3.2.2. New Orleans to Venice

3.2.2.1. General Description

The project is located along the east bank of the Mississippi River from Phoenix, LA, (approximately 28 miles southeast of New Orleans) to Bohemia, LA, and along the west bank of the river from St. Jude, LA (approximately 39 miles southeast of New Orleans), to the vicinity of Venice, LA.

Project Purpose. The project will provide protection from hurricane tidal overflow for 100-year frequency storms. The protected area encompasses approximately 75 percent of the population and 75 percent of the improved lands in the lower Mississippi River delta region.

Project Features. The project consists of the following:

West Bank

St. Jude to City Price - 3 miles of enlarged back levees from St. Jude to City Price

Reach A - 13 miles of enlarged back levees from City Price to Tropical Bend and two 54-in. flap-gated culverts.

Reach B1 – 12 miles of enlarged back levees from Tropical Bend to Fort Jackson and a floodgate at Empire.

Reach B2 – 9 miles of enlarged back levees from Fort Jackson to Venice.

West Bank River Levee (WBRL) – 34 miles of enlarged west bank Mississippi River levees from City Price to Venice.

East Bank

Reach C – 16 miles of enlarged back levees from Phoenix to Bohemia and 10 flap-gated culverts.

Pre-Katrina Conditions.

St. Jude to City Price Pre-Katrina Status

- Construction in this area started in 1993. Before Hurricane Katrina, the one first enlargement levee construction contract was completed in this area. Remaining work in this area consists of a second enlargement levee contract.

Figure 25. Extent of New Orleand to Venice hurricane protection in Plaquemines Parish. The New Orleans to Venice consists of six distinct reaches; Reach C, Reach St. Jude to City Price, Reach A, Reach B1 and Reach B2, and Reach WBRL City Price to Venice.

Reach A Pre-Katrina Status

- Construction in this area started in 1986. Before Hurricane Katrina, all of the second enlargement levee construction contracts and floodwall contracts had been completed. There were a total of 15 construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach B1 Pre-Katrina Status

- Construction in this area started in 1968. Before Hurricane Katrina, all of the required levee enlargement construction contracts and floodwall contracts had been completed. There were a total of 29 construction contracts that were completed in this reach.

Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach B2 Pre-Katrina Status

- Construction in this area started in 1974. Before Hurricane Katrina, all of the fourth enlargement levee construction contracts and floodwall contracts had been completed. There were a total of nine construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach C Pre-Katrina Status

- Construction in this area started in 1972. Before Hurricane Katrina, the third enlargement levee construction contracts had been completed. There were a total of seven construction contracts that were completed in this reach. Remaining work in this area consists of a fourth enlargement levee contract.

West Bank River Levees (WBRL) Pre-Katrina Status

- Construction in this area started in 1989. Before Hurricane Katrina, the first enlargement levee construction contracts and floodwalls had been completed. There was also one a second enlargement levee construction contract that had been completed. There were a total of 16 construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of three second enlargement levee contracts and I-wall cappings.

3.2.2.2. History

On 30 July 1962, the Chief of Engineers submitted a report that recommended improvements along the Mississippi River below New Orleans to prevent damages to the developed areas of St. Bernard and Plaquemines Parishes from hurricane tidal surges and overflow. The plan recommended increasing the heights of existing back levees and modifying existing drainage facilities at four primary reaches: Reach A on the west bank between City Price and Empire (Tropical Bend); Reach B on the west bank between Empire (Tropical Bend) and Venice; Reach C on the east bank between Phoenix and Bohemia; and Reach D on the east bank for about 8 miles between Violet and Verret.

Three months later, the 1962 Flood Control Act (Public Law 87-874) authorized the project for hurricane-flood protection on the Mississippi River Delta at and below New Orleans, LA, in accordance with the recommended plan submitted by the Chief of Engineers. Following the authorization, the hurricane protection project was officially named the New Orleans to Venice, LA, Hurricane Protection Project.

Despite congressional authorization, the project plans were far from finalization. On 5 February 1964, the Plaquemines Parish Commission Council requested the division of Reach B into two separate units: Reach B1, between Empire (Tropical Bend) and Fort Jackson; and Reach

B2, between Fort Jackson and Venice. On 25 March 1964, the Chief of Engineers approved the division of the Reach B, subject to the proviso that the Plaquemines Parish Commission Council pay for a closure levee at Fort Jackson that would be required to complete the independently constructed Reach B1 loop.

