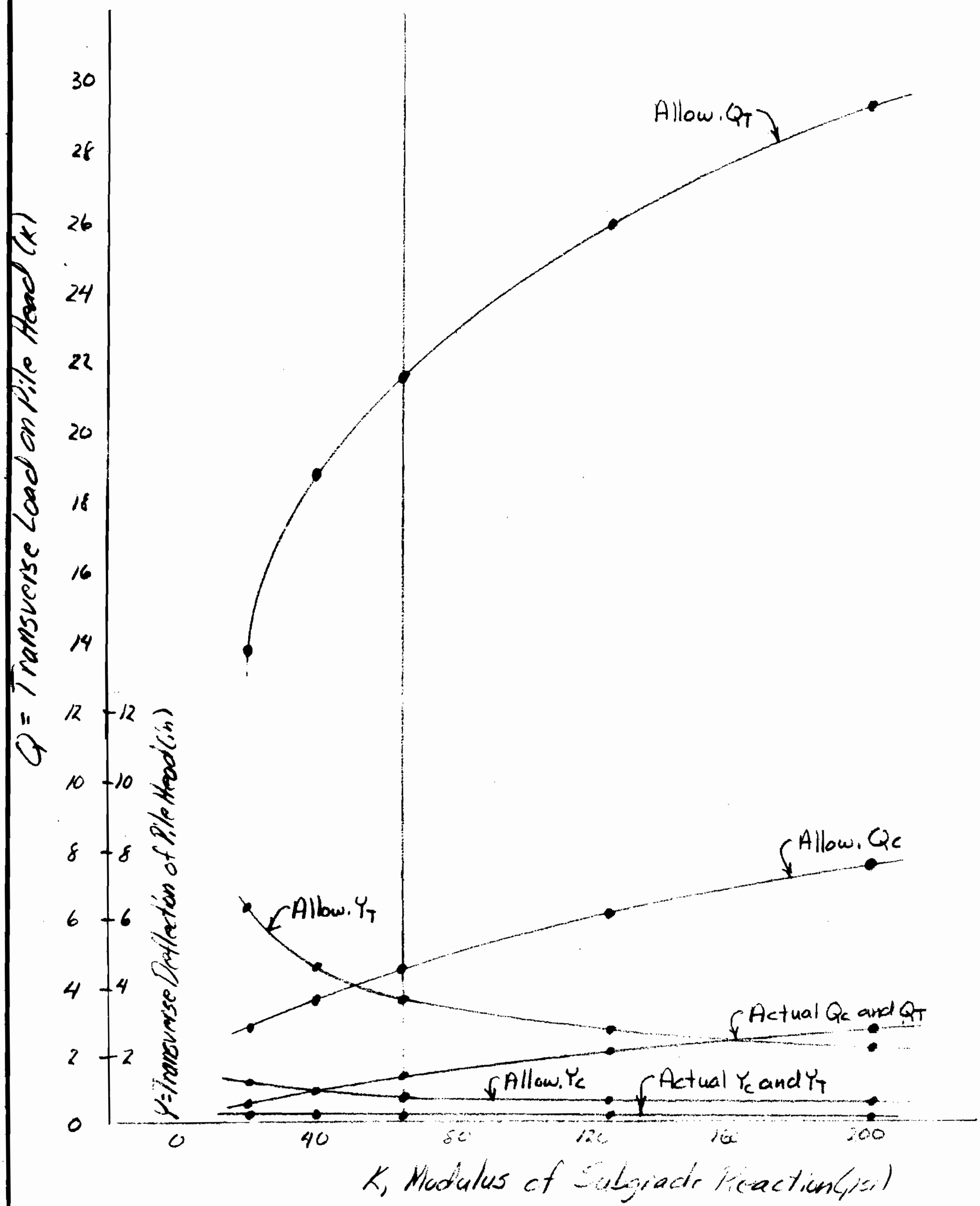
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE SUMMARY	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453B1
		DATE 10/29/72 PAGE 6 OF 7 PAGES

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. Load (K)	Tension Act. Load (K)	Compression All. Load (K)	Compression Act. Load (K)
Abutment Pier	Static	20	15.72	.619	2.89	.619
		40	18.80	1.004	3.73	1.004
		65	21.58	1.393	4.59	1.393
		125	25.95	2.117	6.17	2.117
		200	29.49	2.806	7.68	2.806
	Dynamic	20	15.35	.712	1.95	.712
		40	18.35	1.244	2.63	1.244
		65	21.09	1.785	3.37	1.785
		125	25.39	2.634	4.80	2.634
		200	28.89	3.262	6.21	3.262

ENCLOSURE 1

FOR USE ON U.S. GOVERNMENT WORK ONLY

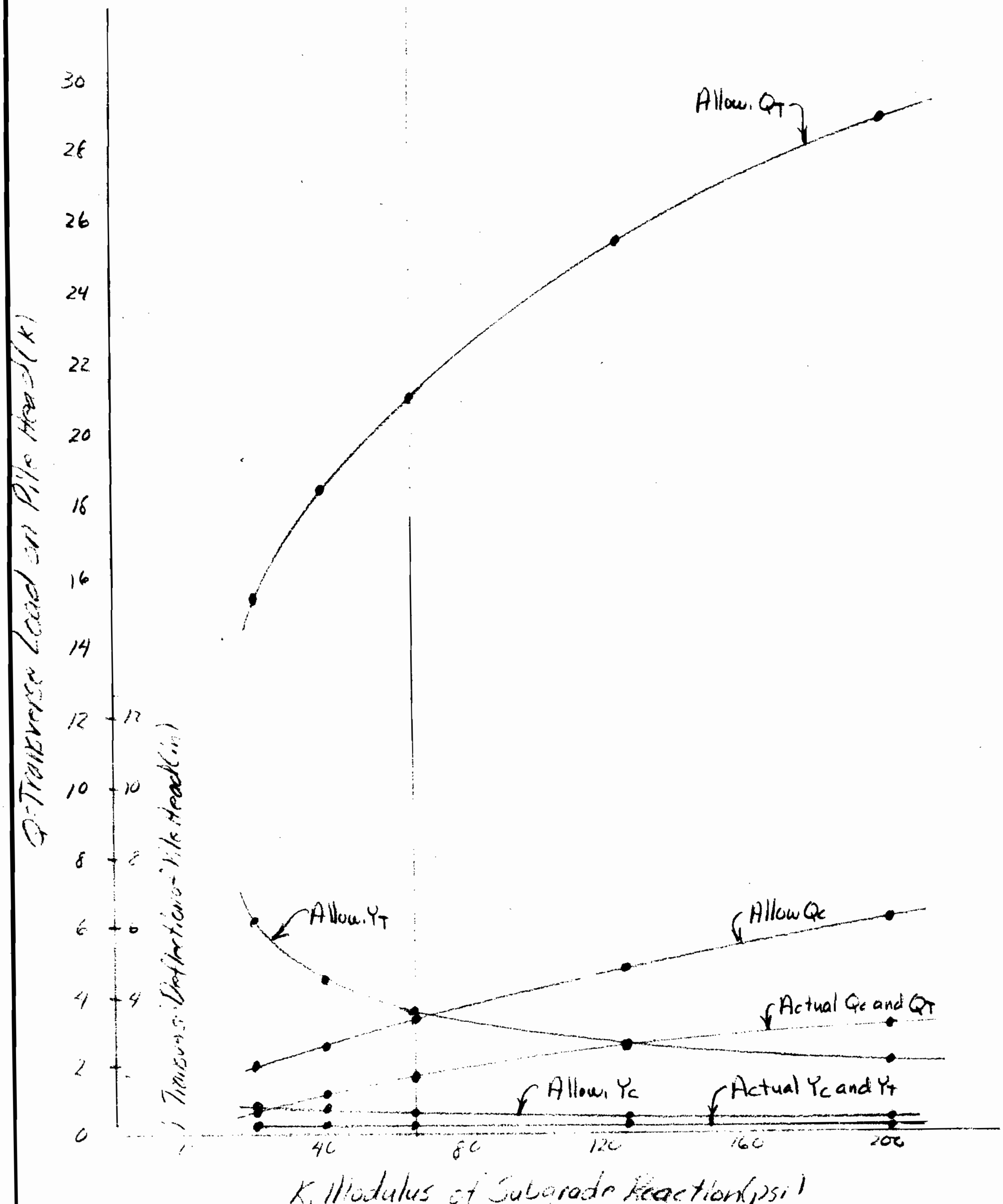
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL ABUTMENT PIER STATIC COMPUTED RWR CHECKED	PROJECT Chef Menteur FILE NO. 452P1 DATE 4/29/72 PAGE 7 OF 7 PAGES



ENCLOSURE 2

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL ABUTMENT PIER DYNAMIC COMPUTED PWF CHECKED	PROJECT Chef Menteur FILE NO. 453B1 DATE 10/29/72 PAGE 1 OF 1 PAGES



ENCLOSURE 3

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE SUMMARY	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE No. 453B1
		DATE 10/19/72 PAGE 1 OF 5 PAGES

SUMMARY OF CRITICAL TRANSVERSE PILE DEFLECTIONS

Critical is defined as the greatest ratio of actual to allowable

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. defl. (in)	Tension Act. defl. (in)	Compression All. defl. (in)	Compression Act. defl. (in)
Barrier Piers 1 & 2	Static	20	No Piles In Tension		1.358	.061
		40			.955	.060
		65			.746	.060
		125			.532	.060
		200			.416	.059
	Dynamic	20	9.199	.108	2.115	.067
		40	6.506	.108	1.509	.066
		65	5.109	.107	1.189	.065
		125	3.693	.105	.863	.063
		200	2.919	.103	.686	.061
Barrier Piers 3 & 4	Static	20	No Piles In Tension		.293	.022
		40			.210	.022
		80			.151	.021
		125			.123	.020
		200			.100	.019

ENCLOSURE 3

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE SUMMARY	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE No. 453(B)
		DATE 10/9/72 PAGE 2 OF 5 PAGES

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. defl. (in)	Tension Act. defl. (in)	Compression All. defl. (in)	Compression Act. defl. (in)
Carriers Piers 3 & 4	Dynamic	20	9,230	.105	2.091	.061
		40	6.519	.103	1.482	.060
		80	4.614	.102	1.053	.058
		125	3.686	.100	.846	.057
		200	2.917	.099	.673	.055
Carriers Pier 5	Static	20	No Piles In Tension		.555	.024
		40			.395	.024
		80			.282	.023
		125			.228	.023
		200			.182	.022
	Dynamic	20	9.224	.112	2.206	.068
		40	6.525	.111	1.564	.067
		80	4.615	.110	1.111	.065
		125	3.691	.108	.893	.064
		200	2.908	.105	.710	.062
Barrier Pier 6	Static	20	No Piles In Tension		.363	.051
		40			.268	.051
		90			.193	.050
		125			.172	.049
		200			.147	.048
		20	9.023	.147	1.862	.096
		40	6.391	.146	1.279	.095
		90	4.276	.145	.859	.093
		125	3.636	.144	.732	.092
		200	2.885	.142	.584	.089

ENCLOSURE 3

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	Subject: Steel Pile Summary	Project: Chef Menteur
	Computed: RWR Checked:	File No. 453B1
		Date: 10/9/72 Page 3 of 5 Pages

SUMMARY OF CRITICAL TRANVERSE PILE LOADS

Critical is defined as the greatest ratio of actual to allowable

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. load (K)	Tension Act. load (K)	Compression All load (K)	Compression Act load (K)
Barrier Piers 1 & 2	Static	20	No Piles In Tension		3,053	.133
		40			3.613	.222
		65			4.059	.318
		125			4.732	.514
		200			5.265	.724
	Dynamic	20	20.687	.237	4,755	.145
		40	24.606	.395	5.707	.242
		65	27.810	.565	6.473	.344
		125	32.829	.909	7.668	.546
		200	36.916	1.265	8.678	.752
Barrier Piers 3 & 4	Static	20	No Piles in Tension		.659	.048
		40			.793	.080
		80			.962	.129
		125			1.095	.175
		200			1.263	.236

ENCLOSURE 3

FOR USE ON U.S. GOVERNMENT WORK ONLY

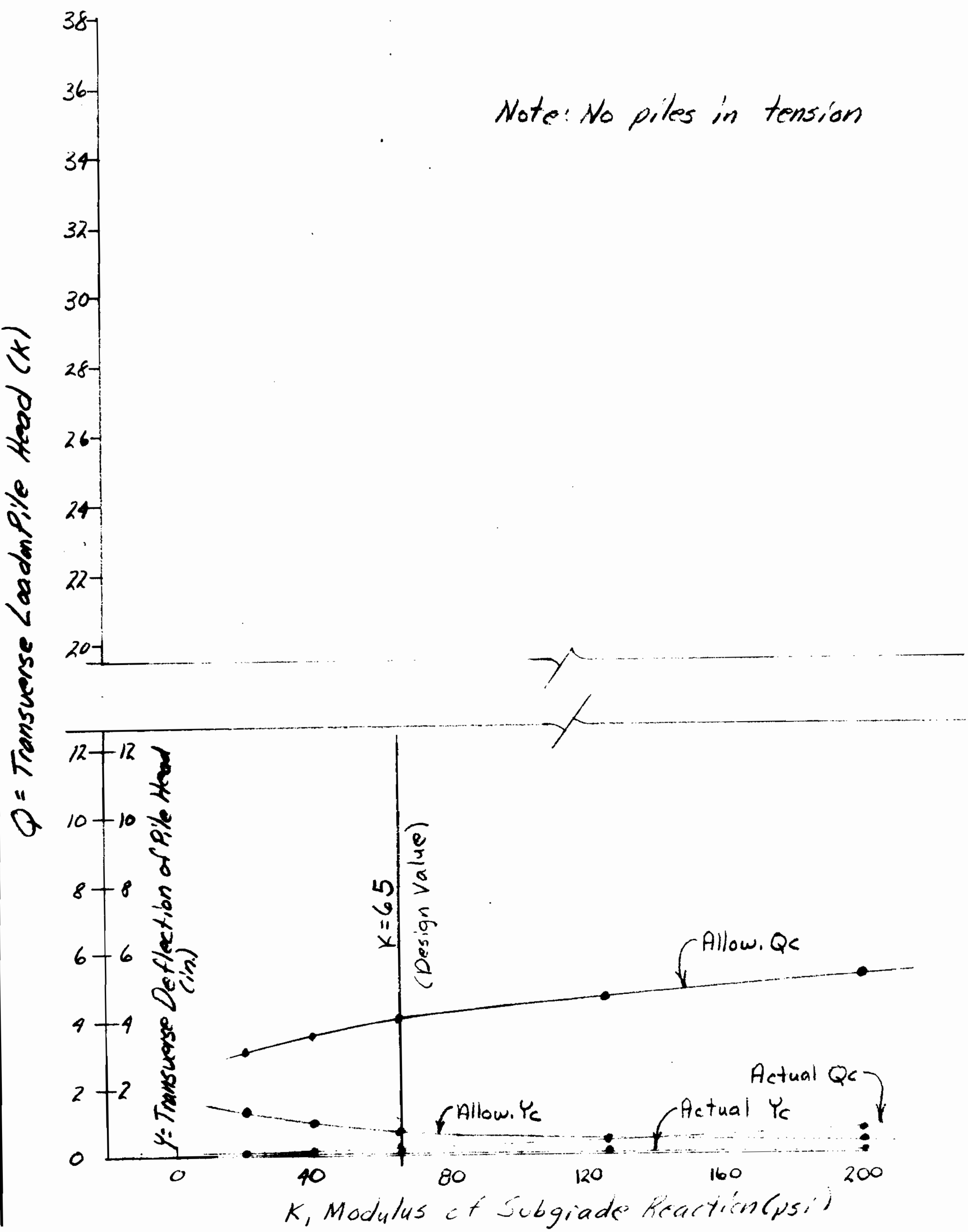
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT STEEL PILE SUMMARY	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE No. 453B1
		DATE 10/9/72 PAGE 4 OF 5 PAGES

Design Strip	Loading	Subgrade Modulus (psi)	Tension All. Load (K)	Tension Act. Load (K)	Compression All. Load (K)	Compression Act. Load (K)
Barrier Piers 3 & 4	Dynamic	20	20.757	.229	4.702	.132
		40	24.656	.380	5.606	.220
		80	29.345	.631	6.697	.361
		125	32.767	.867	7.520	.494
		200	36.886	1.212	8.509	.680
Barrier Pier 5	Static	20	No Piles In Tension		1.247	.053
		40			1.492	.088
		80			1.793	.144
		125			2.024	.195
		200			2.307	.265
	Dynamic	20	20.742	.245	4.960	.148
		40	24.677	.409	5.915	.246
		80	29.350	.678	7.066	.404
		125	32.806	.934	7.934	.553
		200	36.779	1.292	8.978	.763
Barrier Pier 6	Static	20	No Piles In Tension		.816	.112
		40			1.012	.187
		90			1.343	.337
		125			1.525	.426
		200			1.856	.591
	Dynamic	20	20.290	.322	4.052	.210
		40	24.171	.528	4.837	.350
		90	29.707	.977	5.968	.628
		125	32.517	1.242	6.508	.792
		200	36.488	1.744	7.381	1.097

ENCLOSURE 3

FOR USE ON U.S. GOVERNMENT WORK ONLY

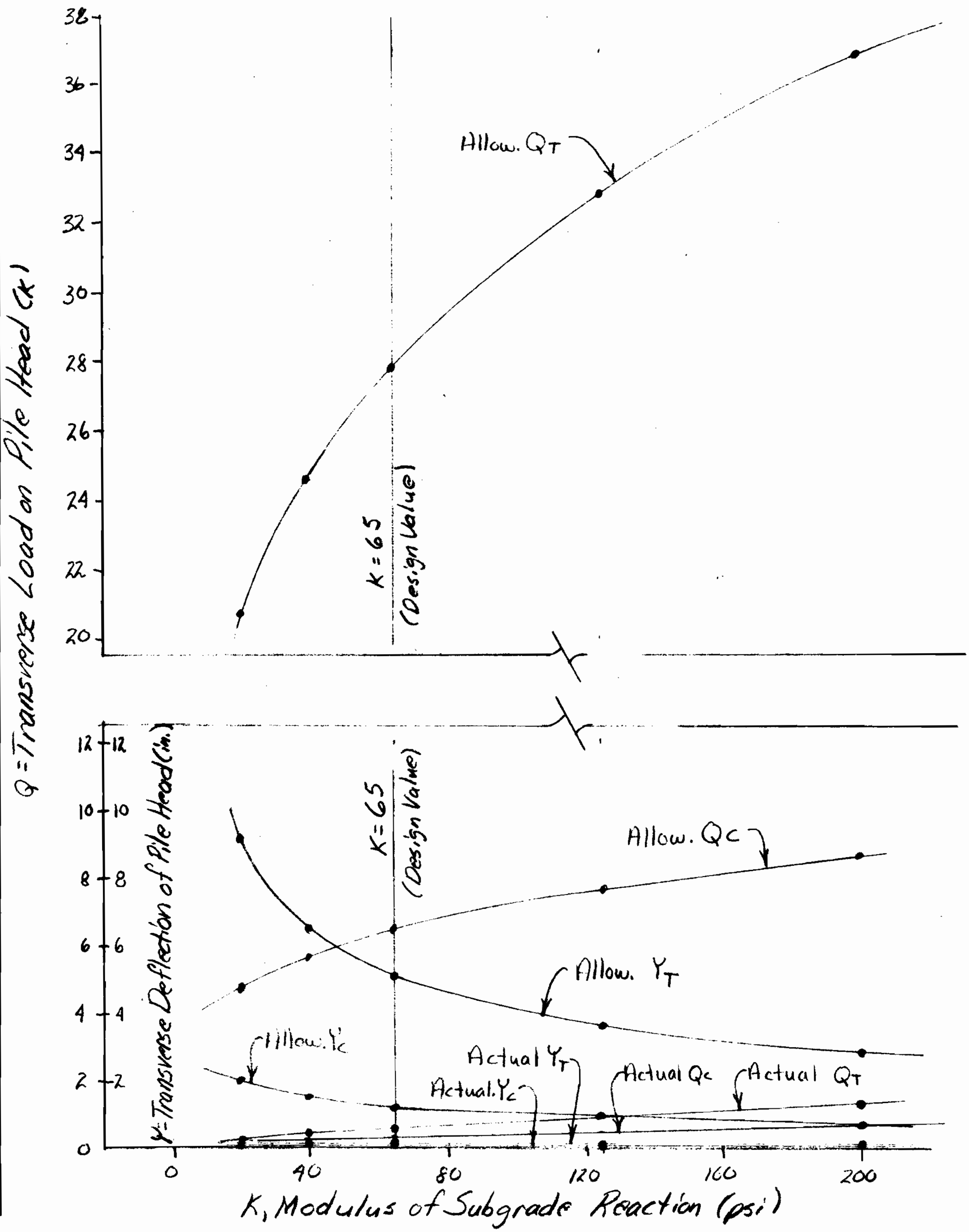
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIERS 1 & 2 STATIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453B1
		DATE 10/10/72 PAGE 5 OF 5 PAGES



ENCLOSURE 4

FOR USE ON U.S. GOVERNMENT WORK ONLY

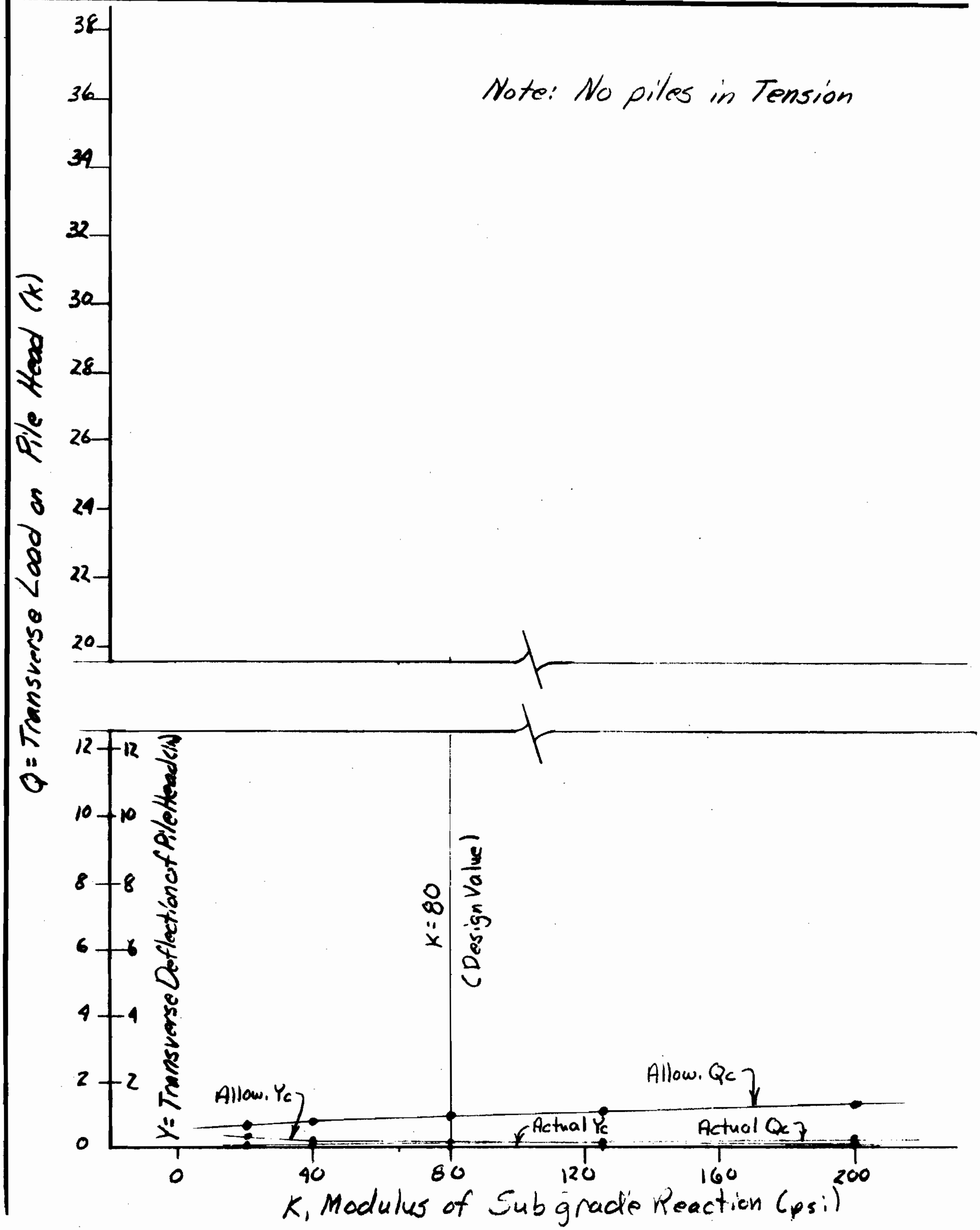
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs K - STEEL BARRIER PIERS 1 & 2 DYNAMIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE No. 452 B1
		DATE 10/10/72 PAGE 1 OF 1 PAGES



ENCLOSURE 5

FOR USE ON U.S. GOVERNMENT WORK ONLY

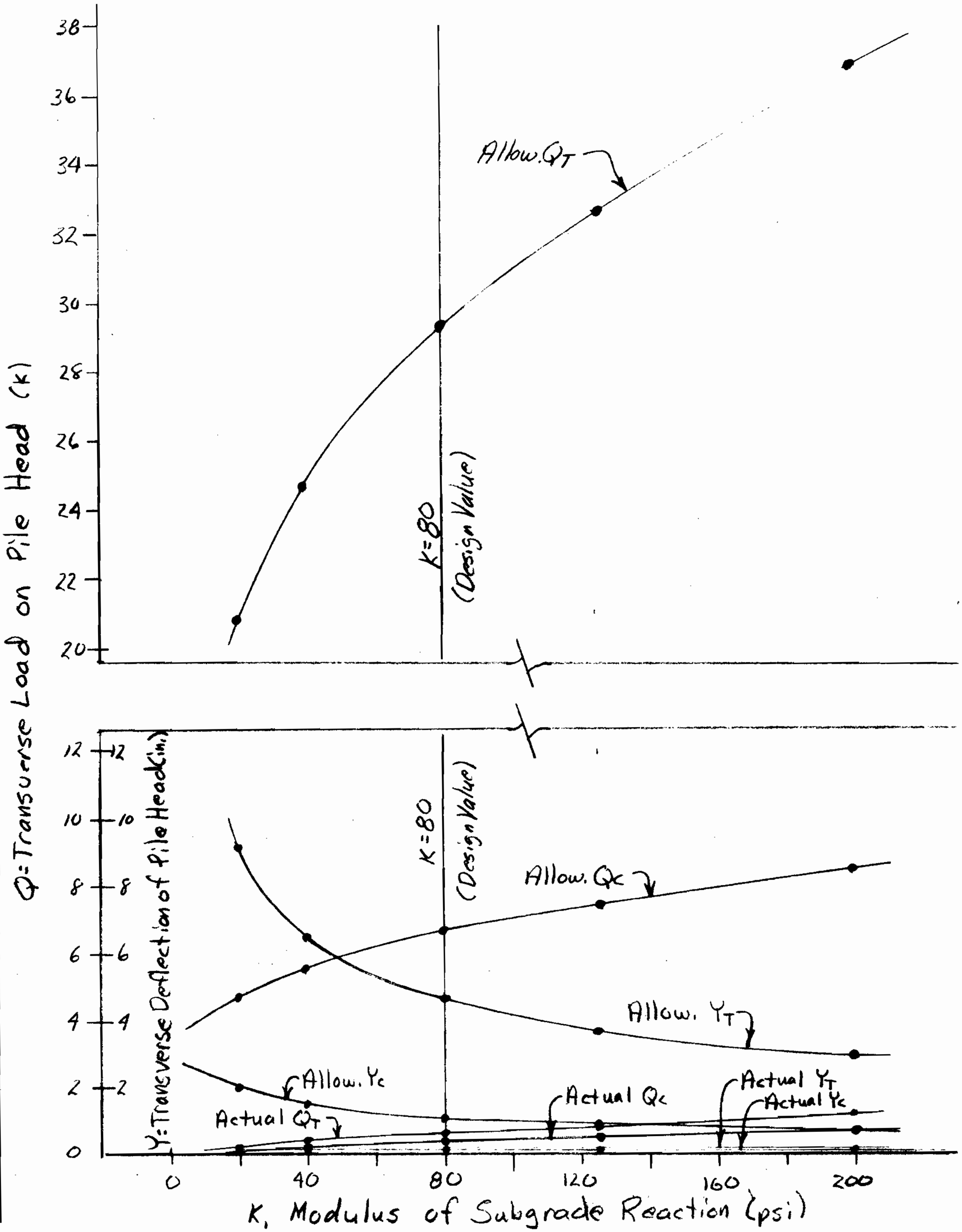
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIERS 3 & 4 STATIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE No. 453B1
		DATE 10/10/72 PAGE ___ OF ___ PAGES



ENCLOSURE 6

FOR USE ON U.S. GOVERNMENT WORK ONLY

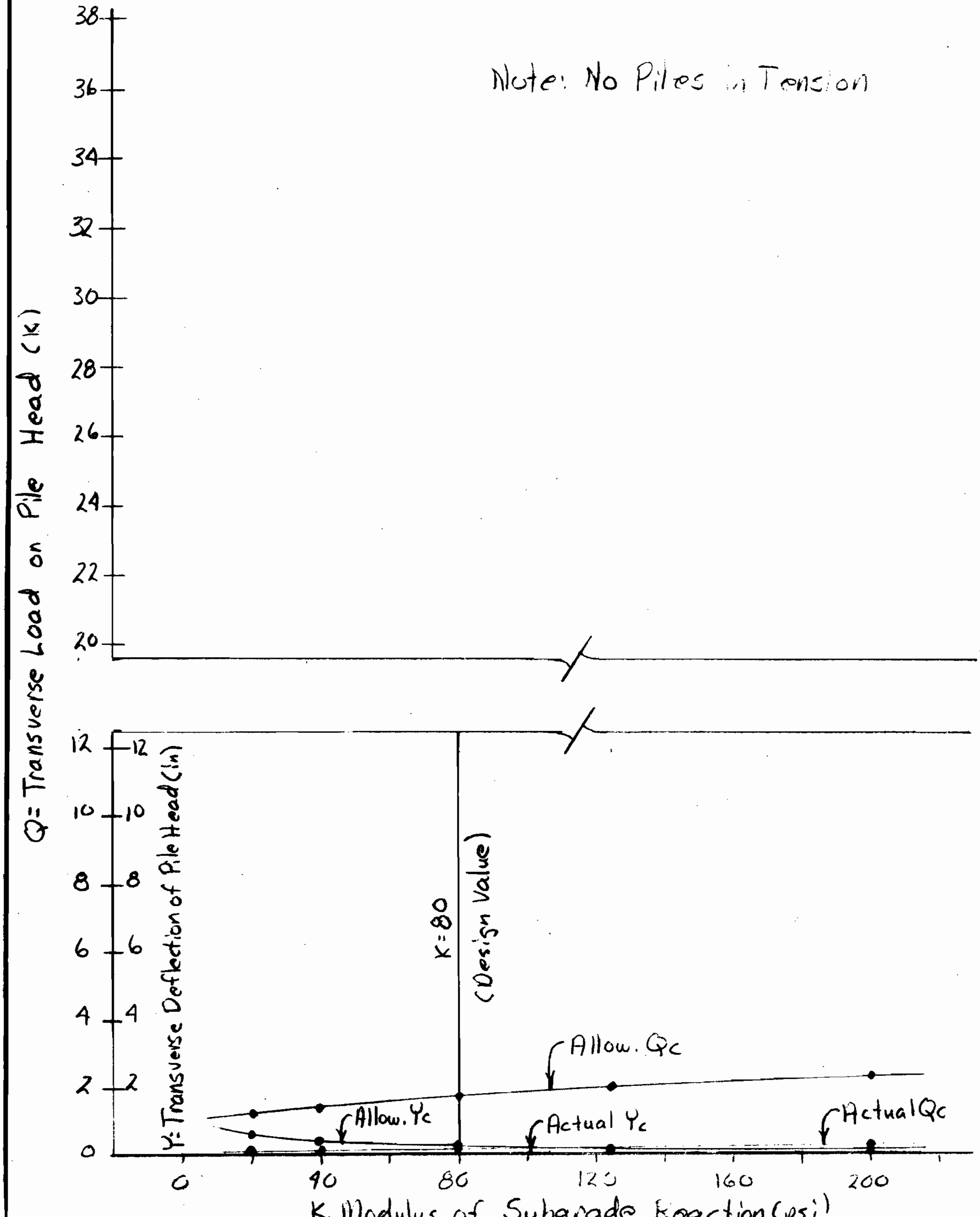
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIERS 3 & 4 DYNAMIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453B1
		DATE 10/10/72 PAGE 1 OF 1 PAGES



ENCLOSURE 7

FOR USE ON U.S. GOVERNMENT WORK ONLY

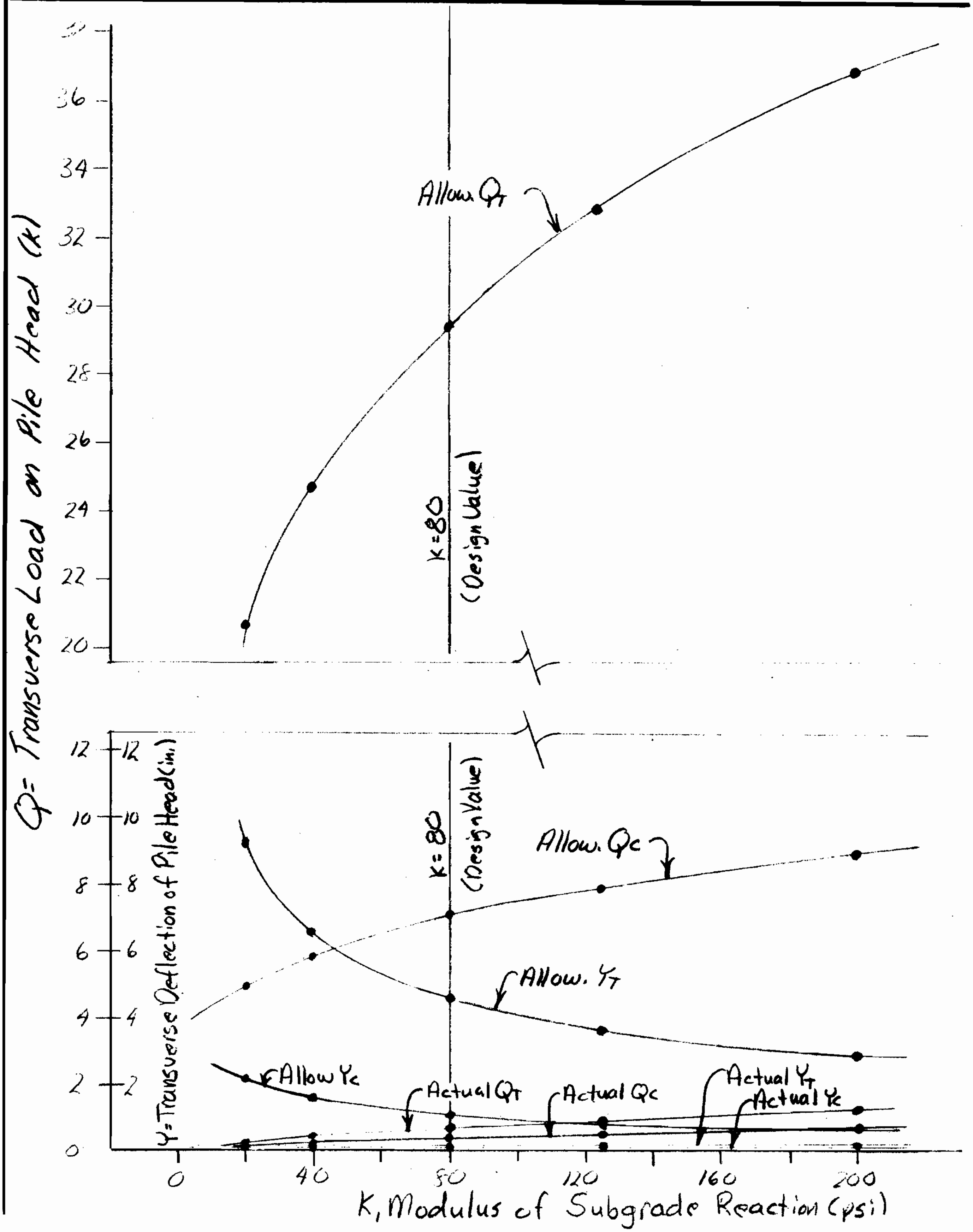
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIER 5 STATIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453 B1
		DATE 10/10/72 PAGE 1 OF 1 PAGES



ENCLOSURE 8

FOR USE ON U.S. GOVERNMENT WORK ONLY

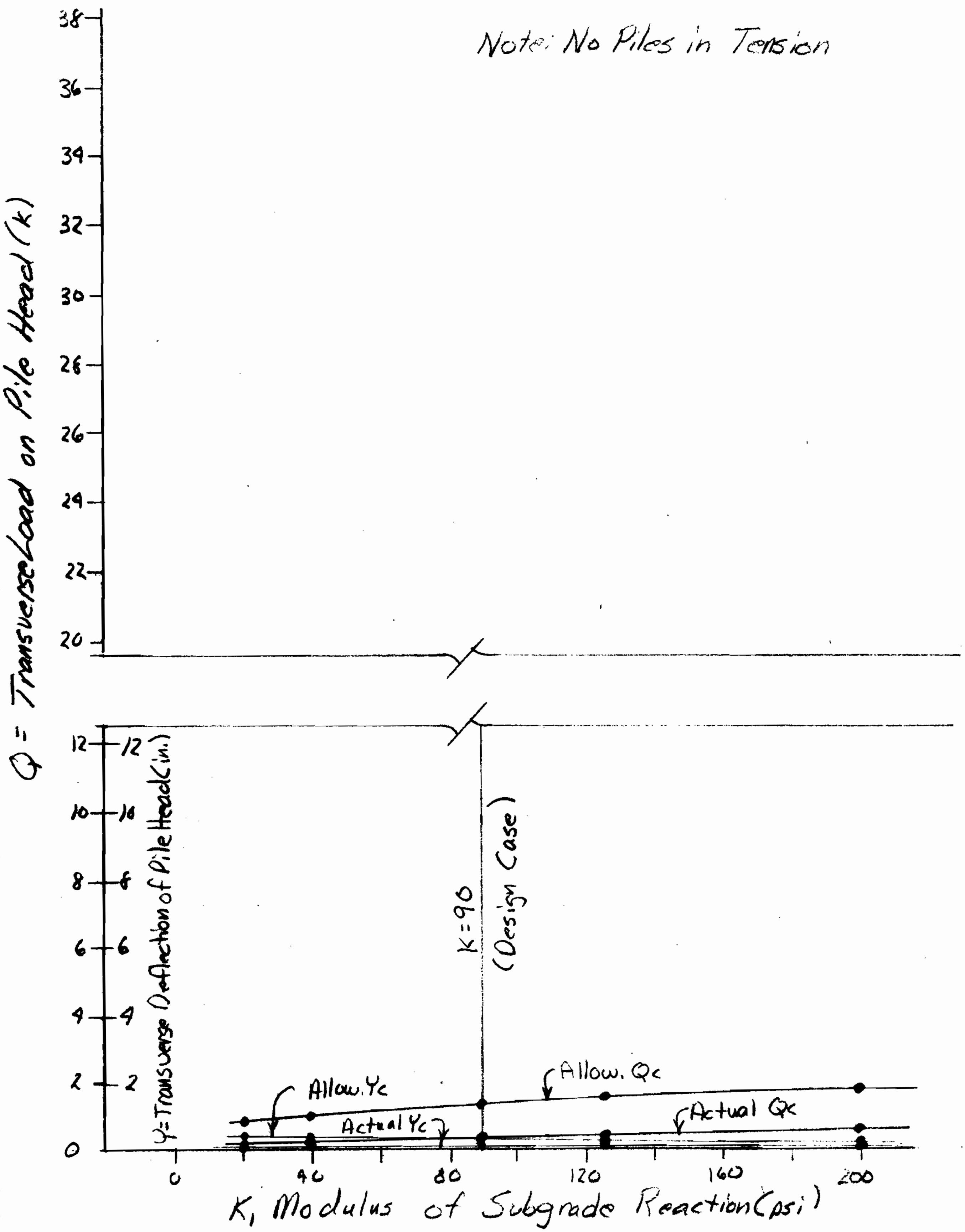
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIER 5 DYNAMIC COMPUTED RWR CHECKED	PROJECT Chef Menteur FILE NO. 453B1 DATE 10/10/72 PAGE 1 OF 1 PAGES



ENCLOSURE 9

FOR USE ON U.S. GOVERNMENT WORK ONLY

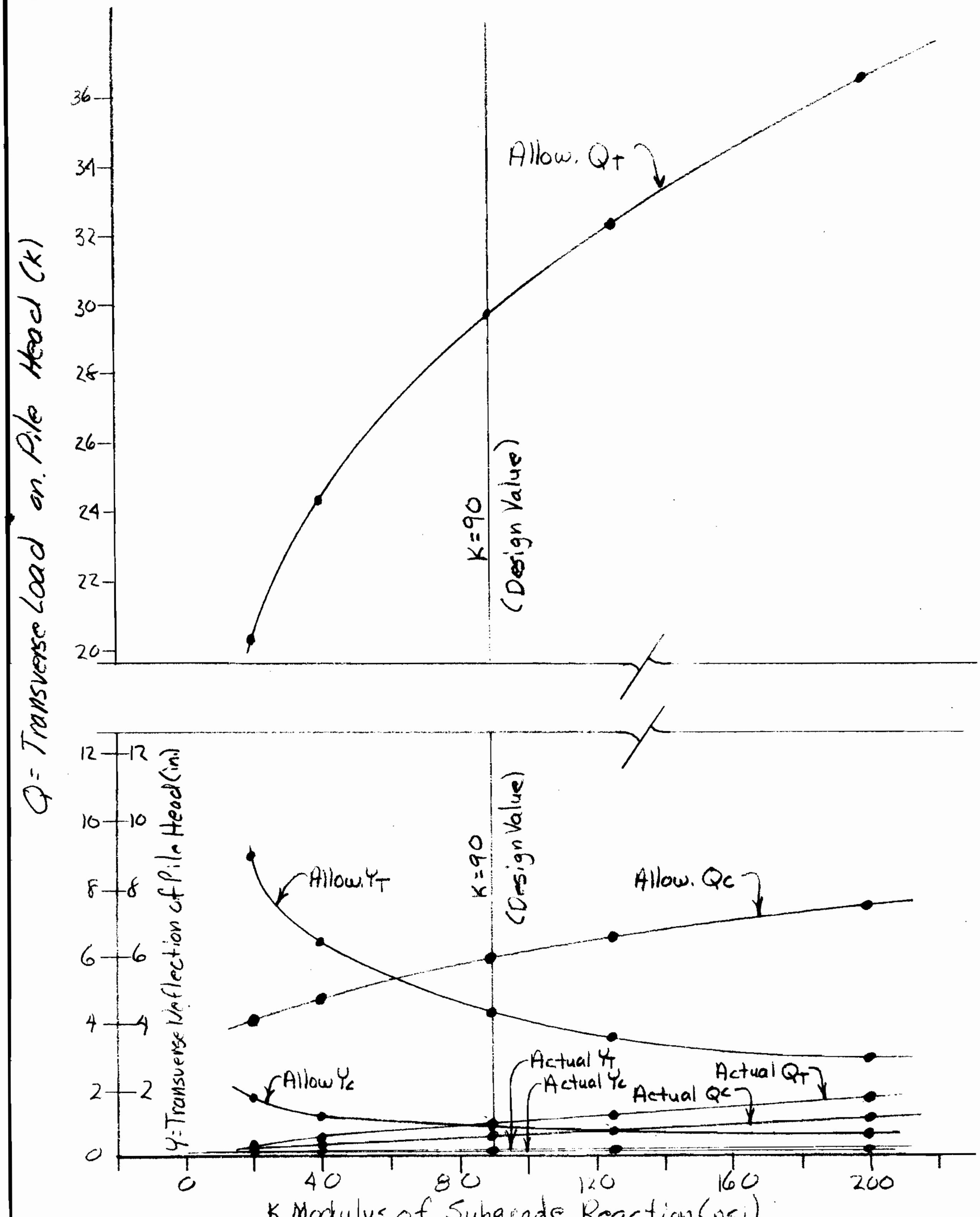
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIER 6 STATIC COMPUTED RWR CHECKED	PROJECT Chef Menteur FILE NO. 453B1 DATE 10/10/72 PAGE 1 OF 1 PAGES



ENCLOSURE 10

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Q & Y vs. K - STEEL BARRIER PIER 6 DYNAMIC	PROJECT Chef Menteur
	COMPUTED RWR CHECKED	FILE NO. 453B1
		DATE 10/10/72 PAGE 1 OF 1 PAGES



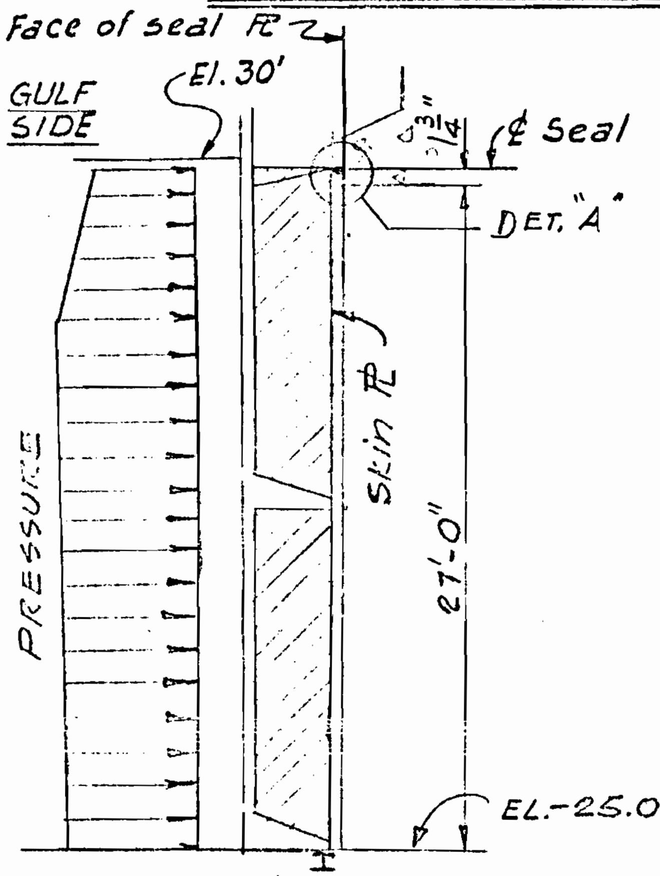
APPENDIX C

Control Structure Calculations

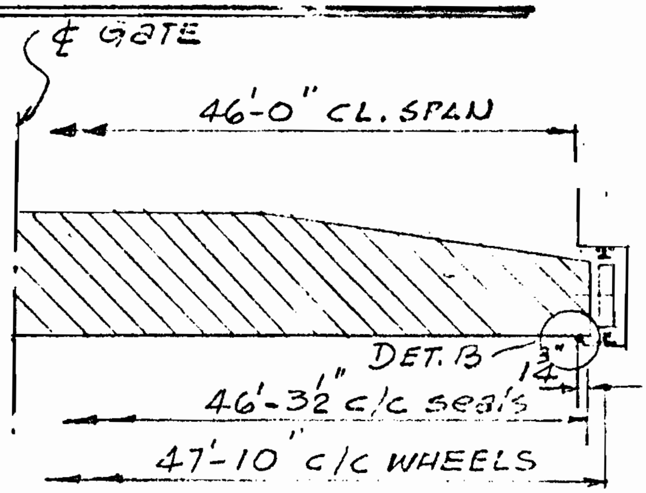
FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE GATE. GEN. ARRANGEMENT COMPUTED H.S. CHECKED	PROJECT CHEF MENTEUR FILE No. 453 B1 DATE MAY 1971 PAGE 1 OF ___ PAGES