A second post-authorization change to the project emanated from dissatisfaction with the project features covering St. Bernard Parish. On 8 May 1964, the House Committee on Public Works adopted a resolution directing a restudy of the hurricane protection for St. Bernard Parish. The restudy, completed on 29 November 1966, recommended the enlargement of the Chalmette Area Plan feature of the Lake Pontchartrain, LA, and Vicinity Hurricane Protection Project, which had been authorized by the 1965 Flood Control Act. The extension of the Chalmette Area Plan to all of the area in St. Bernard Parish, for which hurricane protection could be economically justified, subsequently encompassed the proposed protected area under Reach D of the New Orleans to Venice Hurricane Protection project. The House committee resolution was then closed out with a negative report recommending the deauthorization of Reach D from the project.

Upon the appropriation of funds, construction of the project began in 1966; however, another hurdle remained concerning the treatment of the main river levees that were authorized, constructed, and maintained under the Mississippi River and Tributaries (MR&T) project. Ever since the inception of plans for the New Orleans to Venice Hurricane Protection Project, the Chief of Engineers and the Mississippi River Commission recognized the possible necessity to modify the main river levees to accomplish the level of hurricane protection envisioned. On 29 October 1969, the Corps of Engineers initiated review of the New Orleans to Venice Hurricane Protection Project. As part of this review, two alternate plans were developed for protecting the west bank project areas from hurricane tidal surges from Breton Sound. The first option consisted of raising the west bank river levee to a grade sufficient enough to prevent overtopping by tidal surges from the east. The second option consisted of a barrier levee on the east bank from Bohemia to a point 10 river miles above the Head of Passes, coupled with minor enlargement of the west bank levees from Fort Jackson to Venice. The project review determined that the latter option was both more feasible and economical at that time. Preparation of a design memorandum for the east bank barrier levee was authorized on 2 July 1970.

In February 1985, the Plaquemines Parish Commission Council requested that further work on Reach A be deferred, and that designs for the east barrier levee or the west bank river levee be undertaken. As a result of this request, a restudy of the two alternate plans was conducted. The restudy, in turn, recommended the west bank river levee plan, which necessitated the enlargement of 33.8 miles of river levee from City Price to Venice. The preparation for the design memorandum for the west bank river levee was authorized on 24 July 1986 and later approved in March 1987.

3.2.2.3. Datum - Subsidence and Vertical Datum Problems in New Orleans, LA.

Because of technological gains, the Corps is able to more accurately track subsidence of projects – something that could not be done as reliably in the past. Based on a recent study, we can now estimate that the New Orleans area is subsiding at a rate of 6 to 17 mm/yr or 2 to 5½ ft per century. In New Orleans itself, subsidence is about 3 feet per century. Subsidence is as much as 10 feet per century in Venice.

The IPET, an independent group activated by the Corps to study the response of the hurricane protection system during Hurricane Katrina, identified problems with using the previous vertical datum to which survey benchmarks were referenced. IPET's ability to accelerate analysis of this issue, which was ongoing by the Corps' New Orleans District and the NOAA's NGS, led to the identification of two major problems with elevations in the New Orleans area: subsidence and the use of the old vertical datum elevations as equal to local mean sea level, a common misunderstanding in the engineering community up until the 1990s.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), and structure and levee elevations. It has been known since 1985 that the elevations of benchmarks in and around New Orleans were inaccurate, due to subsidence, and needed to be updated. The exact amount of subsidence was not known until a 2004 survey conducted by the NGS in cooperation with the Louisiana Spatial Reference Center, the Corps and state and local governments was performed on 86 benchmarks in southern Louisiana.

The 2004 survey pointed out inaccuracies due not only to subsidence, but also to distortions and errors in elevations of benchmarks that were assumed to be stable in the past, but had in fact subsided themselves. Based on the 2004 survey, the Corps has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in southern Louisiana. Use of the 2004 survey assures consistency for all elevation surveys performed in the southern Louisiana area.

The IPET has developed a relationship between the current local mean sea level and the 2004 survey, which is referred to as the North American Vertical Datum of 1988 (2004.65 Adjustment). Local mean sea level in the city of New Orleans is about ½ foot above the 2004 datum. The Corps will use the 2004 elevations and their varied relationship to the local mean sea level throughout the area to precisely determine the elevations of levees and other critical flood protective structures. This datum will also be used by the construction industry and others in southern Louisiana for a variety of projects that rely on elevations relative to the local water surface.