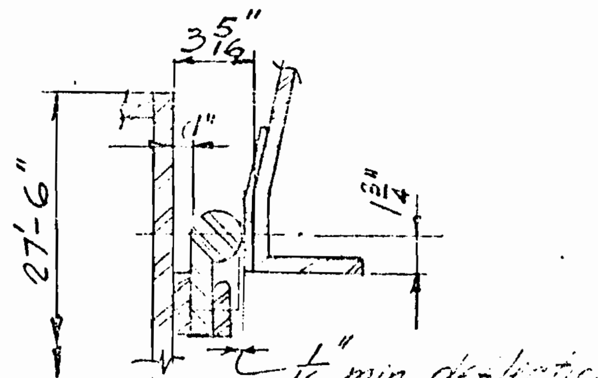
CONTROL STRUCTURE WHEEL GATES



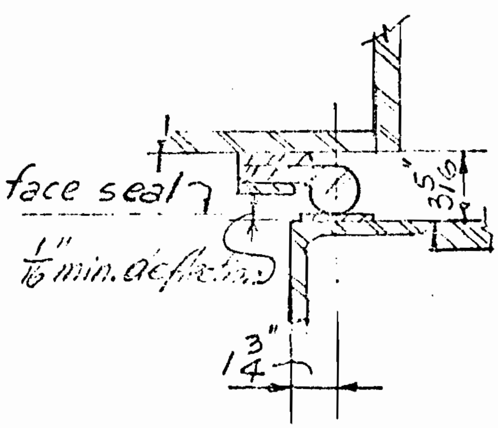
TYP. CROSS SECTION



SECTION A-A



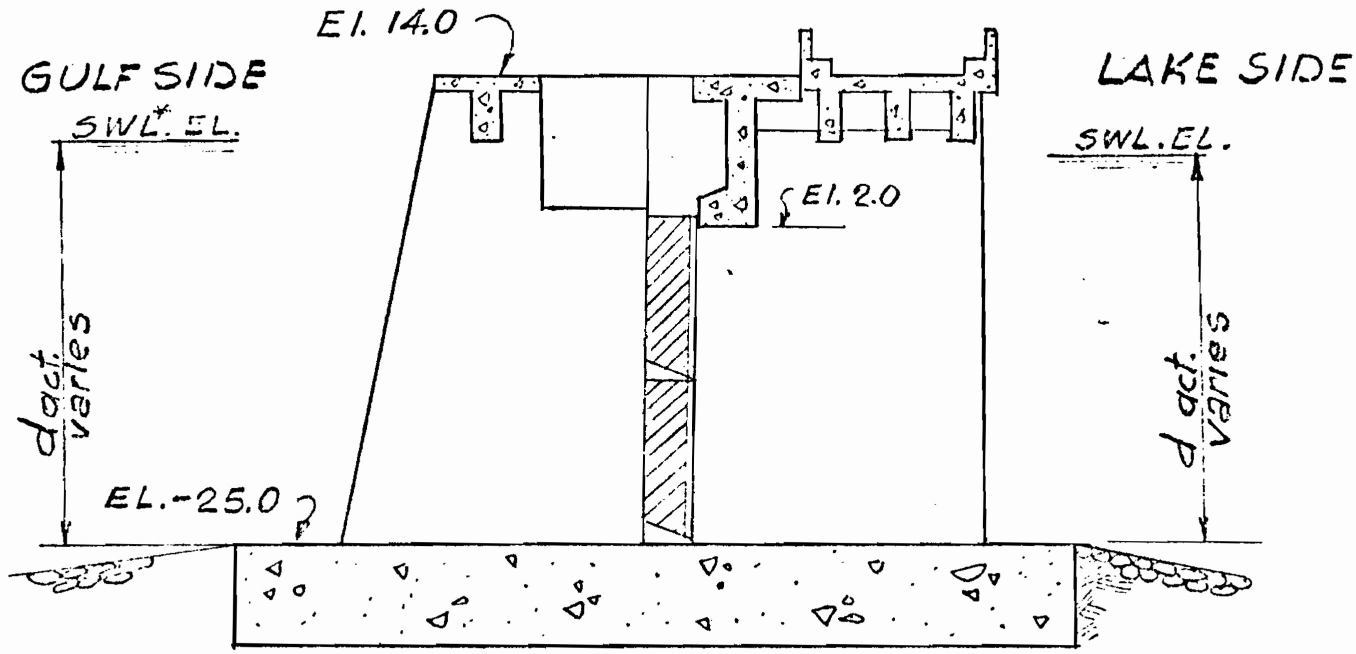
DETAIL-A



DETAIL-B

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WAVE CLASSIFICATION COMPUTED H.S. CHECKED [illegible]	PROJECT CHEF MENTEUR FILE NO. 453-B1 DATE MAY 1971 PAGE 2 OF ___ PAGES



CLASSIFICATION OF WAVES

REFERENCE:

1. STATICS - DESIGN HURRICANE - RECORD FROM N.O.
2. SHORE PROTECTION PLANING & DESIGN T.R. No 4
3. SINCE IN CHANNEL NONBREAKING WAVE CONDITION EXIST, THE DESIGN WAVE OF H_{10} IS SELECTED AS SUITABLE FOR SEMIRIGID STRUCTURES

CASE	SWL* ELEV.	H_{10} Ft	d_b Ft	d_{act} Ft	WAVE CLASSIFICATION
1	12.8	9.30	10.48	37.8	STANDING
2	11.8	8.48	9.60	36.8	"
3	10.5	7.44	8.52	35.5	"
4	11.5	7.62	11.67	36.5	"

* SWL — STILL WATER ELEVATION

Dynamic Load for Gate Design.

Gates should be investigated for a design Wave H_{10} (average of the highest 10% of all waves) as in structure loading the design wave will be a standing wave.

	Case 1	Case 2	Case 3	Case 6
H_{10}	9.30'	8.48'	7.44'	7.62'
L_o (deepwater wave length, eq. 1-5)	229.84'	209.72'	190.52'	471.86'
S.W.L (STILL WATER ELEV)	+12.8	+11.8	+10.5	+11.5
d (depth of water at structure)	37.8	36.8	35.5	36.5
d/L_o	0.1642	0.1758	0.1864	0.0776
1) Table D-1 d/L_d	0.1952	0.2048	0.2137	0.1208
L_d	193.8	180	166	302
2.) Table 4-2 $L_d h_o$	325	260	195	275
h_o	1.68	1.44	1.17	0.91
2) Table 4-3 (P_1) pressure at bottom in lbs/S.FT. SEE SHT 3a	320	280	240	350
Crest of Clapolis (EL) (SWL + h_o + h)	+23.78	+21.72	+19.11	+20.03

(1) STRUCTURAL DESIGN, TABLE D-1, Appendix D,
(2) " " Chapter 4, Physical Factors

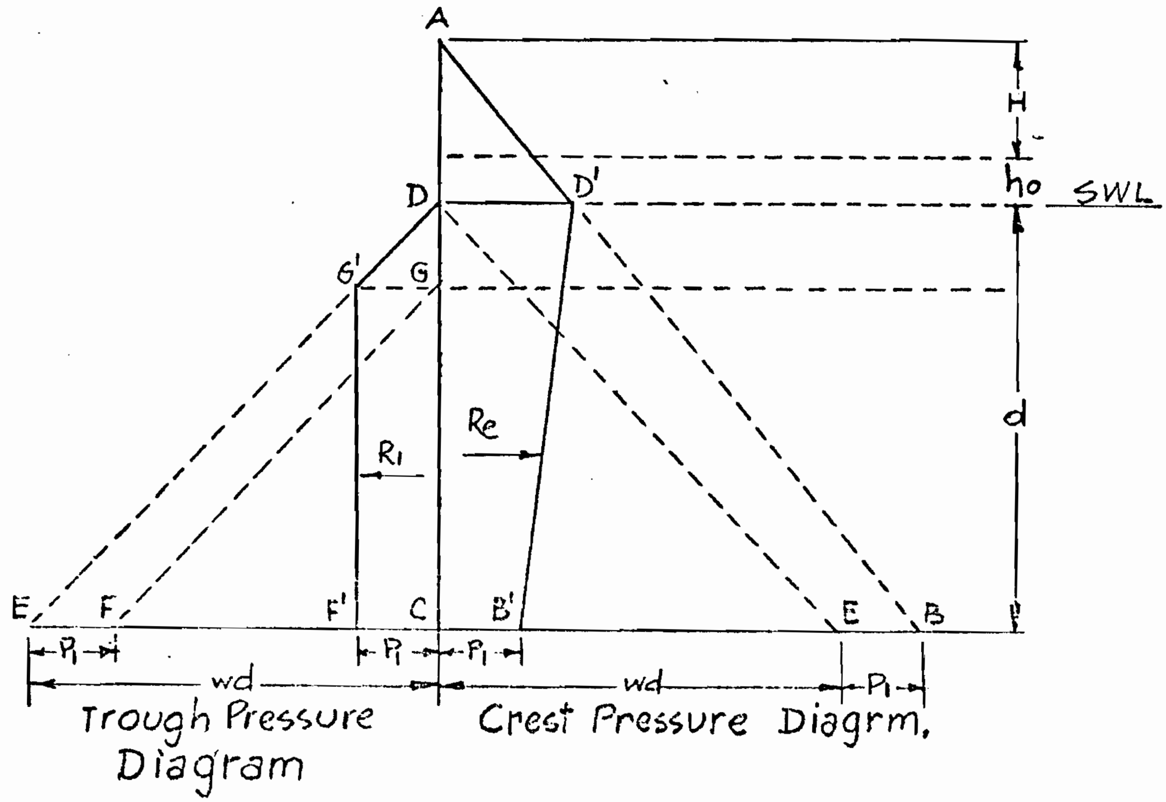


FIGURE 4-1 CLAPOTIS ON VERT. WALL

d = depth from stillwater level

H = Height of Original Free Wave

L = Length of Wave

P_1 = Pressure at Clapotis, Adds to or Subtracts from Stillwater Pressure.

h_o = Height of Orbit Center (Or mean level) above SWL.

$$P_1 = \frac{wH}{\cosh \frac{2\pi d}{L}}$$

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WAVE LOADING (CHANNEL)	PROJECT CHEF MENTEUR
	COMPUTED [illegible] CHECKED [illegible]	FILE No 453 BI
		DATE MAY 1971 PAGE 5 OF ___ PAGES

Water loading on gates

cases: 1 THRU 4, 6 & 7

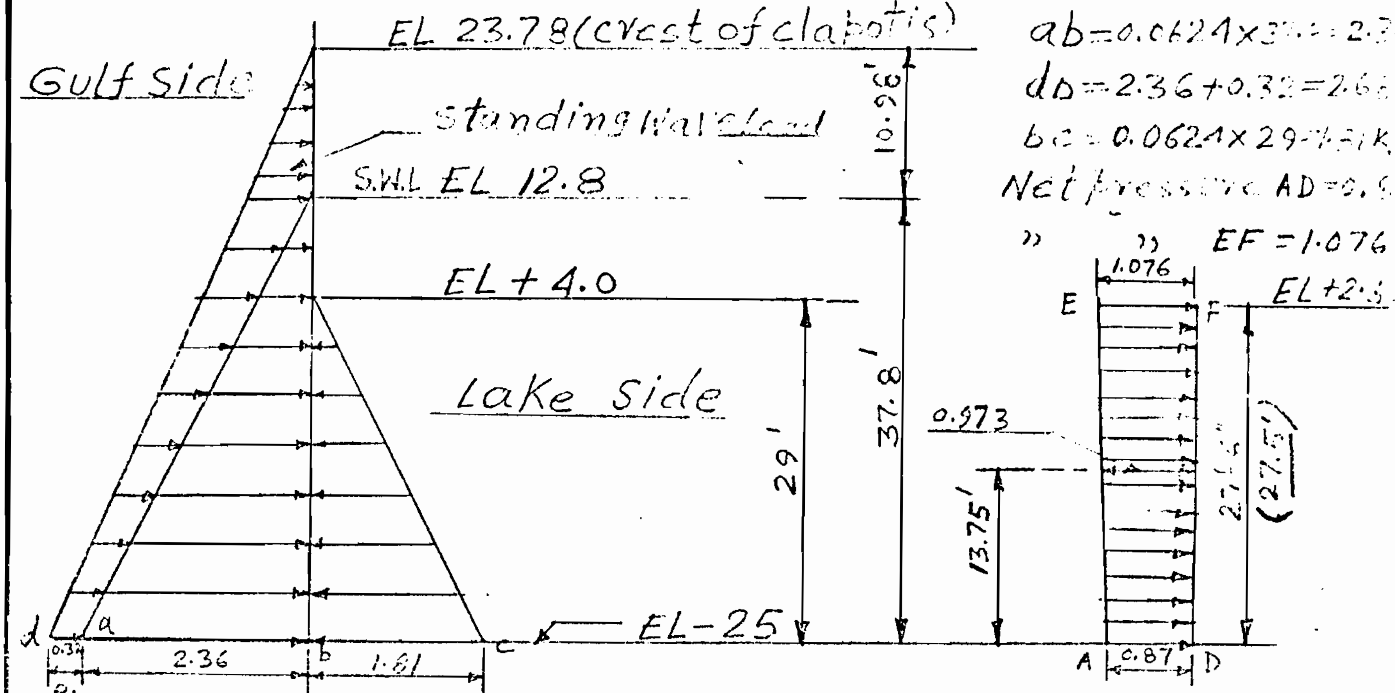
ab = 0.0624 × 3[illegible] = 2.3[illegible]

dD = 2.36 + 0.3[illegible] = 2.6[illegible]

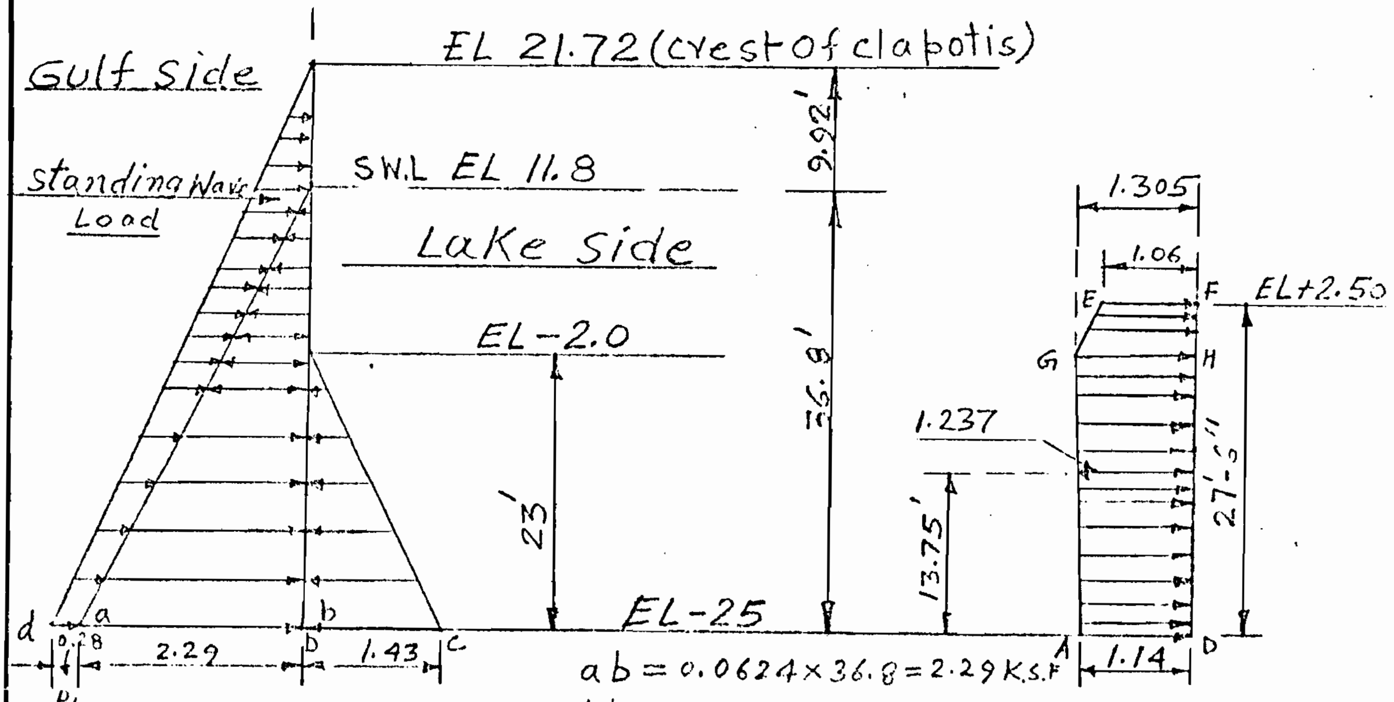
bc = 0.0624 × 29 = 1.81 K[illegible]

Net pressure AD = 0.[illegible]

" " EF = 1.076 K[illegible]



CASE-1



CASE 2.

ab = 0.0624 × 36.8 = 2.29 K.S.F

db = 2.29 + 0.28 = 2.57 K/S.F.

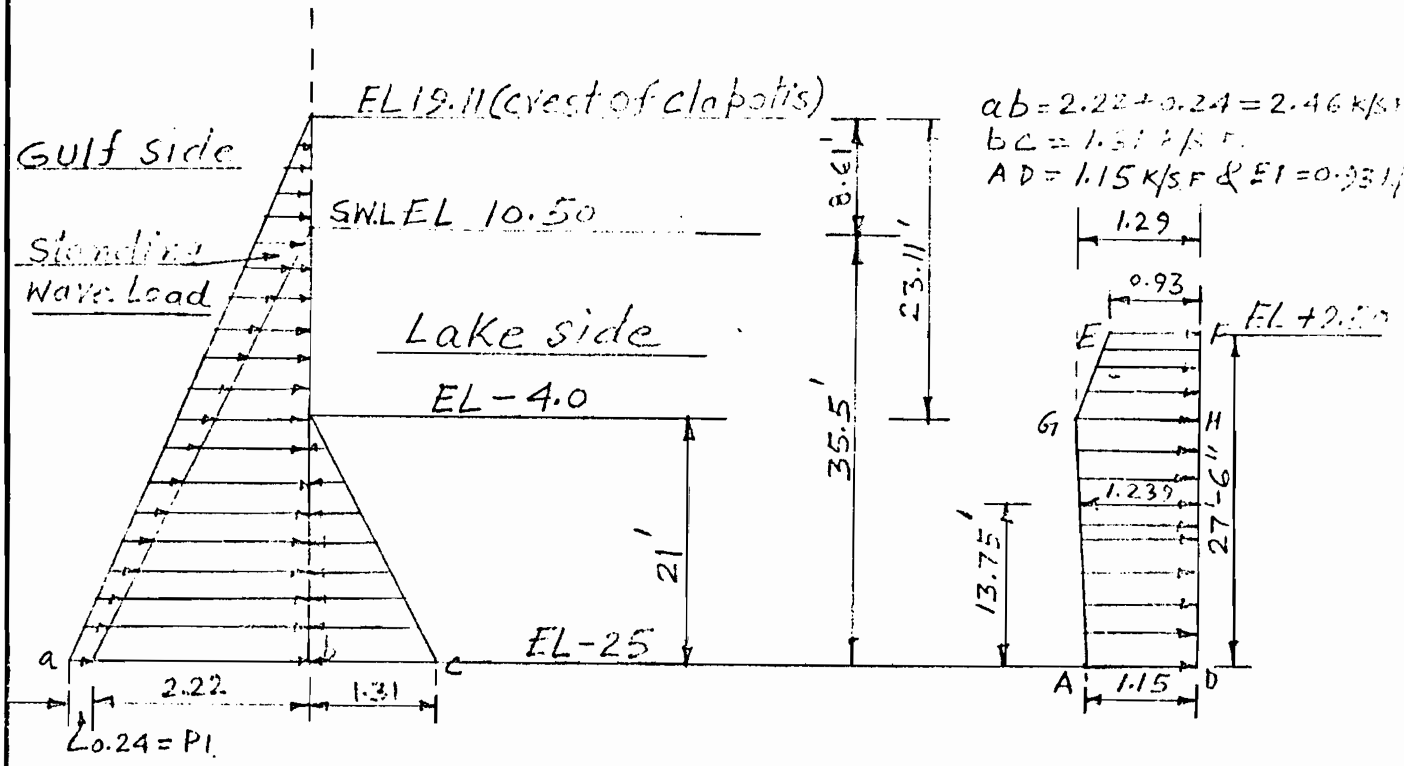
bc = 0.0624 × 23 = 1.43 K/S.F.

Net Pressure AD = 1.14 K/S.F.

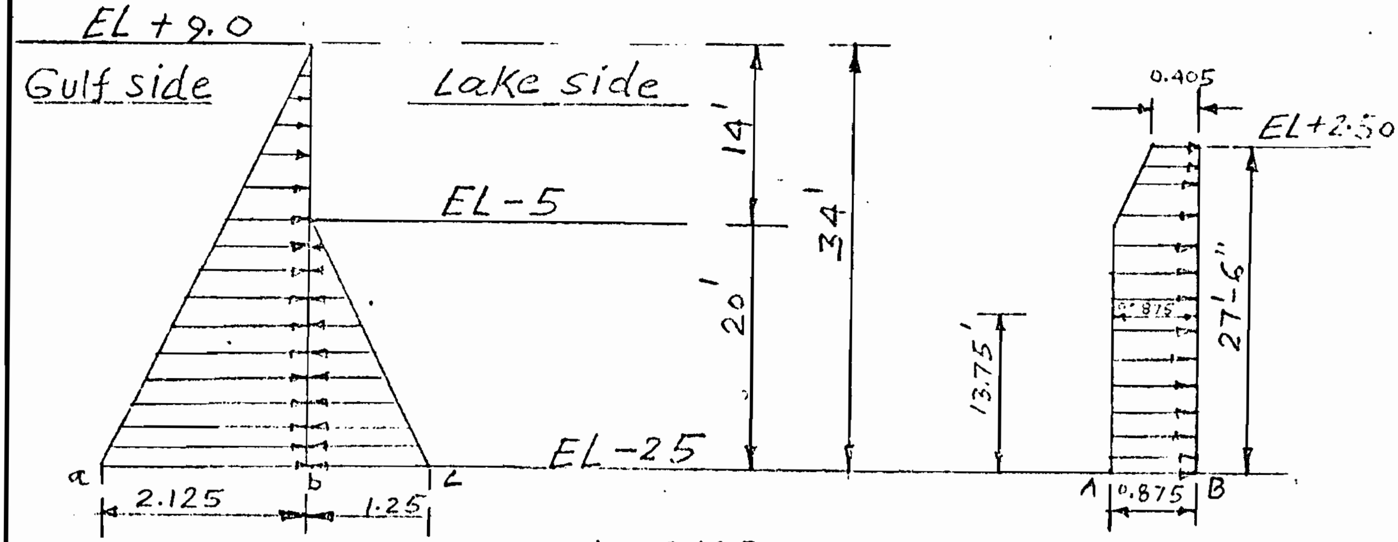
" " EF = 1.06 K/S.F.

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WAVE LOADING (CHANNEL)	PROJECT CHEF MENTEUR
	COMPUTED ARW CHECKED [illegible]	FILE No. 453 B1
		DATE MAY 1971 PAGE 6 OF ___ PAGES

ab = 2.22 + 0.24 = 2.46 K/SF
bc = 1.31 K/SF
AD = 1.15 K/SF & EF = 0.93 K/SF

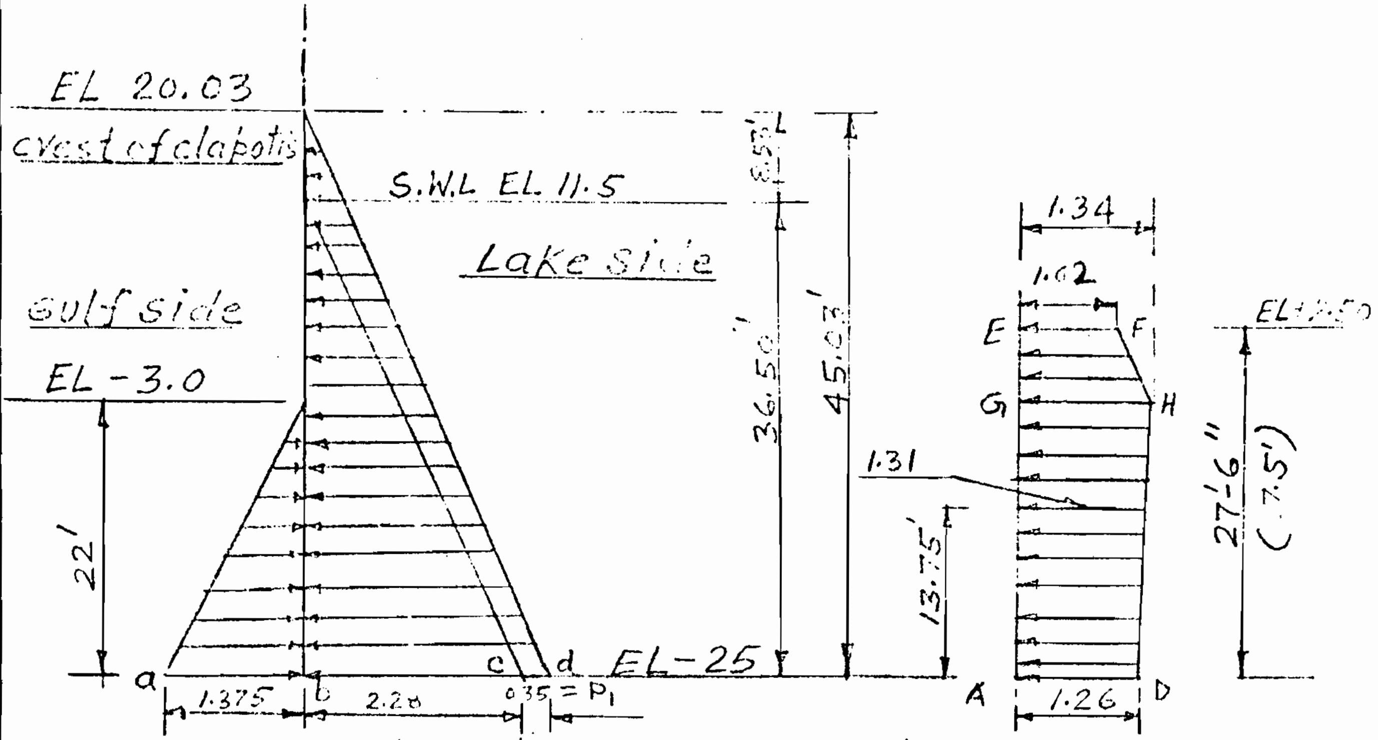


CASE-3



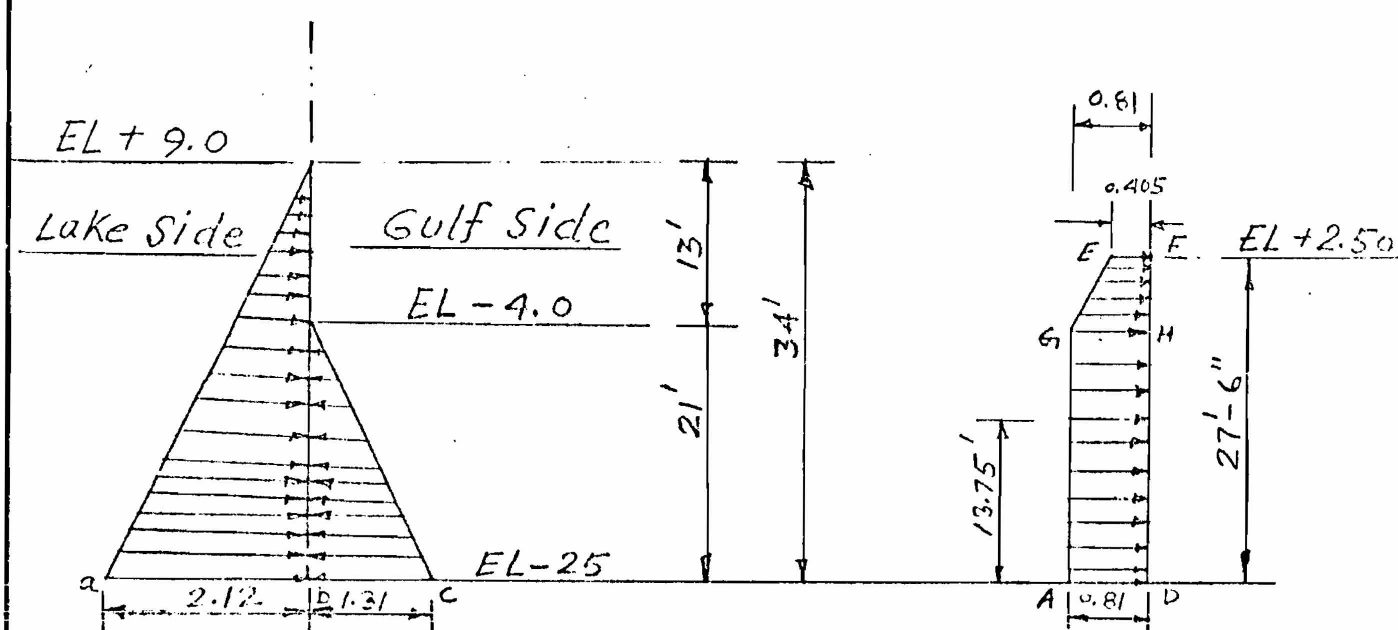
CASE-4

ab = 2.125 KSF.
bc = 1.25 KSF
Net Pressure AB = 0.875 KSF
" " EF = 0.405 KSF.



CASE-6.

$bc = 2.28$ K/S.F.
$ab = 1.375$ K/S.F.
$bd = 2.28 + 0.35 = 2.63$ K/S.F.
$EF = 1.02$ K/S.F.
$GH = 1.34$ K/S.F.



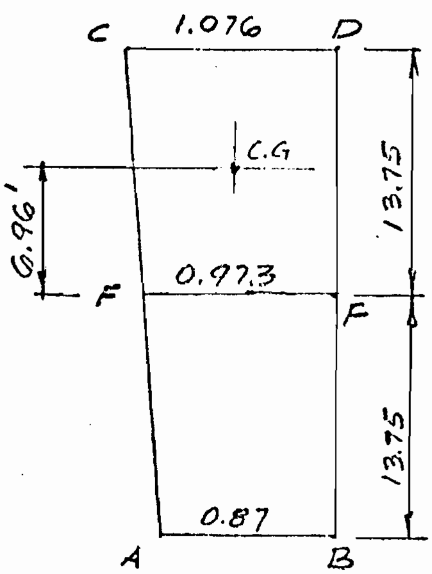
CASE-7.

$ab = 2.12$ K/S.F
$bc = 1.31$ K/S.F.

Net pressure AD = 0.81 K/S.F
" " EF = 0.405 K/S.F
" " GH = 0.81 K/S.F

LOADING CASE	AVE. LOAD/FT OF GATE SECT.(k)		TOTAL LOAD PER SECT.(k)		MAX. WHL. LD(k)		MAX. WHL. L[illegible] ADJUSTED [illegible] NORMAL ST[illegible]
	TOP	BOT.	TOP	BOT.	TOP SECT.	BOT. SECT.	
1	14.05	12.7	664	586	166	146.5	123
2	17.04	16.35	790	760	197.5	190	148
3	16.35	16.4	760	762	190	190.5	143
4	10.4	12.0	484	560	121	140	140
6	17.5	17.65	812	820	203	205	154
7	9.83	11.10	455	515	114	128	128

CENTER OF GRAVITIES



$$EF = 0.87 + \frac{0.206}{2} = 0.973^{KSF}$$

$$P_R^t = \frac{1.076 + 0.973}{2} \times 13.75 \times 46.3 \cong 605^K$$

$$P_R^b = \frac{0.87 + 0.973}{2} \times 13.75 \times 46.3 \cong 586^K$$

C.G. TOP SECTION:

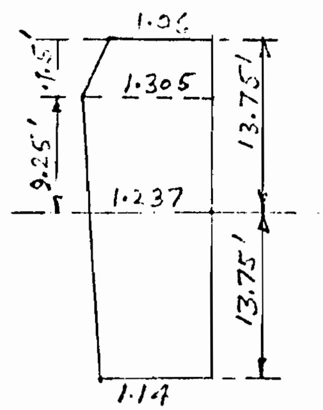
$$c = \frac{13.75(2.152 + 0.973)}{3 \times 2.054} = 6.96'$$

ECCENTRICITY $= 6.96' - 6.875 = 0.085'$

CASE-1

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WAVE LOADING (CHANNEL)	PROJECT CHEF MENTEUR
	COMPUTED [illegible] CHECKED [illegible]	FILE NO. 453 61
		DATE [illegible] PAGE 9 OF ___ PAGES

CASE - 2



C.G. top Section

$$A_1 = \frac{1.06 + 1.305}{2} \times 5.5 = 5.32$$

$$A_2 = \frac{1.305 + 1.237}{2} \times 9.25 = 11.75$$

$$A_1 + A_2 = 17.07$$

$$c_1 = \frac{4.5(2.61 + 1.06)}{3 \times 2.365} = 2.33'$$

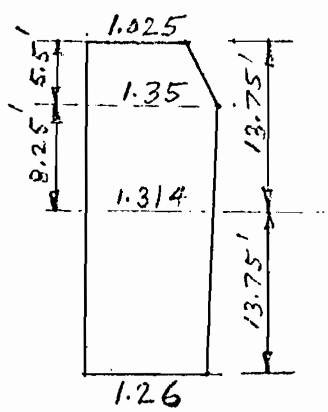
$$c_2 = \frac{9.25(2.61 + 1.237)}{3 \times 2.542} = 4.66'$$

$$11.75 \times 4.59 = 54.00$$
$$-5.32 \times 2.17 = -11.55$$

$$\therefore c = \frac{42.45}{17.07} = 2.48'$$

$$\therefore \text{Eccentricity} = 6.875' - 6.770' = 0.105'$$

CASE - 6



C.G. top section.

$$A_1 = \frac{1.025 + 1.35}{2} \times 5.5 = 6.5$$

$$A_2 = \frac{1.314 + 1.35}{2} \times 8.25 = 10.92$$

$$c_1 = \frac{5.5(2.70 + 1.025)}{3 \times 2.375} = 2.88'$$

$$c_2 = \frac{8.25(1.314 + 2.70)}{3 \times 2.664} = 4.15'$$

$$10.92 \times 4.10 = 44.80$$
$$-6.5 \times 2.62 = 17.00$$

$$\therefore c = \frac{27.80}{17.42} = 1.595' \approx 1.60' \text{ say.}$$

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT: CONTROL STRUCTURE WAVE LOADING (CHANNEL)	PROJECT: CHEF MENTEUR
	COMPUTED: [illegible] CHECKED: [illegible]	FILE NO. 453 B1
		DATE [illegible] PAGE 10 OF ___ PAGES

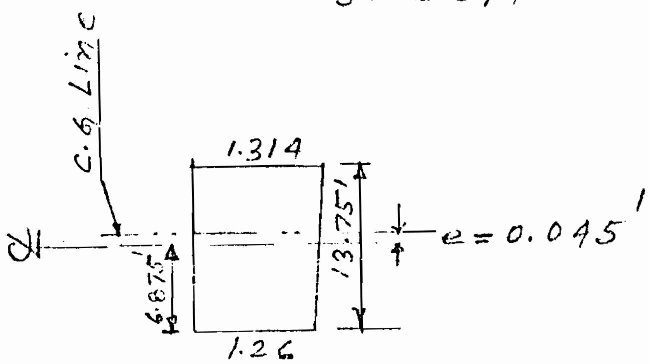
∴ Eccentricity = ([illegible] − [illegible]) = [illegible]′

$$\text{Moment} = \frac{[illegible] \times [illegible]}{1.33}\ \text{K.ft.} = 137.20\ \text{K}'$$

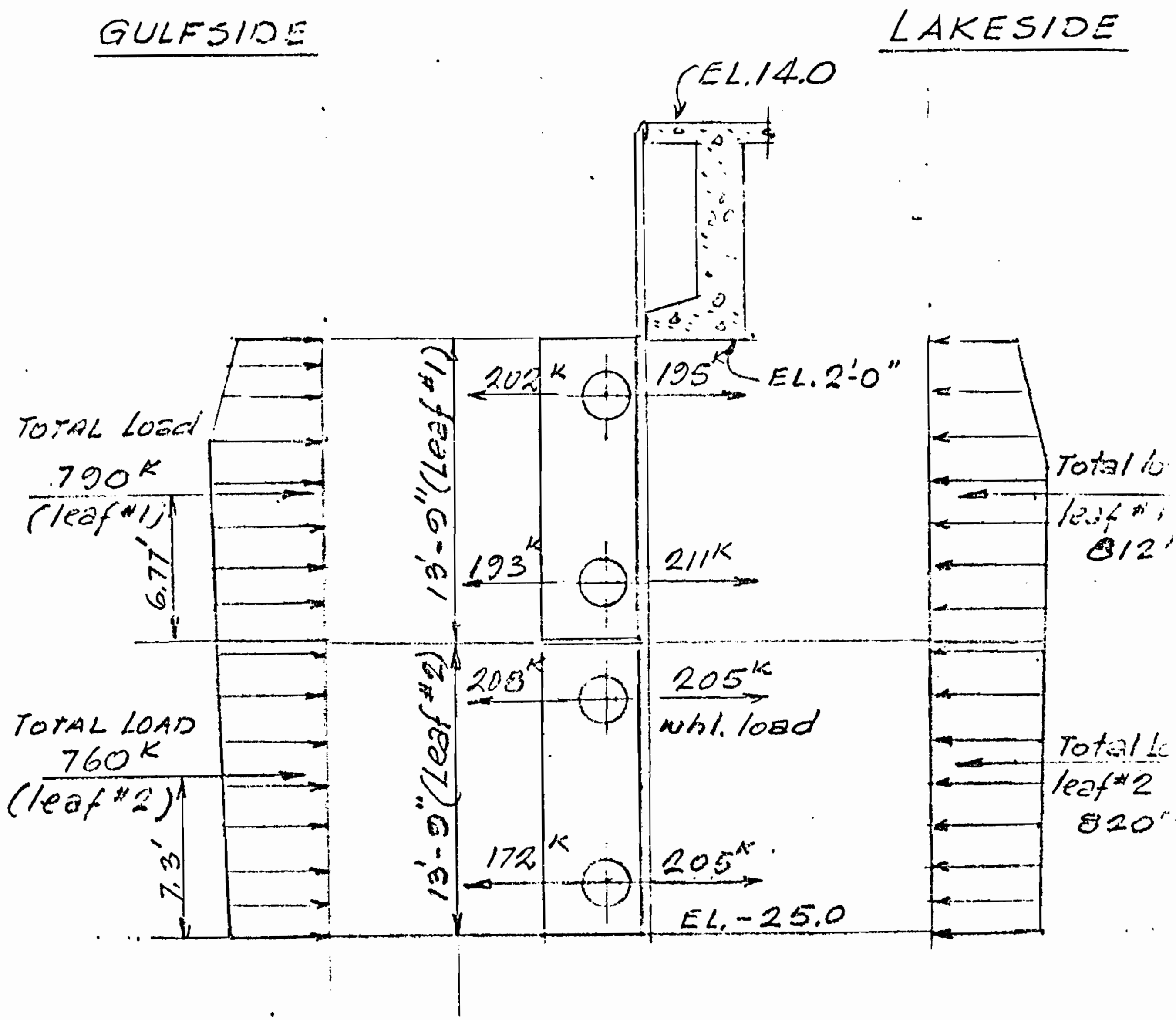
∴ Extra load/wheel on top section $= \frac{137.[illegible]}{8.5 \times 2} = \pm [illegible].0$

<u>C.G. Bottom Section</u>

$$C = \frac{13.75\,(1.314 + 2.52)}{3 \times 2.574} = \underline{6.83'}$$



Eccentricity = (6.875′ − 6.83′) = 0.045′



LOADING CASE 2 Loading case 6

LOADING DIAGRAMS

NOTE: FOR DETAILED LOADING DIAGRAMS FOR CASES 1 thru 7 see p.p. 5-7

FIGURE 1

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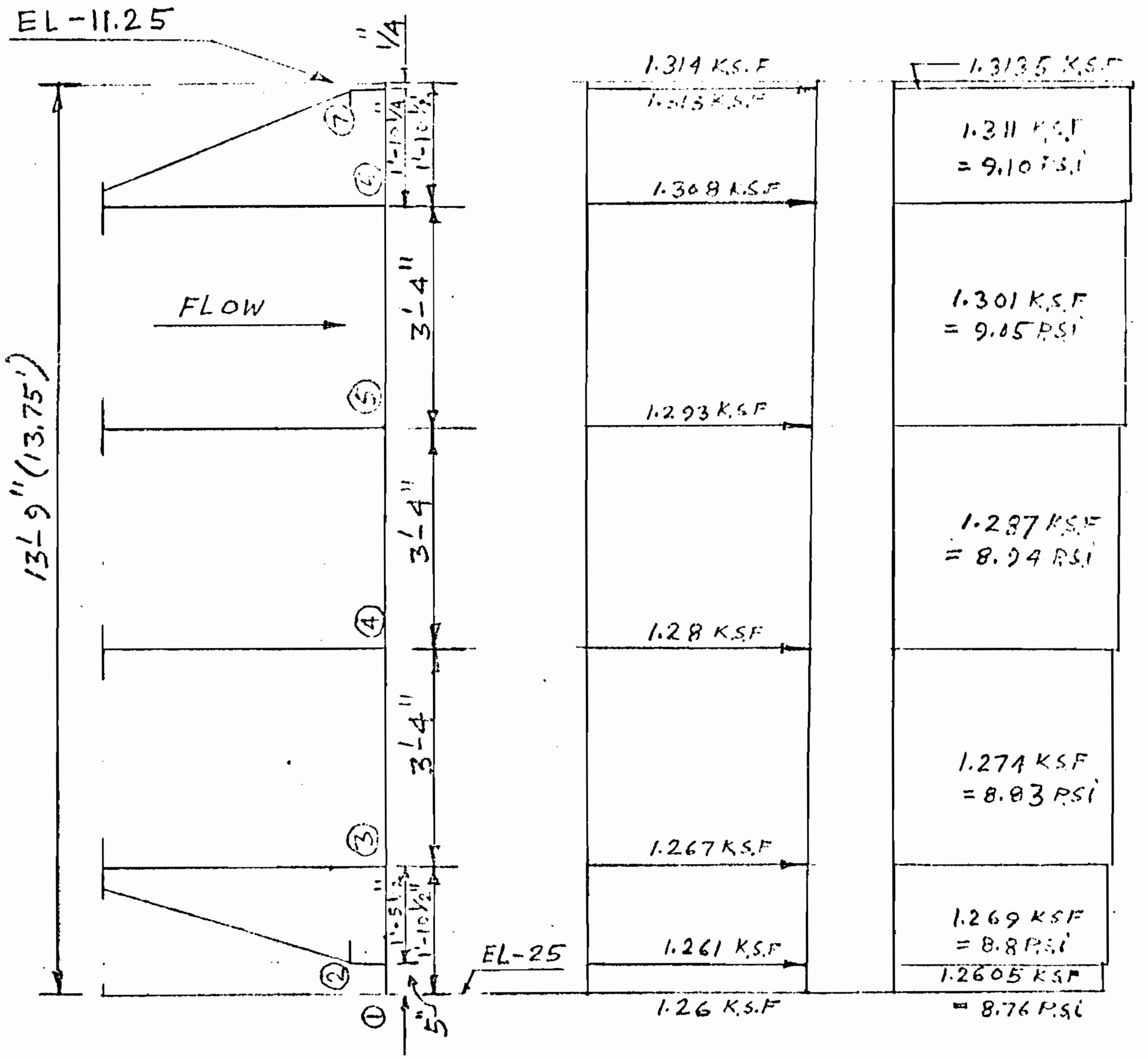
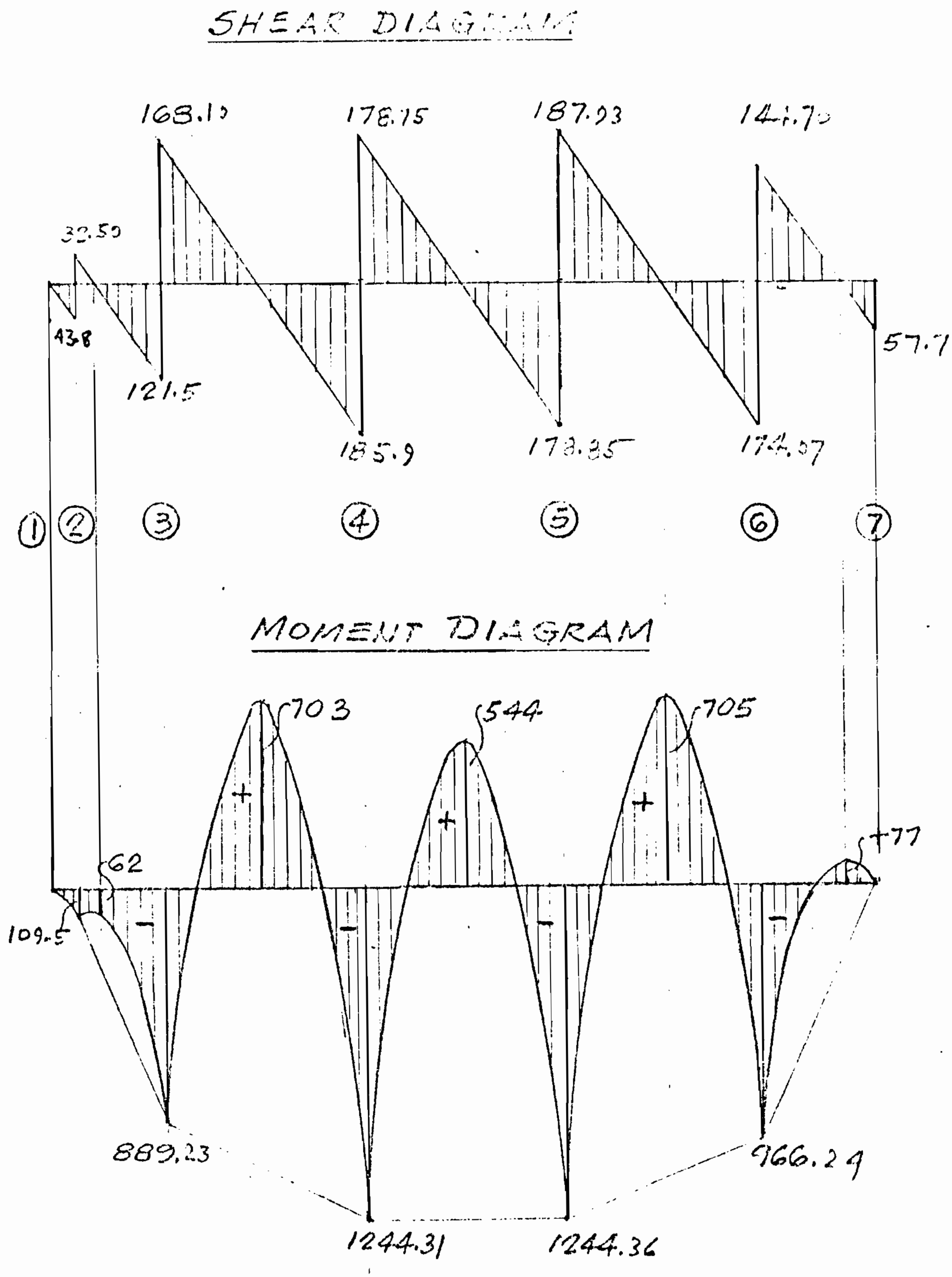
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WHEEL GATE	PROJECT CHIEF MENTRUE
	COMPUTED [illegible] CHECKED	FILE NO. 453 B1
		DATE MAY 77 PAGE 11 OF PAGES

SKIN PLATE-BOTTOM LEAF (CASE 6 GOVERNS)