More information can be found in the "Geodetic and Water Level Datum" report.

3.2.2.4. Design Hurricane

100-Year Storm

The design hurricane is a hurricane that would produce a 100-year stage. A hurricane of lesser intensity, which would indicate a lower levee grade and an increased frequency, would expose the protected areas to hazards to life and property that would be disastrous in the event of a design hurricane.

The characteristics of the 100-year storm were derived from the SPH parameters. The 100-year storm meteorological parameters differed from the SPH only in wind velocities and CPI. A SPH storm was considered to have a recurrence interval of once in 100 years anywhere within Zone B. The probability of the SPH storm striking a smaller subzone, such as along Reach B2, would be less. The frequency of the SPH at the site of a protective structure was assumed to be dependent upon its exposure and the direction of approach of the storm.

Using observed high water mark and stage data, combined with computed wind tide elevations using different CPIs, a surge frequency curve was constructed representative of a portion of the hurricane protection system. The frequency curve also considered statistics on the critical direction of approach. The frequency of the computed wind tide elevations was adjusted based on the percentage of each direction followed by historic hurricanes. The probabilities of equal stages for both groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions. From this curve, the 100-year stage was identified.

Design Criteria and Assumptions - Functional Design Criteria.

Hydrology and Hydraulics.

The design hurricane characteristics are shown in Table 30; the design tracks are shown in Figures 26–28. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 30
Design Hurricane Characteristics

Location	Track	CPI in.	Radius of Maximum Winds, nautical miles	Forward Speed, knots	Maximum Wind Speed[1], mph	Direction of Approach
Reach A	B	28.0	30	11	85	South
Reach B1	B	28.0	30	11	91	South
Reach B2	B	28.0	30	11	91	South
Reach C	Des H	28.0	30	11	96	Southeast
Mississippi River Levees, Mile 49	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 40	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 25	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 15	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 10	1	27.6	30	11	96	Southeast

[1] Wind speeds represent a 5-minute average 30 ft above ground level.

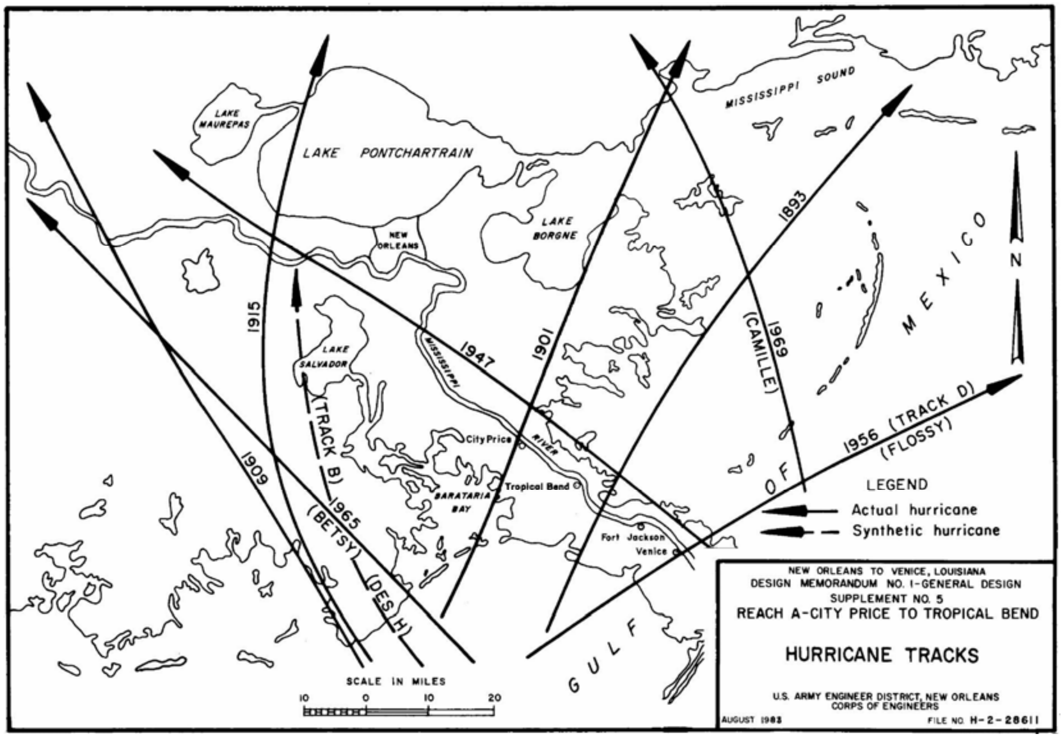


Figure 26. Hurricane tracks, Reach A, B1, and B2.