	①		②		③		④		⑤		⑥	⑦
B.M. SPCG (")	5		17.5		40		40		40		22.25	
LOAD (lbs/")	8.76		8.80		8.85		8.94		9.05		9.10	
DIST. FACT		0	1	0.7	0.3	0.5	0.5	0.5	0.5	0.42	0.58	
F.E.M. lb-IN	0	+109.5	-225	+225	-1180	+1180	-1190	+1190	-1205	+1[illegible]	-376	+376
		0	0	0	0	0	0	0	0	0	-188	-376
			+115.5	+662.5	+286.5	+5	+5	+7.5	-7.5	-260	-372	0
	0		+334.3	+57.8	+2.5	+143.[illegible]	+3.75	+2.5	-134	+3.75		0
			-334.3	-42.21	-18.09	-73.5[illegible]	-73.52	+65.8	+65.7	-1.57	-2.18	
	0		-21.11	-167.2	-36.76	-9.05	+32.9	-36.76	-0.79	+32.9		0
			+21.11	+143	+61.20	-11.93	-11.92	+18.7[illegible]	+19.77	-13.[illegible]	-19.0	
	0		+71.50	+10.56	-5.97	+30.60	+9.39	-5.[illegible]	-6.9	+9.39		[illegible]
			-71.50	-3.20	-1.39	-20	-20	+6.43	+6.43	-3.94	-5.45	
	0		-1.60	-35.7	-10.0	-0.7	+3.22	-10.0	-1.97	+3.22		0
			+1.60	+32.0	+13.70	-1.26	-1.26	+6.00	+6.00	-1.35	-1.37	
	0		+16.00	+0.80	-0.63	+6.85	+3.00	-0.63	-0.68	+3.00		0
			-16.00	-0.12	-0.05	-4.77	-1.97	+0.70	+0.70	-1.26	-1.74	
MOM IN lb/in	0	+109.5	-109.5	+889.23	-889.23	+1244.31	-1244.31	+1241.36	-1241.36	+964.24	-964.24	0
V' (lbs)		43.90	77	77	177	177	178.0	178.[illegible]	181	181	101.20	101.[illegible]
ΔV (lbs)		—	44.5	44.5	8.00	8.[illegible]	0.95	[illegible]	[illegible]3	6.0[illegible]	43.50	[illegible]
V (lbs)		43.[illegible]	32.5	121.5	168.10	185.[illegible]	177.75	17[illegible]	1[illegible]	1[illegible]	[illegible]	57.7
REACT. (lbs)		76.30		289.60		3[illegible]5		36[illegible]		31[illegible].77		57.7



SHEAR AND MOMENTS FOR SKIN PL.
CASE 6

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WHEEL GATE	PROJECT CHEF MENTEUR
	COMPUTED [illegible] CHECKED [illegible]	FILE NO. 453.31
		DATE [illegible] PAGE 14 OF ___ PAGES

SKIN PL DESIGN

Use say 3/4" th skin PL

$$S_{\cdot SKIN\,PL} = \frac{1\times(0.75)^2}{6} = 0.0938\ in^3/in$$

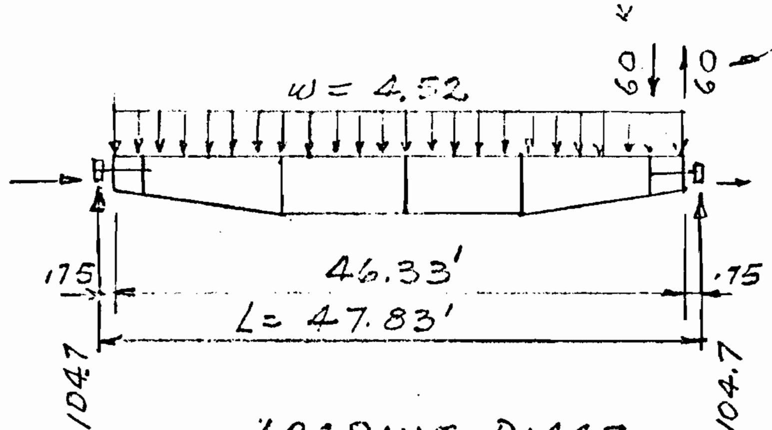
SKIN PL BENDING (CASE 6)

PTS	MAX. SPAN MOMENT IN-Lb	MAX. SUPPORT MOMENT IN-Lb	ACTUAL STRESSES f_b (psi)		ALLOWABLE STRESSES (PSI)
			SPAN	SUPPORT	
1-2	—	109.5	—	1165.	26,500
2-3	62	889.23	658	9470	"
3-4	703	1244.31	7500	13,250	"
4-5	544	1244.36	5780	13,250	"
5-6	705	966.24	7520	10,300	"
6-7	77	—	820	--	"

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	Subject: Control Structure, Wheel Gate	Project: [illegible]
	Computed: H.S. Checked:	File No: [illegible]
		Date: [illegible]/71 Page 15 of

BEAM DESIGN (Analize max. loaded Bm ⑥)



From cantlv. wheel React. see p. 22

p. 12

$w = (318.7 + 57.7) \times 12 = 45[illegible]$

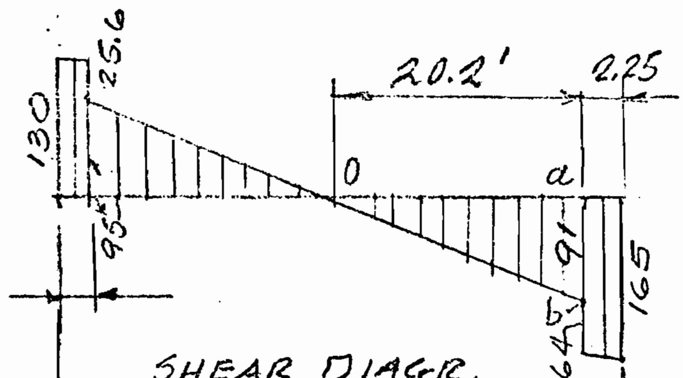
$= 4.52^{K/I}$

LOADING DIAGR.

REACTIONS:

From water pressure:

$4.52 \times 23.17 \quad 104.7$



$V_1 = 104.7 + 25.6 \cong 130^K$

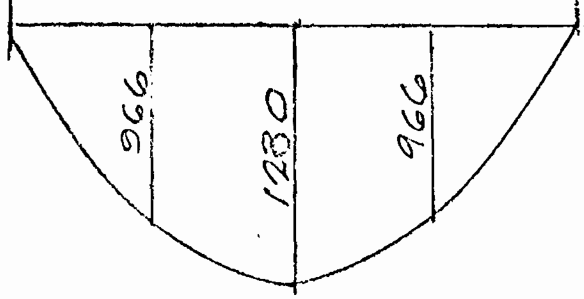
$V_2 = 104.7 + 60 \cong 165^K$

$V_{ab} = 165 - 64 - 4.52 \times 2.25$

$= 91$

SHEAR DIAGR.

$x = \frac{91}{4.52} = 20.2'$



$M_{\mathcal{L}} = 91 \times \frac{20.2}{2} + 155 \times 2.25 + 10 \times \frac{2.25}{2} = 1280^{\prime K}$

$M_{L/4} = 104.5 \times 11.96 - 4.52 \times \frac{11.21^2}{2}$

$= 1250 - 284 = 966$

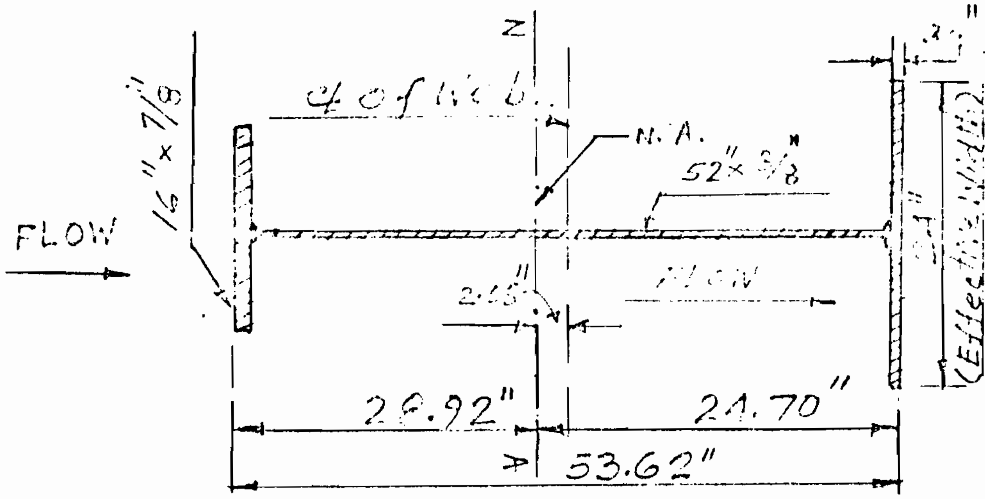
MOMENT DIAGR.

By load proportion moments for beams ② & ⑤ are:

At $\mathcal{L}$ $M_3 = M_5 = 1290 \times \frac{366.78}{376.4} = 1260^{\prime K}$

$M_4 = 1290 \times \frac{364.65}{376.4} = 1250^{\prime K}$

Intermediate Beam Design (4&5) CASE 6 CONT.



$b_e = 0.75 \times 32 = 24"$

$A_1 = 24 \times \frac{3}{4} = 18 \, in^2$

$A_2 = 52 \times \frac{3}{8} = 19.5 \, in^2$

$A_3 = 16 \times \frac{7}{8} = 14 \, in^2$

	AREA(A)	Y	AY	d	Ad^2	I_o	I_{x-x}
SKIN PL.	18.0	0.375	6.76	24.33	10650	—	10650.00
WEB PL.	19.5	26.75	522	2.05	82	4400.00	4482.00
FLG PL.	14.00	53.187	743	28.48	11320	—	11320.00
	51.50	24.70	1271.76				26452

$\therefore I_{x-x} = 26452 \; in^4$

Section Modulus:

$$Z_{SKIN} = \frac{26452}{24.70} \, in^3 = 1070 \, in^3$$

$$Z_{flg} = \frac{26452}{28.92} \, in^3 = 916 \, in^3$$

Stresses

BM-5

$$f_{b(SKIN)} = \frac{1260 \times 12}{1070} \, ksi = 14.1 \, ksi = 14{,}100 \, psi < 26{,}500 \text{ all.}$$

$$f_{b(Flg)} = \frac{1260 \times 12}{916} \, ksi = 16.5 \, ksi = 16{,}500 \, "$$

BM-4

$$f_{b(SKIN)} = \frac{1250 \times 12}{1070} \, ksi = 14.0 \, ksi = 14{,}000 \, "$$

$$f_{b(Flg)} = \frac{1250 \times 12}{916} \, ksi = 16.35 \, ksi = 16{,}350 \, "$$

case 6 cont.

CHECK FOR DEFLECTION (AT-5)

$$\Delta_{max} = \frac{5 \times 316.74 \times (574)^4}{384 \times 29 \times 10^6 \times 26.152}$$

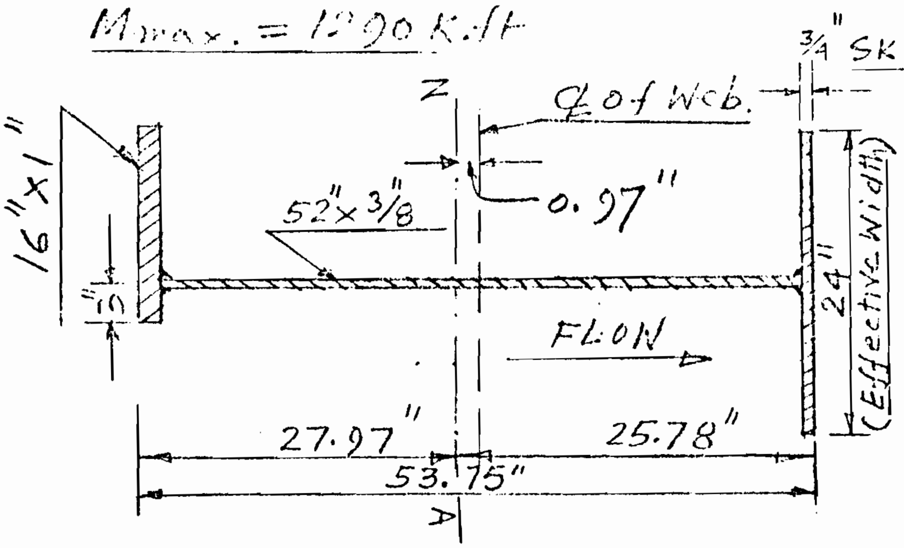
$$= \frac{1833.90 \times 1015 \times 10^8}{2941 \times 10^{11}} in = \frac{1970 in''}{2941 \times 10^{?}}$$

$\therefore \Delta_{max} = 0.67''$

Design of Beam (3&6)

LOADING = 4.52 K/ft.

$M_{max.} = 1290$ K-ft



$b_e = 0.75 \times 32 = 24''$

$A_1 = 24 \times \frac{3}{4} = 18 in^2$

$A_2 = 52 \times 3/8 = 19.5 in^2$

$A_3 = 16 \times 1 = 16 in^2$

	AREA(A)	Y	AY	d	Ad^2	I_o	I_{x-x}
SKIN PL.	18.00	0.375	6.75	25.405	11620	—	11620
WEB PL.	19.50	26.75	522	0.97	18.30	4400	4418.30
FLG PL.	16.00	53.25	851	27.47	12040	1.33	12041.33
	53.50	25.78	1379.75				28079.63

case 6 cont'd

Section Modulus

$$Z_{SKIN} = \frac{28072.63}{25.78} \, in^3 = \underline{1089} \, in^3$$

$$Z_{flg} = \frac{28079.63}{27.97} \, in^3 = \underline{1002} \, in^3$$

Stresses

BM-3
$$f_{b(SKIN)} = \frac{1260 \times 12}{1089} \, ksi = 13.9 \, ksi = 13,900 \, psi < 24,000$$
$$f_{b(Flg)} - \frac{1260 \times 12}{1002} \, ksi = 15.15 \, ksi = 15,150 \, ''$$

BM-6
$$f_{b(SKIN)} - \frac{1290 \times 12}{1089} \, ksi = 14.2 \, ksi = 14,200 \, ''$$
$$f_{b(FLG)} = \frac{1290 \times 12}{1002} \, ksi = 15.5 \, ksi = 15,500 \, ''$$

CHECK FOR DEFLECTION (BEAM 3 & 6)

BEAM - 6

$$\Delta_{max} = \frac{5 \times 376.47 \times (574)^4}{384 \times 29 \times 10^6 \times 28079.63} \, in = \frac{2032 \times 10^5}{3120 \times 10^5} \, in$$

$$\therefore \Delta_{max} = 0.65''$$

Check FLANGE BUCKLING FOR COMPRESSIVE STRESS.

Assume unsupported length of flange = 11'-8" = 14

$$\text{Allowable } f_b = \frac{12 \times 10^6}{\frac{Ld}{A_f}} = \frac{12 \times 10^6 \times 14}{140 \times 53.625} \, psi$$

$$= 22400 \, psi \quad O.K.$$

CHECK BEAM STRESS FOR CASE 4.

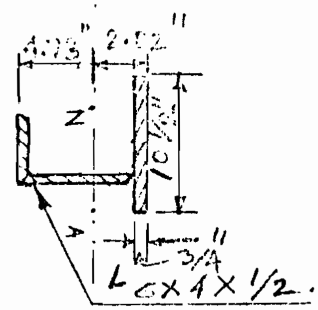
$$M_{max} = 1290 \times \frac{0.875}{1.26} \text{ K.Ft} = 896 \text{ K.Ft.}$$

$$f_{b(SKIN)} = \frac{896 \times 12}{1[illegible]} \text{ KSI} = 9.88 \text{ KSI} = 9880 \text{ PSI}$$

$$f_{b(flg)} = \frac{896 \times 12}{1002} \text{ KSI} = 10.70 \text{ KSI} = 10700 \text{ PSI}$$

EDGE SUPPORT ANGLE (2 & 7)

$$LOAD = 76.30 \times 12 \text{ \#/'} = \underline{915} \text{ \#/'} = \underline{0.915} \text{ K/'}$$



$$b_e = 17\tfrac{1}{2}'' - 12'' + 5'' = 10\tfrac{1}{2}''$$

$$A_1 = 10.5 \times 0.75 = 7.87 \text{ in}^2$$

$$A_2 = 4.75 \text{ in}^2$$

	AREA (in²)	Y	AY	d	Ad²	Io	Ix-x
SKIN PL	7.87	0.375	2.95	1.65	21.40	—	21.40
ANGLE	4.75	4.76	22.60	2.74	35.60	17.40	53.00
	12.62	2.02	25.55				74.40

Assuming continuously supported on a span of 11'-6" horizontally

$$MOM = \frac{1}{10} \times 0.915 \times 11.5^2 = \underline{12.10 \text{ K.ft.}}$$

Section Modulus

$$Z_{skin\ pl} = \frac{74.40}{2.02}\ in^3 = \underline{36.80\ in^3}$$

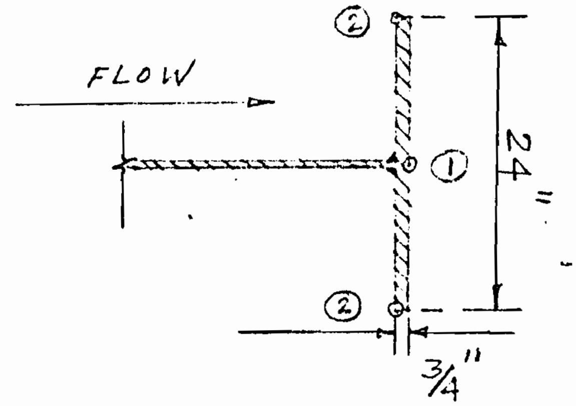
$$Z_{flg} = \frac{74.40}{4.73}\ in^3 = \underline{15.70\ in^3}$$

Stresses Reduced to Normal.

$$f_{b(SKIN\ PL)} = \frac{12.10 \times 12}{36.80}\ ksi = 3.93^{ksi} = 3{,}930^{psi}$$

$$f_{b(Flg)} = \frac{12.10 \times 12}{15.70}\ ksi = 9.23^{ksi} = 9{,}230^{psi}$$

COMBINED STRESSES



AT POINTS ① & ② THERE ARE COMPRESSIVE STRESSES IN VERTICAL DIRECTION DUE TO SKIN PL. BENDING. AT THESE SAME POINTS THERE ARE TENSILE STRESSES DUE TO BEAM BENDING. THESE STRESSES COMBINE IN A RESULTANT EQIVALENT SHEAR STRESS, $V_e = \frac{1}{2}(f_t - f_c)$.

BEAM	SKIN PL BENDING	BEAM BENDING	COMBINED SHEAR STRESS V_e	
3	- 9470	+13,900	11,685	< 16,000
4	-13,250	+14,000	13,625	"
5	13,250	+14,100	13,675	"
6	10,300	+14,200	12,250	"

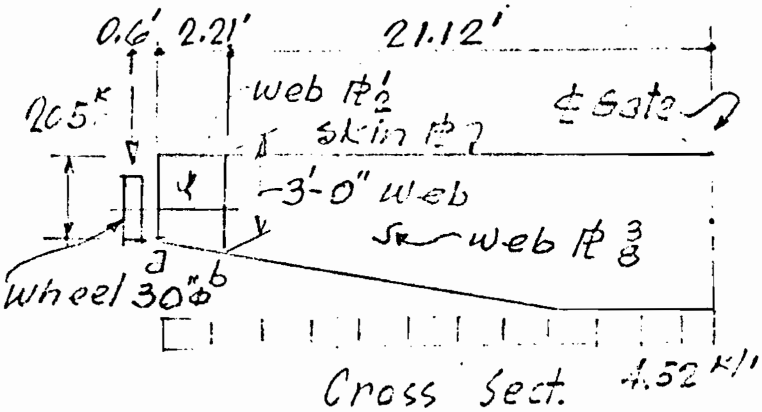
SAMPLE CALCULATION:

BEAM ⑤; $V_e = \frac{1}{2}[14010 - (-13250)] = 13,630$ psi

ALLOWABLE COMBINED BIAXIAL STRESS = 16000 P.S.I

HORIZ. BEAM DESIGN CONT'D:

CHECK SHEAR:



Cross Sect.

Hydrostatic Bm. load

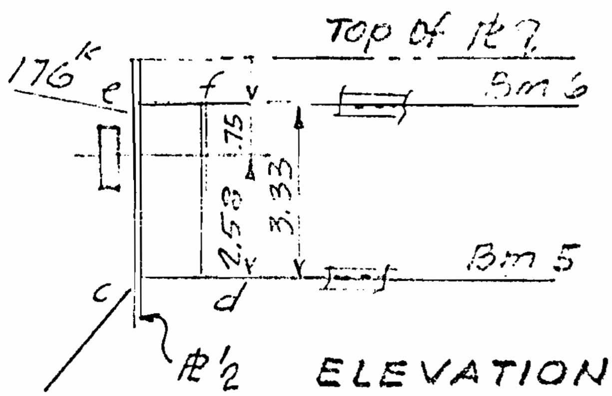
$w = 4.52^{K/'}$ (max.) Bm ⑥ p.1

Shear @ b:

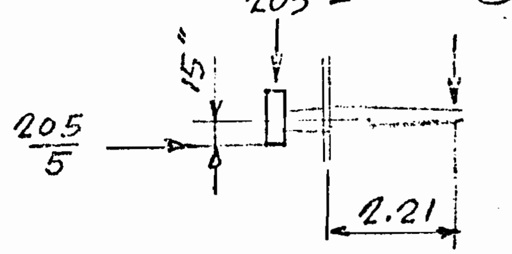
$V_b = 95.2 \cong 4.52 \times 21.12$

Web ℘ ...

$v = \dfrac{95}{36 \times .375} \cong 7.0^{ksi} < ...$



ELEVATION



BEAM SHEAR @ ENDGIRDER:

From lateral thru
Refer. LSS
para.. 53

whl. ld $= 205^K$ p.7

Addl. $\Delta R = \pm\ 78 = 205 \times \dfrac{0.6}{2.21} + \boxed{\dfrac{205}{5} \times {}^{1.25}/_{2.21}}$

max. axle $R = 283^K$

End Reaction of Bm 6 $= 104.5 \approx (4.52 \times 23.2)$

from ΔR $=$ $\underline{60.0}$ $\approx 78 \times \dfrac{2.58}{3.33}$

$R_e = 164.5 \cong 165$ or 176^K } use

$R_c = 283 - 165 = 118^K$ $\quad 121^K$ } P...

$\Sigma\ 297^K$

use for axle design

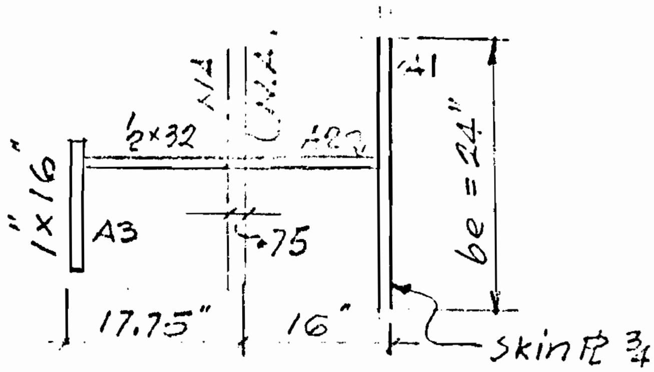
P-24

END GIRDER: $t_w = \frac{1}{2}''$

$v = \dfrac{165}{32 \times \frac{1}{2}} = 10.3 < 16^{ksi}$

HORIZ. BM. DESIGN CONT'D.:
(BEAMS 3 & 6)

MOMENT OF INERTIA AT END:



$A_1 = 18^{\square''}$ (EFFECT. SKIN)
$A_2 = 16^{\square''} = 32\times\frac{1}{2}$ WEB P
$A_3 = 16^{\square''}$ FL. P

MEMB.	A	d	Ad	y	Ay^2	I_o	$I_{gr.}$
A_1	18	.375	7.0	15.63	4400	1	4401
A_2	16	16.75	267.0	0.75	9	1365	1374
A_3	16	33.25	532.0	17.25	4760	1	4761
	50	16.0	806.0				10,536 IN^4

MAX. HORIZ. SHEAR:

$$v_h = \frac{VQ}{I_g} \quad ; \quad V_c = V_{max} = 176^k \text{ @ END p.21}$$
$$V_b = 95.2^k$$
$$I_g = 10,536\ IN^4 \text{ @ END}$$

END GIRDER

FOR SKIN P:

$Q = 18\times 15.63 = 281$ $\qquad v_h = \frac{176\times 281}{10,536} = 4.70^{k/in}$

FOR Flg.

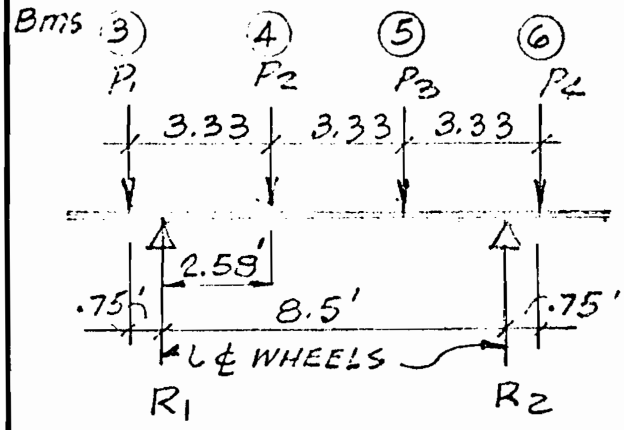
$Q = 16\times 17.25 = 275$ $\qquad v_{h(b)} = \frac{176\times 275}{10,536} = 4.60^k$

REQ'D WELDS:

class E70 Electrod

CHECK MIN. SIZE $\frac{5}{16}''$ FILLET $\quad P = (0.31\times 0.707\times 13)\times 2 = 5.75^k > 4.7^k$ o.k.

END GIRDER: (See sht 25 for load sharing ...)



$R_1 \cong R_2 = 297^K$ (SHT. 22)

$P_1 = P_4 = 176^K$

$P_2 = P_3 = 121^K$

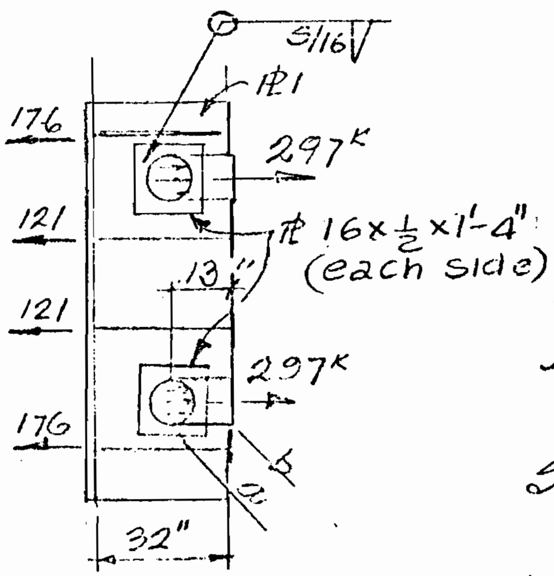
$V_{max} = P_1 = 176^K$

$M_{CANT.} = 176 \times 0.75 = 132.0^{'K}$

$M_{max} = 297 \times 2.58 = 767.0^{'K}$

$\quad - 176 \times 3.33 = -585.0$

$\quad\quad = 182.0^{'K}$



AXLE BEARING:

ASSUME 10" φ HOLE

$f_p = \frac{297}{10 \times 2} = 14.85^{ksi}$ O.K.

(2×1/2+1)

SHEAR ON a-b = 13" LENGTH

$V = \frac{0.5(297)}{13 \times 1} = 11.35^{ksi} < 16.0^{ksi}$

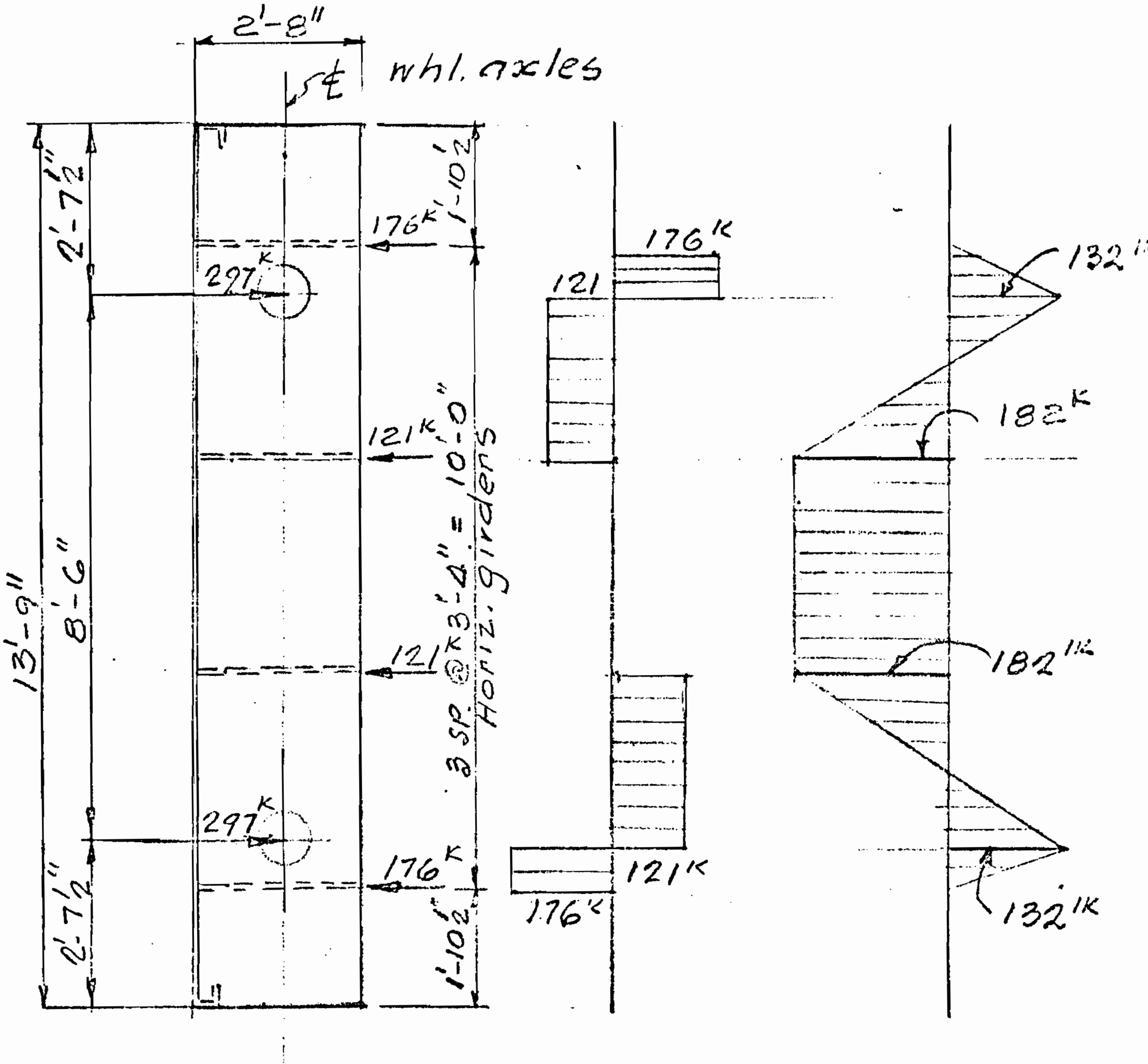
LOAD ON ONE 1/2" PL $F = 297/4 = 74.2^K$

PERIMETER: $\ell = 16 \times 2 = 32''$ Permissible stress = 15 ksi

USING 5/16" weld: (class E70-electrode)

ALLOW. $F = 0.31 \times 32 \times 0.707 \times 13 = 91^K > 74.2$

Anyway provide 5/16" weld all around. O.K.



Load diagram Shear Moment

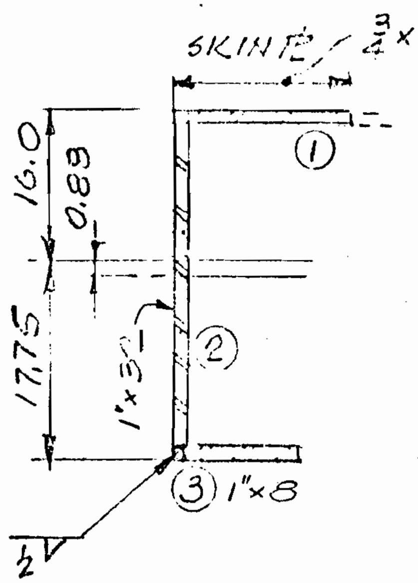
END GIRDER – BOTTOM LEAF

(TOP LEAF SIMILAR – SEE LOADING DIAGRAMS PAGES 5 THROUGH 7)

FIGURE 2

END GIRDER CONT'D:

MOMENT OF INERTIA:



SKIN PL EFFECT. LENGTH

$27/2 = 13\frac{1}{2}''$ ($\frac{1}{2}$ SPAN)

$b/t = \frac{13.5}{.75} = 18'' < 32$

AREAS:

(1) $13.5 \times .75 = 10.1\,\square''$
(2) $1 \times 32 = 32.0$
(3) $1 \times 8 = 8.0$

A	d	Ad	y	Ay^2	I_o	I_g
10.1	0.37	3.7	15.67	2470	–	2470
32.0	16.75	533.0	0.88	18	2730	2755
8.0	33.25	266.0	17.25	2380	–	2380
50.1	16.0	802.7				7605 IN^4

max. $M = 182^{\prime K}$

$$f_b = \frac{182 \times 12 \times 17.75}{7605} = 5.12 \text{ ksi} \quad \text{O.K.}$$

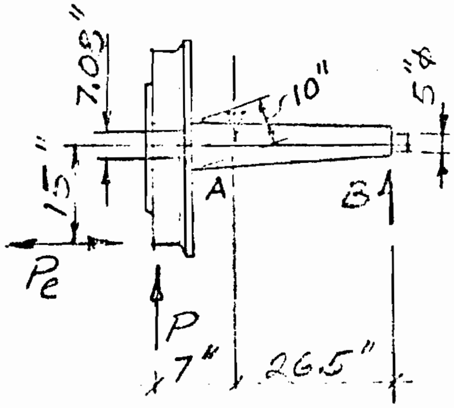
REQ'D. WELDS: max. SHEAR 176^K (p.25)

$I = 7605$, $Q = 8 \times 17.25 = 138\ in^3$

$$v_h = \frac{176 \times 138}{7605} = 3.2\ ^{k/lin\text{-}in}$$

USE $\frac{1}{2}''$ FILLET WELD $\quad P = 0.5 \times 0.707 \times 13 = 4.6 > 3.2^K$ O.K.

DESIGN OF AXLE :



MATERIAL ASTM A273-64
WHEELS AND AXLES AND GATE STRUCTURE SHOULD BE DESIGNED FOR MAX. RADIAL LOAD, ACTING SIMULTANEOUSLY WITH AN ASSUMED SIDE THRUST AT THE WHEEL TREAD. LATERAL $P_e = \frac{1}{3}P$.
(REF. MANUAL EM 1110-2-2701 VERT. LIFT CREST GATES) MORE CONSERVATIVE THAN BEAM DESIGN $P_e = 1/5\,P$ (SEE PAGE-22).
MAX. WHL. LOAD $\cong 205^k$

$$\text{MAX } M_A = 7P + 15P_e =$$
$$= 7(205) + 15\left(\frac{205}{3}\right) = 2455^{\text{IK}}$$

SHEAR : CHECK 10" ϕ AXLE — @ END GIRDER — PT. A

SHEAR: $V = 297^k$ p.22 $\quad v = \dfrac{297}{0.785 \times 10^2} = 3.77^{ksi}$

$$f_b = \frac{2455}{0.098 \times 10^3} = 25.0^{ksi} < 32.5^{ksi}$$

$$f_a = \frac{P}{A} = \frac{1/3\,(205)}{0.785 \times 8^2} = 1.36^{ksi}$$

$\leftarrow$ averg. dia

$$\ell = 26.5 \qquad r_{ave.} = \frac{r_1 + r_2}{2 \times 2} = \frac{5+3}{2 \times 2} = 2$$

$$\frac{\ell}{r} = \frac{26.5}{2} = 13.3 \qquad F_a = 17.1 \times 1.33 = 22.7^{ksi}$$ (EM-1110-1-2[illegible] APPX. I

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad \frac{1.36}{22.7} + \frac{25}{32.5} = 0.06 + 0.77 = 0.83 < 1.0$$

axle design Cont'd.

Axle dia @ Whl. = 7.08"

Allowable stress

$V = \frac{205}{.785 \times 7.08^2} = 5.2^{ksi} < 6.0 \times 1.33 = 8.0^{ksi}$

Axle dia. @ diaphr. pt. A = 5"ϕ

$V_b = 95^k$ (P. 22)

$V = \frac{95}{.785 \times 5^2} = 4.9\ ksi$

Check threaded end at diaphragm

$\frac{205}{3} = 68.5^k$

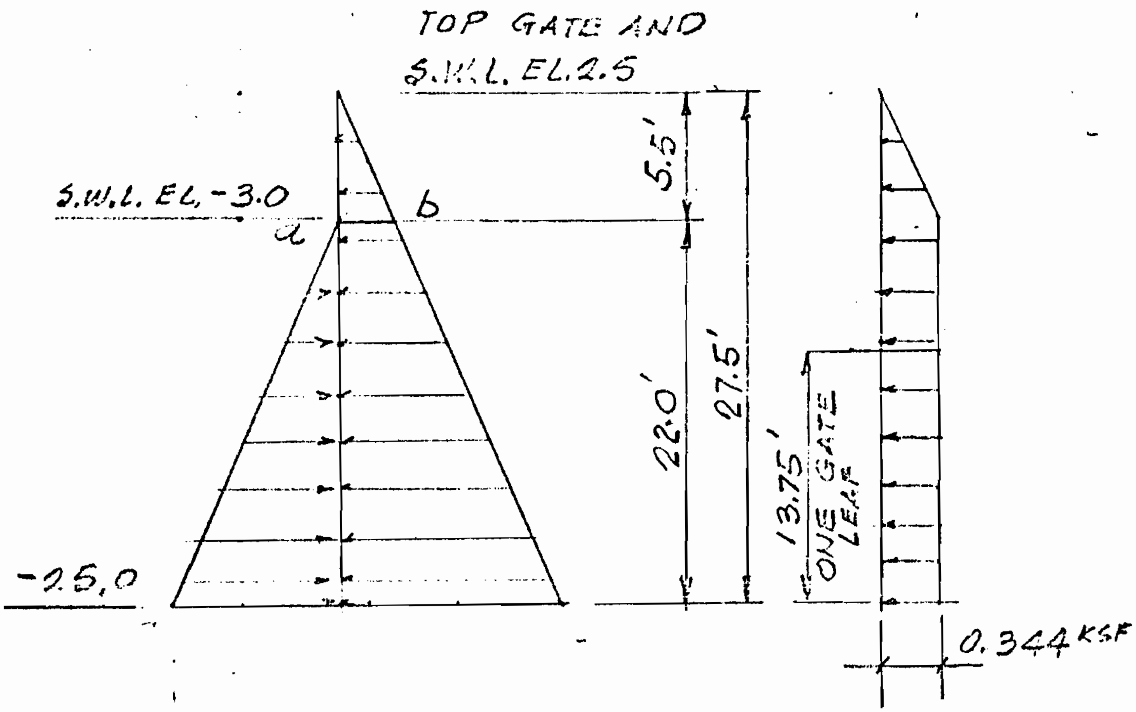
$d_o = \sqrt{\frac{4P}{S_t}} = \sqrt{\frac{4 \times 68{,}500}{10{,}000 \times 1.33}} = 4.5" < 4.675"$ provided

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT [illegible] [illegible] DIAPHRAGM	PROJECT CHEF [illegible]
		FILE NO. [illegible]
	COMPUTED H.G. CHECKED [illegible]	DATE [illegible] PAGE [illegible] OF ___ PAGES

HYDROSTATIC PRESSURE CASE-8
(MAX. HEAD UNDER WHICH GATES OPERATE)

GULF SIDE



NET PRESSURE

AT a-b

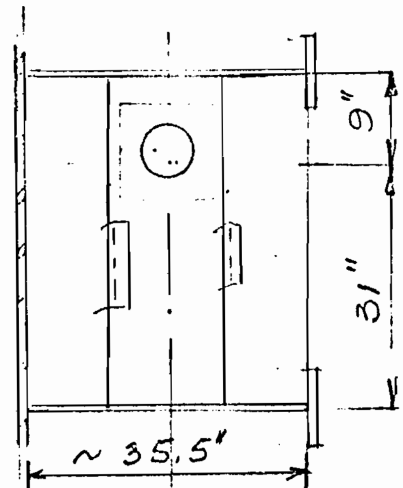
$$q = 0.0624 \times 5.5 = 0.344^{KSF}$$

TOTAL PRESSURE ON BOTT. GATE :

$$W = 0.344 \times 13.75 \times 46.33 \doteq 220^{K}$$

WHL. LOAD:
$$P = 220/4 = 55^{K}$$

DIAPHR. SUPPORTING AXLE END.
(LATERAL THRUST USED FROM OPER. CASE-8)

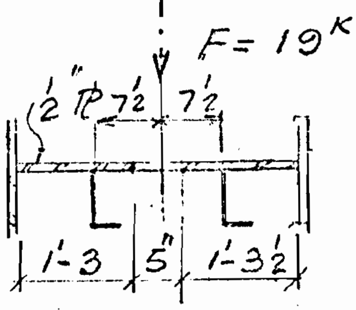


AXLE THRUST:

$$F = \frac{55}{3} = 18.3^k \text{ SAY } 19^k$$

THIS FORCE TO BE TAKEN BY REINF'G L^s COMBINED WITH ½" PL

BENDING & REACTIONS

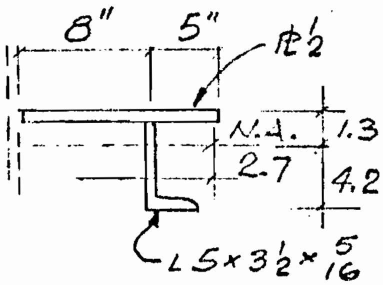


$$\text{max. } R = V = \frac{19}{2} \times \frac{31}{40} = 7.5^K$$

$$\text{max. } M = 7.5 \times 9 = 67.5^{\prime K}$$

$$S_{req} = \frac{67.5}{18} \cong 3.75 \text{ IN}^3$$

CHECK PROJECTIONS (½" PL + L5×3½×5/16)



$10.0/0.5 = 20 > 32(0.5) = 16"$ USE $0.5 \times 16 = 8"$

$4.5/0.5 = 9 < 15.8$ O.K. (PL)

$3.5/0.31 = 11.2 < 12.6$ O.K. (OUTST. L-LEG)

	A	d	Ad	y	Ay^2	I_o	I_g
PL	6.5	0.25	~1.6	1.05	7.2	—	7.2
	2.56	3.90	10	2.6	17.3	6.6	23.9
	9.06	1.3	11.6				31.1 IN^4

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AXLE SU[illegible] CONT'D.:

$$f_{b(max.)} = \frac{67.5 \times 4.3}{.31.1} = 9.1^{ksi} < 18^{ksi}$$

(LOW STRESS ALLOWS FOR POSSIBLE MINOR CHANGE OF AXLE LOCATION)

BOTTOM SECTION

WEIGHT AND CENTER OF GRAVITY

ITEM	No	WT IN FT.	LENGTH FT.	TOTAL WT. Lbs.	ARM IN.	MOMENT IN-lbs
SKIN PL						
3/4 × 165" × 46.6'	1	420	46.6	19,600	0.375	7,350
GIRDER WEB PL						
3/8" × 52" × 20'	4	66.3	80.0	5,300	26.75"	141,500
3/8 × 44" × 10.8	8	56.1	86.4	4,840	22.75"	110,000
1/2 × 33 1/2 × 2.58'	8	57.0	20.6	1,170	17.5	20,500
Flge. PL. (GIRD.)						
16" × 7/8 × 12.70	4	47.6	50.8	2425	44.19	107,000
16" × 7/8 × 20'	2	47.6	40.0	1910	53.19	102,000
16" × 1" × 20'	2	54.4	40.0	2170	53.25	115,500
16" × 1" × 12.70	4	54.4	50.8	2770	44.25	122,500
END PL						
1" × 32" × 13.75'	2	109	27.5	3000	16.75	50,200
Flge. 8" × 1" × 13.75	2	27.2	27.5	747	32.75	24,500
DIAPH. PLs						
3/8 × 52 × 13.4	3	66.3	40	2650	26.75	71,000
1/2 × 35.4" × 13.4	2	60.0	26.8	1600	18.45	29,500
L 6 × 4 × 1/2 × 46.7	2	16.2	93.4	1510	4.76	7130
DIAPH. FLGE PL						
6 × 1/2 × 1.5'	15	10.2	22.5	230	53.0	12,200
6 × 1/2 × 4.0'	10	10.2	40.0	410	34.0	13,900
				50,330		934,830

HARZA ENGINEERING COMPANY CHICAGO

SUBJECT Control Structure Wheel Gate

COMPUTED H.B. CHECKED [illegible]

PROJECT [illegible]

FILE No. 455-181

DATE [illegible] PAGE 33 OF ___ PAGES

CONT'D. WT. AND CENTER OF GRAVITY

ITEM	NO	WT Lbs/FT	LENGTH FT	TOTAL Lbs.	ARM IN	MOMENT IN-lbs
FROM SHT. #				50,330		934,830
STIFF. L 5×3½×5/16	8	8.7	26.6	230	18.75	4350
WEB STIFF. PL						
4"× 3/8 × 4.33'	16	5.1	69.2	354	26.75	9460
4 × 3/8 × 3.75'	16	5.1	60.0	306	23.25	7100
BEAM BRACES						
L 3×3×3/8 × 5.1'	8	7.2	40.7	294	26.3	7730
AXLE SUPPT. PL'S						
18"× 1/2 × 1.5'	8	30.6	12	367	18.75	6880
13 × 3/8 × 1.1'	8	16.6	8.8	147	18.75	2760
WHEELS + AXLES ~1500 EACH	4	Each 1500		6000	18.75	112,500
LIFTG. LUGS	2	Each 670		1340	18.75	25000
SEALS: SIDES, BOT.				330	+1.25	410
BOLTS				190	-0.25 avg.	- 50
CLAMP BARS						
2½ × ½ SIDES		4.25	27.5	117	-2.25	-260.
3 × 5/8 bottom		6.38	46.0	290	2.06	600
SEAL BAR 3"×1		10.2	27.5	281	-0.5	-140
				60,576		1,111,170

ADDING PAINT E.T.C.
TOTAL DRY WT.
SAY 60.0K

$$x = \frac{1{,}111{,}170}{60{,}576} = 18.30'' \text{ Say } 1'\text{-}6\tfrac{1}{4}''$$

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HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCT [illegible] WHEEL GATE - TRA[illegible]	PROJECT CHEF [illegible]
		FILE NO. [illegible]
	COMPUTED H.[illegible] CHECKED [illegible]	DATE [illegible] PAGE 34 OF ____ PAGES

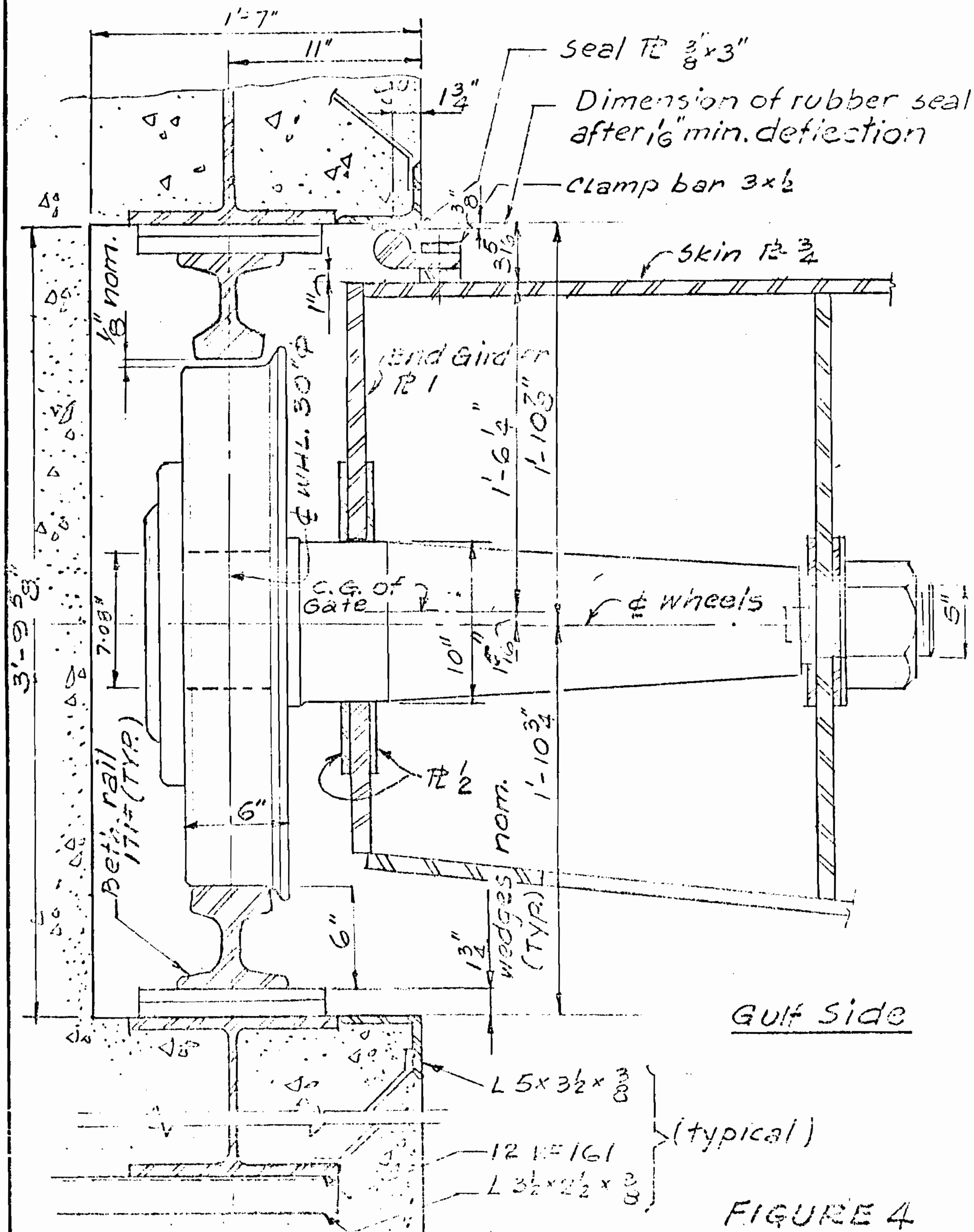


FIGURE 4

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WHEELS	PROJECT CHIEF MENTEL
	COMPUTED HMM CHECKED M.T.C.	FILE NO. 453 B1
		DATE JUNE 27 PAGE 35 OF PAGES

WHEELS

Max.^m Load/Wheel = 205^k f...