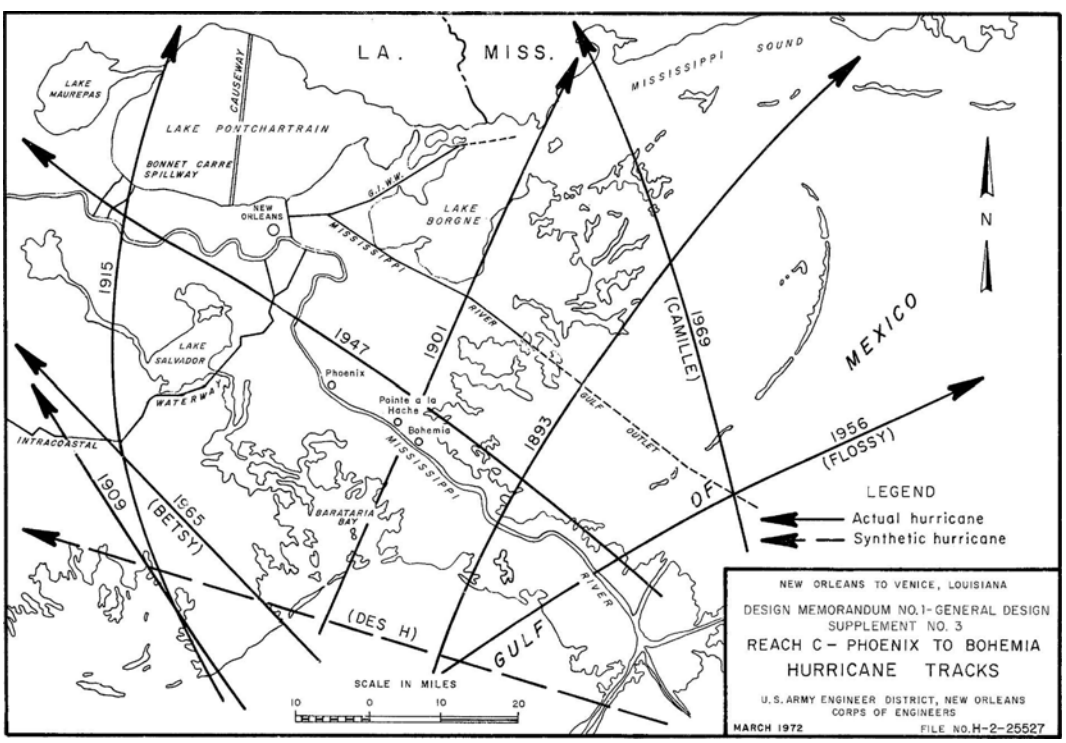


Figure 27. Hurricane tracks, Reach C.