BETHLEHEM 171# RAIL CLASS 'C'

$P = 1600 WD$; $D = 30''$, $W = 3.5''$

$P = 1600 \times 30 \times 3.5 = 168000 \text{ lbs} = \underline{168}^k < \frac{205}{1.33}$ (overstress allowed)

CHECK FOR BRINNELL HARDNESS

We have $\frac{P}{W.D} = \frac{(B.H.N \times 24.5 - 2,200)}{F.S.}$

$$\frac{205000}{30 \times 3.5} = \frac{B.H.N \times 24.5 - 2,200}{3.0}$$

$$B.H.N = \frac{1}{24.5}\left[\frac{205000 \times 3}{105} + 2200\right] = 329$$

USE:

WHEEL $\underline{D = 30'', \text{ WIDTH} = 3\tfrac{1}{2}'', \text{ BHN} = 325}$

"BETHLEHEM" 171# RAILS

WITH BHN = 325

WHEELS (Alternate Design Method)

Design data:

load per wheel 205.0^{k} p.22

Diameter of whl. 30 in

Effect width 3.5 in

Material: Q.Q.-S.-763a, Class 3

Yield point 115.0^{ksi}

Brinell hardness (BHN) 325 (p.35)

Modulus of elasticity 28×10^6 psi

Lateral thrust load $0.2 \times 205 = 41^{k}$

Critical ld. (CL)

$P_{cr} = 24.5(BHN) - 2200 = 24.5 \times 325 - 2200 = 5750$ Lb/in

$CL = P_{cr} \times dia \times width =$

$= 5750 \times 30 \times 3.5 = 600\,000$ Lbs

Factor of safety

$$FS = \frac{600\,000}{205} = 3.0$$

Wheels (contd)

Check by Iowa State formula

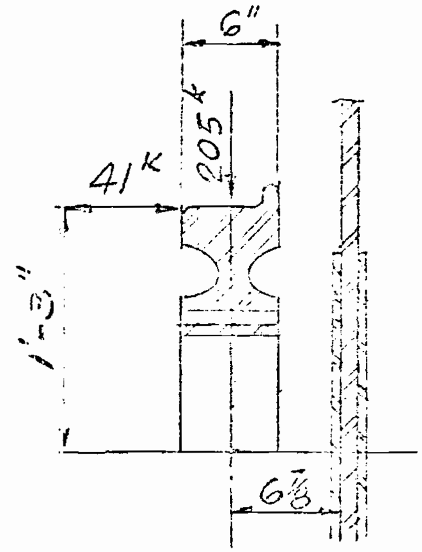
$$P_{cr} = \left[1 + \frac{1.78}{1 + \frac{dia^2}{800\,L^2}}\right] \frac{\pi \times YP^2 \times dia}{E}$$

where D = dia of wheel, inches
L = length of whl, inches
YP = yield point, psi
E = modulus of elast., psi

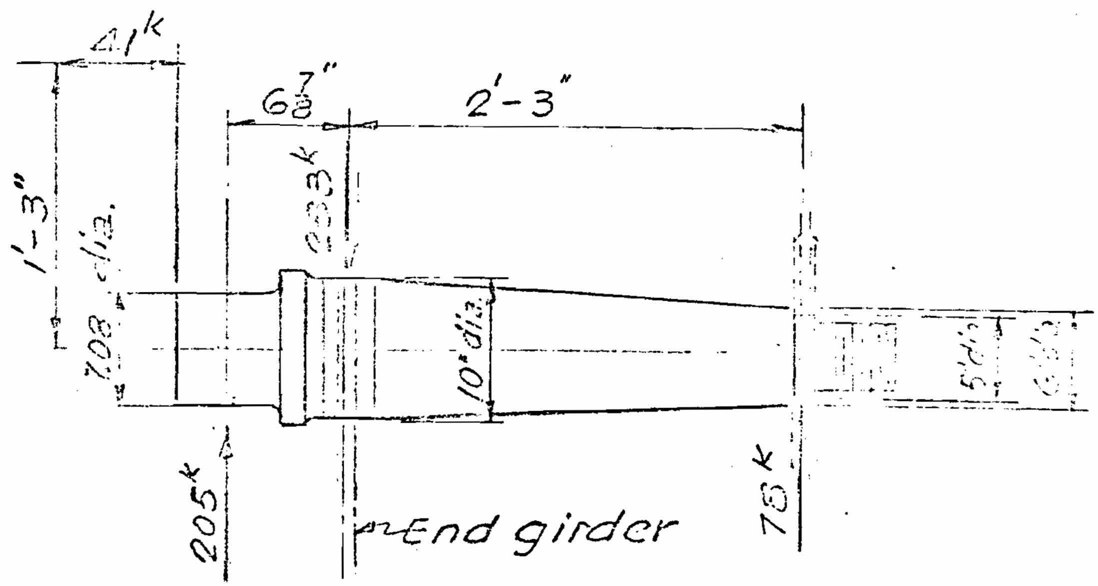
$$P_{cr} = \left[1 + \frac{1.78}{1 + \frac{(30)^2}{800 \times (3.5)^2}}\right] \frac{3.14 \times (115\,000)^2 \times 30}{28\,000\,000} =$$

$= 2.6 \times 44500 = 116000$ lbs

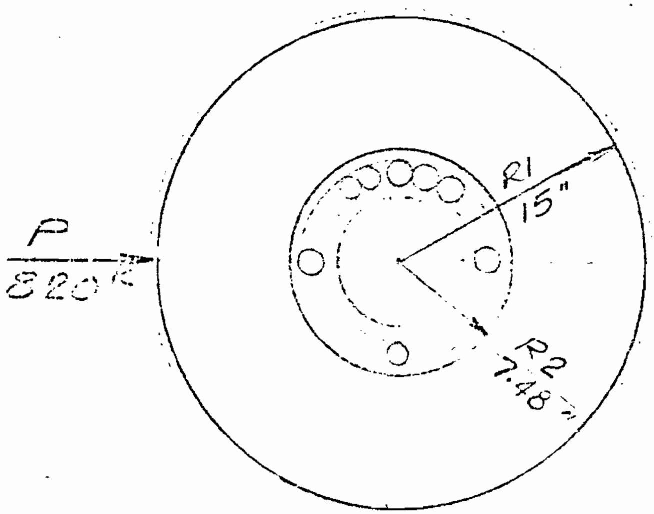
$$F.S = \frac{116000 \times 3.5}{154000} = 2.65$$



WHEEL LOADING



AXLE LOADING



BEARING FRICTION

FIGURE 3

DETAIL DESIGN MEMORANDUM NO.

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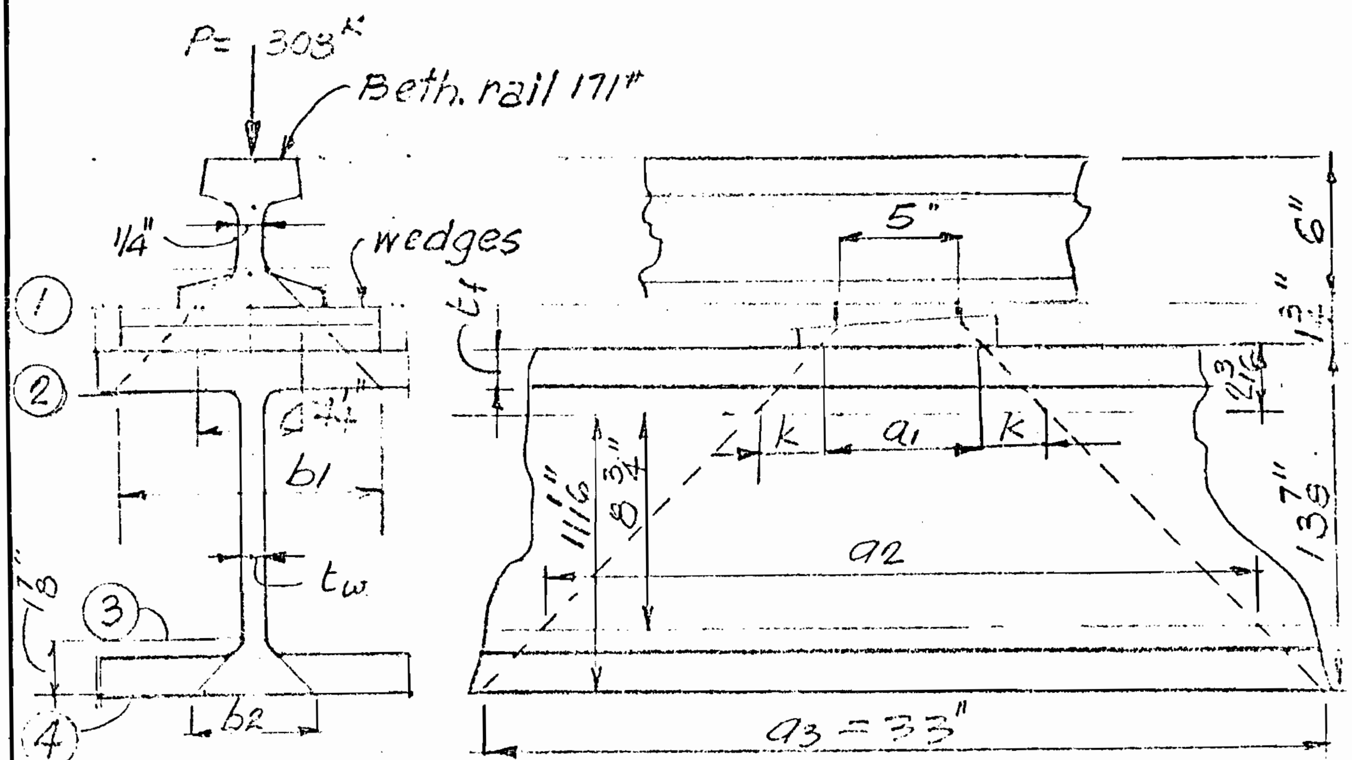
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT [illegible] STRUCTURES [illegible] COMPUTED [illegible] CHECKED [illegible]	PROJECT [illegible] FILE No. 453-31 DATE [illegible] PAGE 39 OF ___ PAGES

EMBEDDED METAL TRACK

Max. wheel load $= 154^k$ (case 6, p.7)

For design of track beam max. whl. load shall be increased by 100% (EM-1110-2-2701), then

design $P = 2 \times 154 = 308^k$



Try 12 WF 161

$b_f = 12\frac{1}{2}''$, $t_f = 1\frac{1}{2}''$

$t_w = \frac{7}{8}''$, $k = 2\frac{3}{16}$

$b_1 = 4\frac{1}{4} + 2(1\frac{3}{4} + 1.5) = 10.75$

$b_2 = \frac{7}{8} + 2 \times 1\frac{7}{8} = 4\frac{5}{8}''$

$a_1 \doteq 6\frac{1}{2}''$, $a_1 + 2k = 10\frac{7}{8}''$

$a_2 = 10\frac{7}{8} + 2 \times 8\frac{3}{4} = 28\frac{3}{8}''$

$a_3 = 10\frac{7}{8} + 2 \times 11\frac{1}{16} = 33''$

Check stresses at critical sections:

(1) Contact Area: $(1\frac{3}{4} + 2 \times 1\frac{1}{2}) \times 5 = 21.2^{\square''}$

avg. $p = \frac{308}{21.2} = 14.6^{ksi}$ O.K.

(2) Assume steel and concr. resist load in proportion to moduli of elasticity

$= \frac{30 \times 10^6}{3 \times 10^6} = 10$

$(10.75 - 0.87)$

$\text{Area} = (\overset{9.5}{6.5 + 2 \times 1.5}) \times (9.88/10 + .87) = 17.6^{\square''}$
equiv. stl. area

avg. $p = \frac{308}{17.6} = 17.5^{ksi}$ (steel) O.K

(3)

$\text{Area} = 28.375 \times .87 = 24.6^{\square''}$

stl. shape $= 9.5 \times 0.87$

avg. $p = \frac{308}{24.6} \times \frac{8.3}{17.6} = 5.90^{ksi}$ O.K.

(4) $\text{Area} = 33 \times 4.625 \cong 152^{\square''}$

avg. $p = \frac{308}{152} \times \frac{8.3}{17.6} = 0.955^{ksi}$ O.K.

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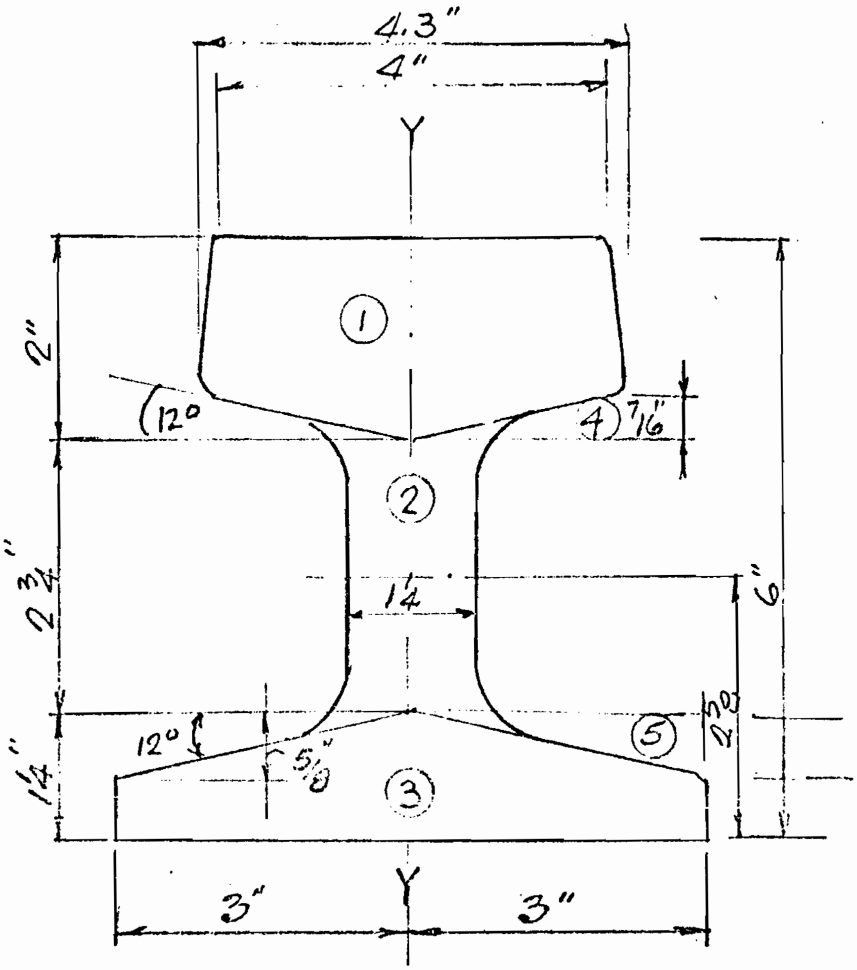
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTU... GUIDE RAIL	PROJECT CHEF ...
	COMPUTED H.S CHECKED ...	FILE NO 453-B1
		DATE OCT. '52 PAGE 41 OF ___ PAGES



171-lb CRANE RAIL

AREAS:

(1) $= 2 \times 4.15 = 8.3$

(2) $= 1.25 \times 2.75 = 3.42$

(3) $= 1.25 \times 6 = 7.5$

(4) $= \frac{2.08}{2} \times .437 = 0.455$

(5) $= -\frac{3.0}{2} \times .625 = -0.94$

MOM. OF INERTIA ABT. Y-Y AXIS

(1) $1/12 \times 2 \times 4.15^3 = 11.9$

(2) $1/12 \times 2.75 \times 1.25^3 = 0.45$

(3) $1/12 \times 1.25 \times 6^3 = 22.5$

34.85
9.37
25.48 IN^3

(4) $-0.45 \times 1.44^2 \times 2 = -1.87$

(5) $-0.94 \times 2^2 \times 2 = -7.50$

$S_T = \frac{25.48}{2.15} = 11.85 \ IN^3$

$S_b = \frac{25.48}{3} = 8.50 \ IN^3$

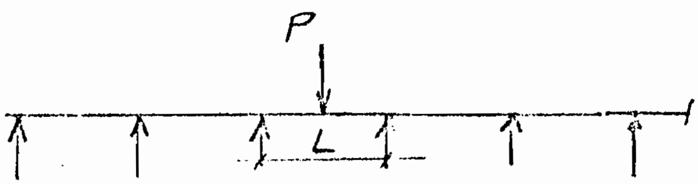
HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE GUIDE RAIL	PROJECT CHEF [illegible]
	COMPUTED H. S. CHECKED [illegible]	FILE NO. 453-B1
		DATE [illegible] PAGE 12 OF ____ PAGES

CHECK RAIL BETH. 171-LB

PROPERTIES: $A = 16.8$; $S_{Hd} = 245\ IN^3$, $S_{bBE} = 244$

max WHL. $P = 154^k$ (FOR NORMAL STRESS, SHT. 8)

MAX. CLIP SPACING:



REF. AISC HANDBOOK FOR "MOMENTS, SHEARS & REACTIONS". FOR CONT. HIGHWAY BRIDGES:

LOADING CONDITION

MAX. MOMENT OCCURS @ MIDSPAN:

FROM TABLE 45; $M/PL = 0.173$

$fS = 0.173\ PL$; $18 \times 24.4 = 0.173 \times 154 L$

$26.6 L = 438$; $L = 16.5''$

SINCE IN THIS CASE THE GATE WILL NOT BE LIFTED, RAIL SUPPORTING CLIPS WILL BE LOCATED JUST UNDER THE WHEELS TO AVOID BENDING, THE RAIL WILL BE CHECKED FOR OPERATING CONDITION, CASE-8.

SPAN BETW. C/C WHEELS = 8'-6" = 102"

USING 6 EQUAL SPACES $L = 102/6 = 17''$

CHECK RAIL FOR THIS SPAN; SEE NEXT SHEET.

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE GUIDE RAIL COMPUTED H.S CHECKED [illegible]	PROJECT [illegible] FILE No. [illegible]-81 DATE [illegible] PAGE 43 OF ___ PAGES

MAX. WHEEL LOAD = 55.0^K (SHT. 29)

HORIZONTAL THRUST $H = 55/3 = 18.4K$

$M_x = 0.173 \times 55 \times 17 = 161.5''^K$

$M_y = 0.173 \times 18.4 \times 17 = 54.2''^K$

$$\max f = \frac{M_x}{S_x} + \frac{M_y}{S_y} = \frac{161}{24.4} + \frac{54.2}{8.5} = 6.62 + 6.38 =$$

$$= 13.60^{KSI} < 18.0$$

USING 5 SPACES: $L = \frac{102}{5} = 20.4''$

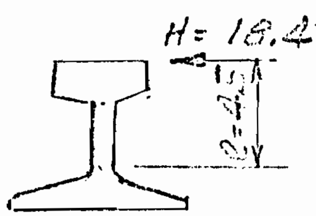
$M_x = 161 \times \frac{20.4}{17} = 193.5''K$

$M_y = 54.2 \times 1.2 = 65''^K$

$$f_{max} = \frac{193.5}{24.4} + \frac{65}{8.5} = 7.92 + 7.65 = 15.57 < 18.0$$

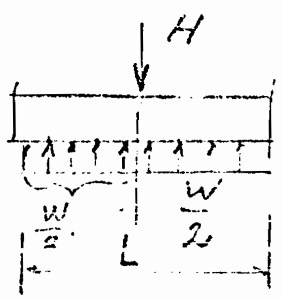
O.K.

CHECK RAIL WEB LATERALLY:



$M = 18.4 \times 4.5 \doteq 83''^K$

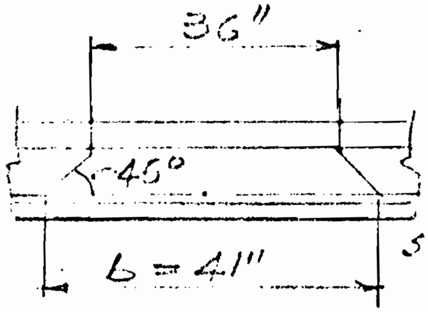
Top Flge. $S = \frac{11.8 - 1.87}{2.15} \doteq 4.6\ IN^3$



ASSUME EFFECT. FLGE. LENGTH TO TRANSFER LOAD TO THE WEB AS FOLLOWS:

$M = fS = W/2 \times L/4 = 18 \times 4.6 \times 9.2\ L/4\ ;\ L = 36''$

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCT. GUIDE RAIL	PROJECT [illegible]
	COMPUTED H.S. CHECKED [illegible]	FILE No. [illegible]-B1
		DATE June 7 PAGE 11 OF ___ PAGES



EFFECT. LENGTH AT max. M

$b = 36 + 2 \times 2\frac{1}{2} = 41''$

STEM $t \cong 1\frac{1}{4}''$

$M = 83^{''K}$; $P = 55^{K}$; FOR "P" ASSUME $b = 2 \times 4.5 = 9''$

$$f_a = \frac{55}{9 \times 1.25} = 4.90^{ksi}$$

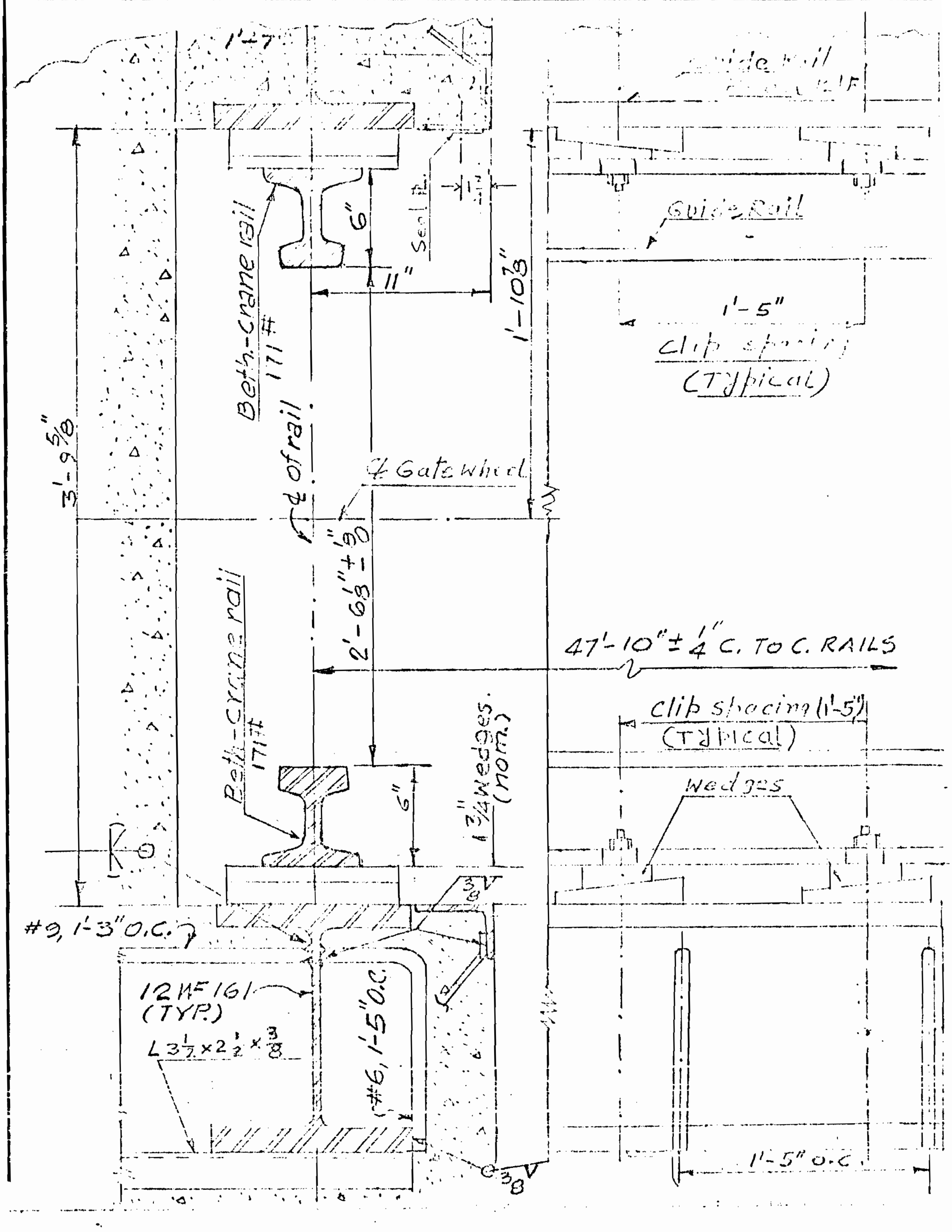
$$f_b = \frac{83 \times 6}{41 \times 1.25^2} = 7.8^{ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{4.9}{22.5} + \frac{7.8}{18} = 0.22 + 0.43 = 0.65 < 1.0$$

O.K.

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE WHEEL GATE-TRACK DET.	PROJECT CHEF MENTEUR
	COMPUTED A.D.M. CHECKED	FILE NO. 453 F1
		DATE Mar '71 PAGE 45 OF PAGES



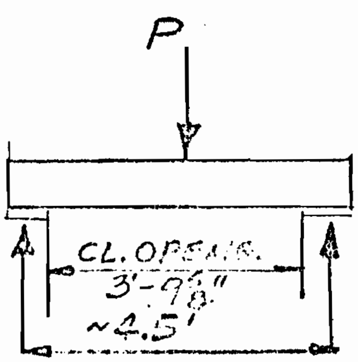
DOGGING DEVICE: STEEL ASTM A36

ALLOW. BASIC WORKING STRESS 18.0^{ksi}
OVERSTRESS 24.0^{ksi}

DOGGING BEAM WILL BE DESIGNED TO SATISFY THE FOLLOWING CONDITIONS:

1. FOR STORAGE OF (ONE) SECTION USE ACTUAL LOAD + 100% IMPACT FOR NORMAL STRESS 18.0^{ksi}

2. DURING PROCESS OF DOGGING USE WT. OF 2 SECTIONS + 50% IMPACT FOR ALLOW. STRESS OF 24.0^{ksi} (SHORT DURATION)



GATE WT. = 62.0^{k} p. 33
USE 65^{k}

FOR CASE-1

$$P = 65/2 + 100\% = 65^{K}$$

$$M = 65 \times 4.5/4 = 73.2^{IK}$$

$$S_{req'd} = \frac{73.2 \times 12}{18} = 48.8\ in^3$$

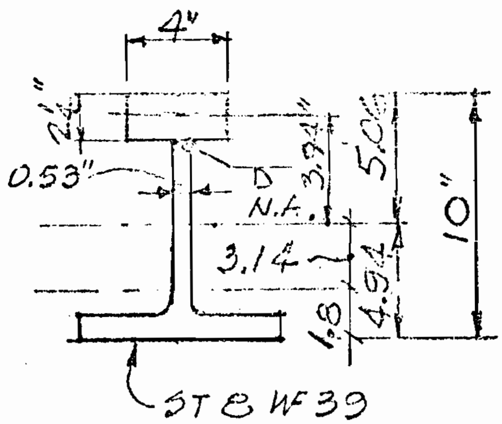
CASE - 2

$$P = \frac{65+65}{2} \times 1.5 = 97.5^K$$

$$R = 97.5/2 = 48.8^K$$

$$M_{max} = 97.5 \times {}^{4.5}/_{4} = 109.0^{'K}$$

SINCE THE HEIGHT OF THE BEAM IS RESTRICT TO 10"(±), A BUILD UP SHAPE WILL BE DEVELOPED



AREAS:

FLANGE = 4×2¼ = 9□"

ST 8 = 11.5□"

	AREA	y	Ay	d	Ad²	Io	I
Flge.	9.0	1.12	10.0	3.94	140	3.8	143.8
ST 8	11.5	8.2	94	3.14	113	60	173.0
	20.5	5.06	104				316.8

$$S_T = \frac{316.8}{5.06} = 62.5 \text{ in}^3$$

$$S_b = \frac{316.8}{4.94} = 64 \text{ in}^3$$

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT [illegible] DEVICE COMPUTED H.S. CHECKED [illegible]	PROJECT [illegible] FILE NO. 453-[illegible] DATE [illegible] PAGE [illegible] OF ___ PAGES

STRESSES:

CASE-1 $f_{b(max.)} = \frac{73.2 \times 12}{62.5} = 14.0^{ksi} < 19.0$

O.K.

CASE-2 $f_{b(max)} = \frac{100 \times 12}{62.5} = 20.9 < 24.0$ O.K.

SHEAR:

CASE-1

$max.\ V = 65/2 = 32.5^{K}$

$v = \frac{32.5}{10 \times 0.53} = 6.2^{ksi} < 12.0$

CASE-2 $v = \frac{49}{10 \times 0.53} = 9.3^{ksi} < 16.0$

WELD REQUIREMENTS:

$V = 49^k$ (case 2) $\quad I = 316.8 \text{ in}^4$

$$Q = 9 \times 3.94 = 35.4 \text{ in}^3$$

$$v_h = \frac{VQ}{I} = \frac{49 \times 35.4}{316.8} = 5.46^{k/lin.-in.}$$

Req'd min. weld size = $\frac{3}{8}''$ FILLET

CAPACITY $F = 2 \times 17.5 \times .375 \times .707 = 9.275.46^{k/i}$

HOWEVER USE FULL PENETRATION —WELD

BEARING ON CONCRETE:

USING ALLOWABLE BEARING $f_p = 0.75^{ksi}$

$$\text{REQ'D } A = \frac{32.5}{0.75} = 43.3^{\square''}$$

<u>USE 6"×1"×12"</u> BEARING $\mathbb{P}$ $A = 72^{\square''}$

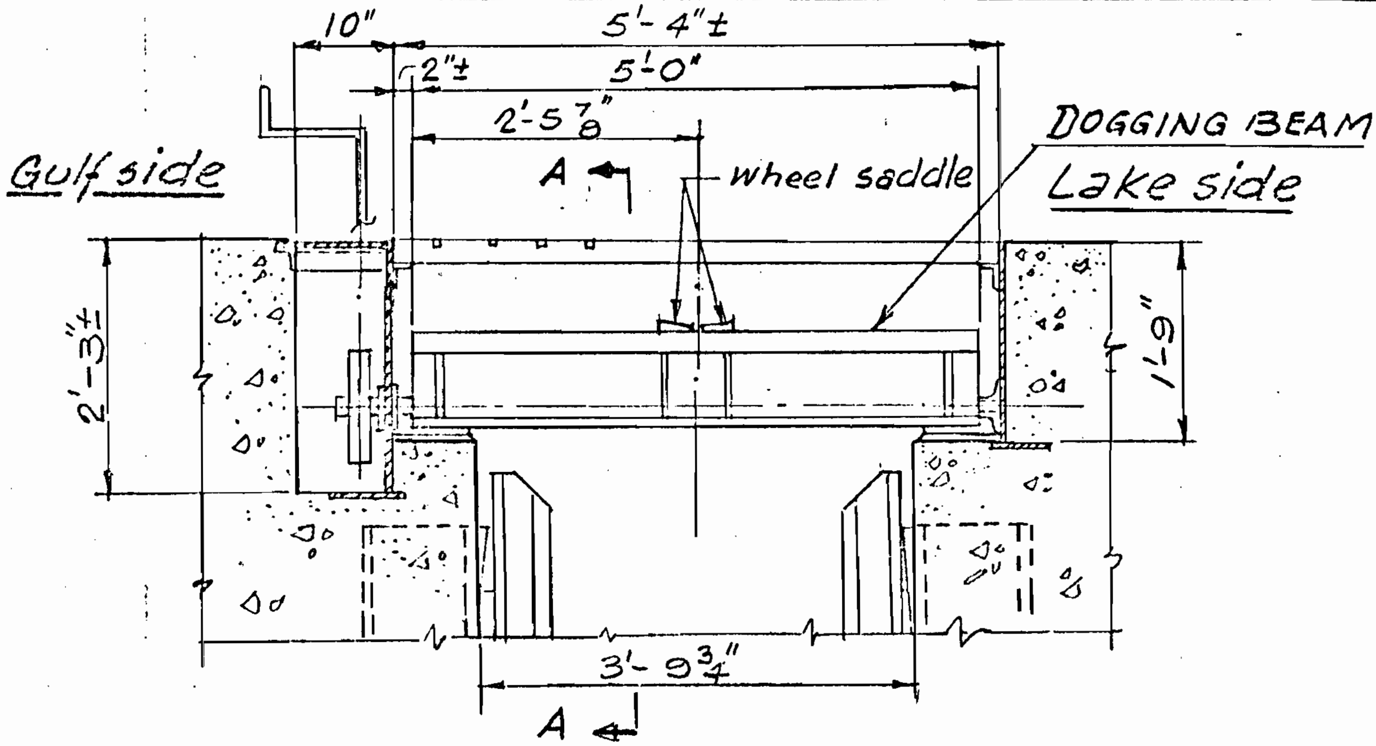
FOR CASE-1

$$f_p = \frac{32.5}{72} = 0.45^{ksi} < 0.75$$

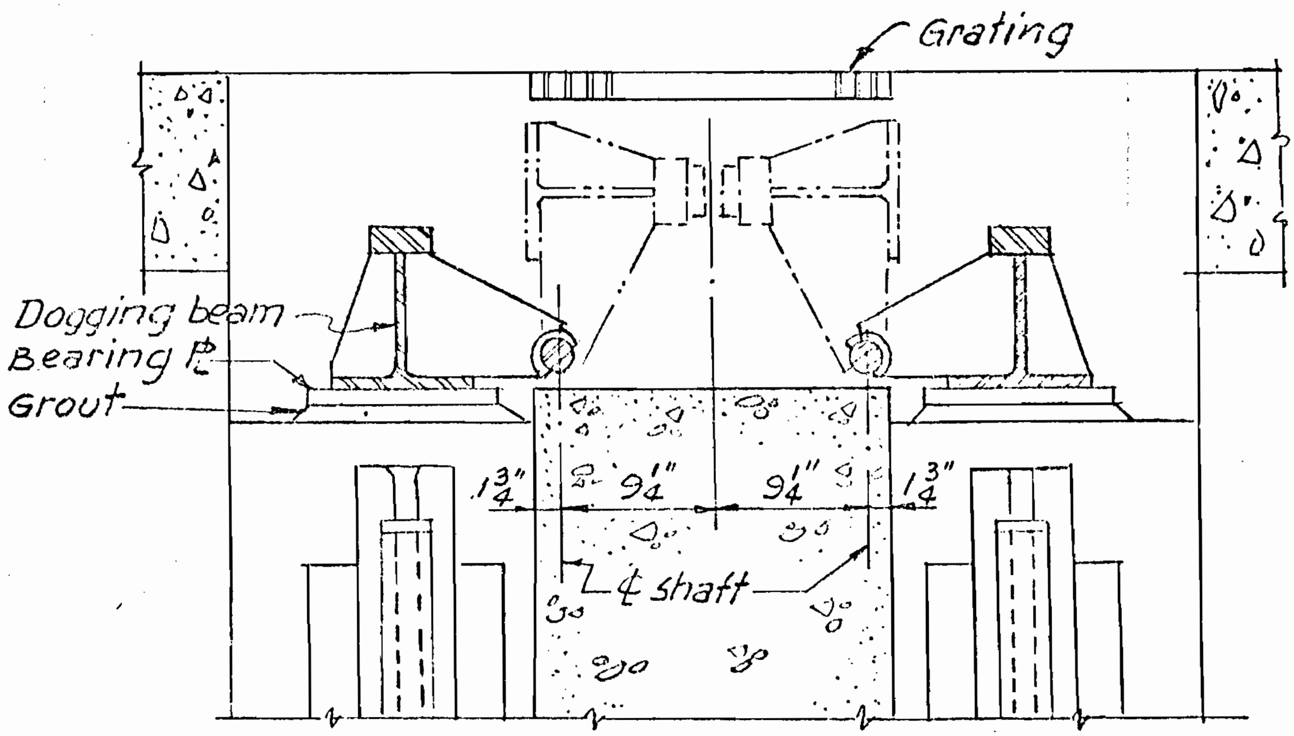
FOR CASE-2 $\quad f_p = \frac{49}{72} = 0.68^{ksi} < 1.0$

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE DOGGING DEVICE COMPUTED H.S CHECKED M.P.W.	PROJECT CHEF MENTEUR FILE NO. 453-B1 DATE July 71 PAGE 50 OF ___ PAGES



DOGGING DEVICE - SECTION THRU PIER



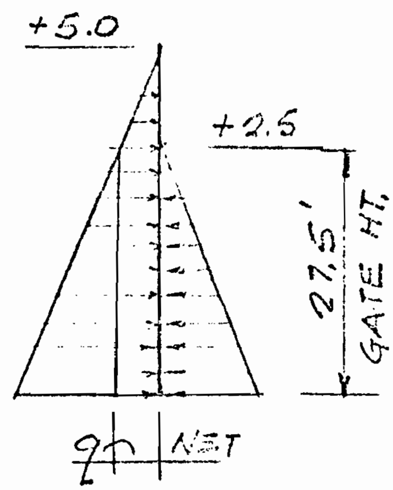
SECTION A-A

FIGURE 6

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT CONTROL STRUCTURE HOIST CAPACITY	PROJECT CHEF MENTEUR
	COMPUTED [illegible] CHECKED [illegible]	FILE NO. 453-131
		DATE [illegible] PAGE 51 OF ___ PAGES

HOIST LOAD

CASE-3 (GOVERNING CASE FOR GATE OPERATION)



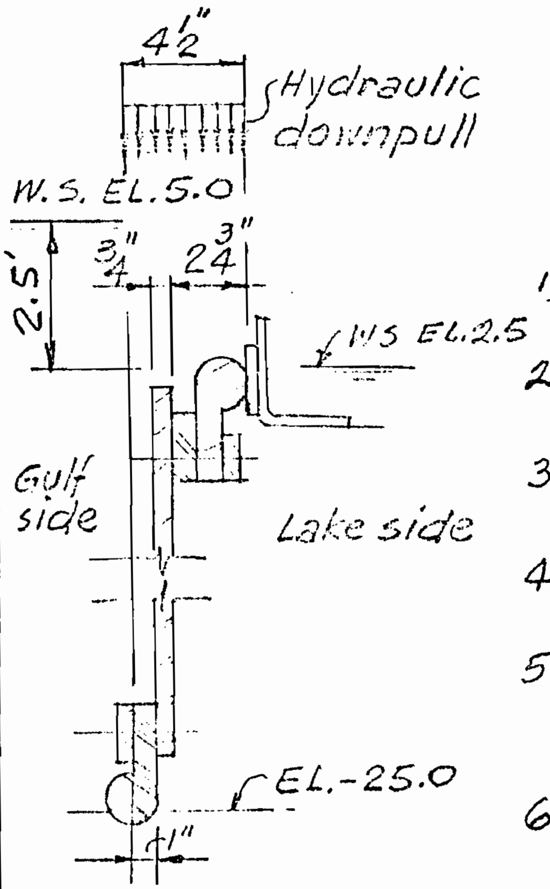
EFFECTIVE HEAD = 2.5'

$q = 2.5 \times 62.4 = 156$ psf

TOTAL WATER PRESSURE ON ONE gate leaf:

$W = .156 \times 13.75 \times 46.33 \cong 100^{K}$

WHL. LOAD $= 100/4 = 25^{K}$



THE FORCES RESISTING TO OPEN THE GATE WILL CONSIST OF:

1) Submerged wt. of gate

2. WHL. BEARING FRICTION

3. WHL. ROLLING FRICTION

4 SIDE & TOP SEAL FRICTION

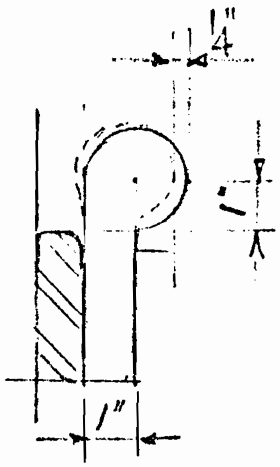
5 WATER LOAD ON TOP & BOT. SEALS (Hydraulic downpull)

6. WEIGHT OF LIFTING DEVICE

GATE WT. DRY — 62.0^k USE 65^k

Submerged wt. $\frac{490-62.5}{490} \times 65 = 56.5^k$

2 LEAFS $W = 56.5 \times 2 = 113.0^k$



ASSUME THE SEAL IS SET FOR $\frac{1}{4}''$ NOMINAL DEFLECTION.

FORCE TO DEFLECT $\frac{1}{4}'' \cong 11^{\#}/IN$

(FROM CURVES, FIG. 27 BY "GATE RUBBER CO." CATALOG)

COEFFICIENTS:

RUBBER SEAL FRICTION COEFF. = 1.0

u_1 = COEFF. OF FRICTION FOR WHL. BEARINGS (SPHERICAL ROLLER BRGS. W/GREASE = 0.005)

FRICTION FORCE = LOAD × COEFF. CONSIDERED TO ACT AT PITCH LINE WHICH IS $\frac{O.D.+Bore}{2} = \frac{A+B}{2}$

u_2 - COEFF. OF ROLLING FRICTION = 0.01 IN

ASSUME SEAL CONTACT WIDTH = $1\frac{1}{2}'' = 0.125'$

SAY O.K.

HOIST PULL

GATE WT (WET) 2×56.5 113.0^k

WHL. BEARING FRICTION :

$$(100 \times 2)\frac{10+15.6}{2 \times 2} \times \frac{1}{15} \times 0.005 = 0.43$$

TOTAL LOAD ON TWO LEAFS

ROLLING FRICTION $W\mu_2/R_2 = 200 \times \frac{0.01}{15} = 0.13$

SIDE SEAL FRICTION $2 \times 27.5 \times 0.156 \times 0.125^*$ 1.07

TOP SEAL FRICT. $46.3 \times 0.156 \times 0.125 =$ 0.90

SEAL DEFLECTION FORCE :

$$[(2 \times 27.5) + 46.3] \times 12 \times 11/1000 = 13.3$$

WATER LOAD ON SEALS :

$0.37' \times 46.3 \times .156$ = 2.68

131.51^K

SAY 132.0^K

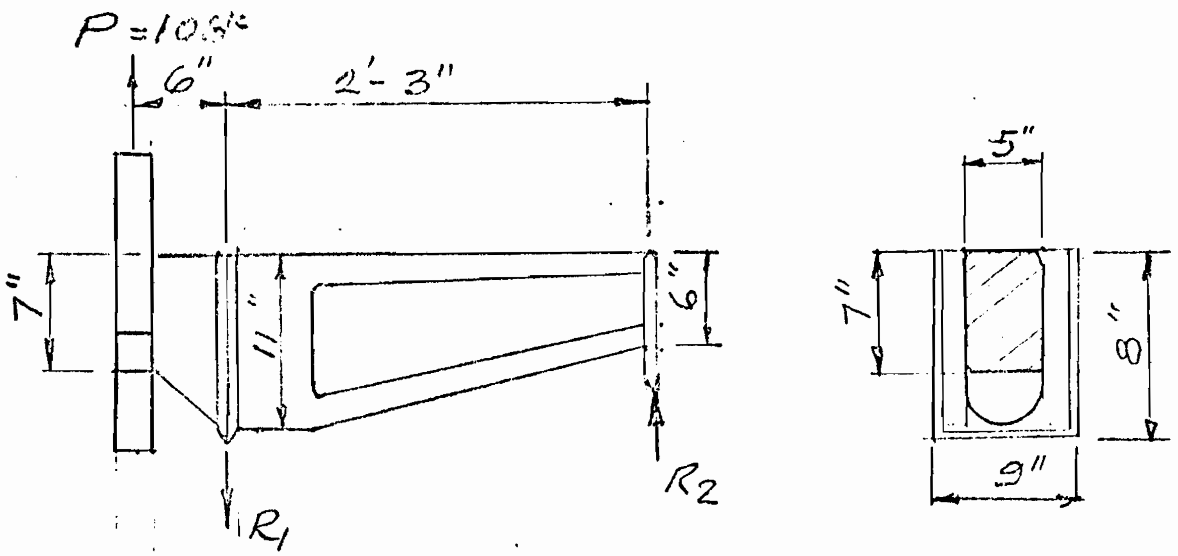
WEIGHT OF LIFTING DEVICE 9.0

141.0^K

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT Control Structure Lifting Hook COMPUTED H.S. CHECKED [illegible]	PROJECT Chef Menteur FILE No. 453-B1 DATE July 71 PAGE 54 OF

LIFTING HOOK



$P = 70.5 = 141^K/2$ (LOAD ON ONE HOOK p.53)

$\underline{35.5} = 50\%$ IMPACT

106^K USE 100^K

$R_1 = 106 \times 2.75/2.25 = 130^K,\ R_2 = 130 - 106 = 24\,K$

$\max M = 106 \times 6'' = 636^{''K}$

$f_b = \dfrac{636 \times 6}{10^2 \times 5} = 7.6^{ksi}$ O.K

SHEAR @ R_1

$f_v = \dfrac{106}{10 \times 5} = 2.1^{ksi}$ O.K.

AT R_2

$f_v = \dfrac{24}{6 \times 1.5} = 2.65^{ksi}$ O.K.