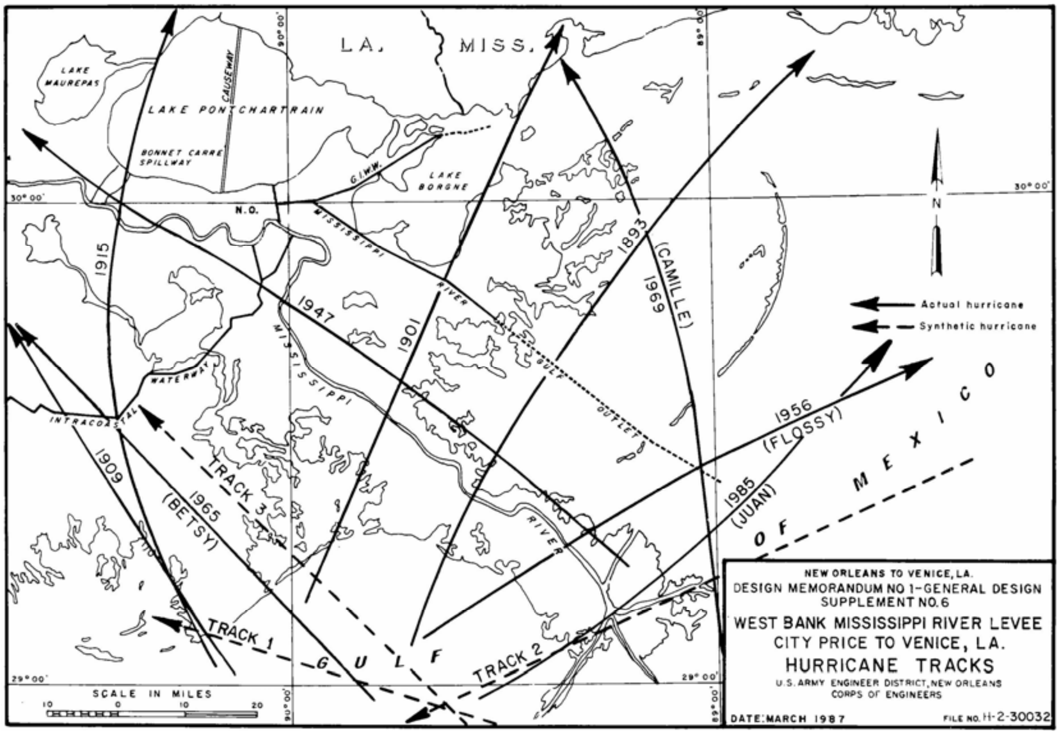


Figure 28. Hurricane track, West Bank Mississippi River Levee.

Surge. Surge elevations were computed using the same methodology as used for IHNC for Orleans East Bank, with an additional step. Surge heights were verified for the September 1915 hurricane and the September 1956 hurricane (Table 31). For Reach B1 and B2, surges were also verified for Hurricane Betsy, in September 1965. Computed surge heights for Hurricane Betsy using the same Z factors averaged about 2.9 ft higher than observed surge heights. This was attributed to the effect of the high forward speed of Hurricane Betsy. A fast moving hurricane does not allow enough time for the surge heights to approach the steady state of water superelevation. For design purposes, Z factors derived from the slow moving hurricanes were used.

Table 31
Verification of Hurricane Surge Heights

Location	Surge Adjustment Factor, Z	September 1915		September 1956	
		Observed ft MSL	Computed ft MSL	Observed ft MSL	Computed ft MSL
Belair	0.52	-	-	5.3	6.2
Phoenix	0.52	-	-	8.5	7.8
Pointe a la Hache	0.52	12.0	12.4	10.3	10.2
Ostrica	0.64	-	-	12.1	12.2
Buras	0.80	7.9	8.7	-	-
Grand Isle	0.80	9.0	8.8	3.9	4.1

Next, the same methodology used to adjust surge heights for the Chalmette Extension was applied. The surge reference line was located approximately 6.3 miles from the upper end of Reach A, near City Price, extending to approximately 2.5 miles from the lower end of Reach A, near Tropical Bend. On the east bank, the surge reference line extended up to approximately 7 miles from Phoenix and to about 2 miles from Davant. Table 32 shows the wind tide level at the surge reference line and at the levee location for Reach A; similar information was not contained in the DM for Reach C.

Table 32
Wind Tide Levels

Location	Wind Tide Level, Surge Reference Line ft NGVD	Wind Tide Level at Levee Location ft NGVD
Segment 1, Reach A, 0+00 – 83+30	11.2	8.9
Segment 2, Reach A, 83+30 – 315+00	10.4	8.2
Segment 3, Reach A, 315+00 – 477+00	10.8	9.6
Segment 4, Reach A, 477+00 – 613+00	11.0	10.1
Segment 5, Reach A, 613+00 – 681+67.45	11.2	10.3

The National Engineering Science Company (NESCO) was contracted to evaluate surge along the Mississippi River. NESCO used a bathystrophic storm surge technique to compute surge at Nairn, LA, a community approximately 15 miles south of Pointe a la Hache. The equations used to determine the behavior of a hurricane surge hydrograph as it propagates upstream were rearranged in terms of discharge. The modified equations are:

$$\frac{d(Q/A)}{dt} = -g\frac{\partial H}{\partial x} - g\frac{Q|Q|}{C_h{}^2 A^2 (H + d_0)} - g(I_0 - I_b)$$

$$B_s \frac{\partial H}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

where

Q = initial steady state discharge in the downstream direction, cubic feet per second

A = the cross-sectional area of the river, square foot

t = the routine time, seconds

g = acceleration due to gravity

H = surge height above the initial water surface profile in the river, feet

x = the distance upstream from some initial point of surge input, feet

C_h = the Chezy coefficient

d_0 = the hydraulic depth for irregular cross sections, feet

I_0 = the slope of the water surface under the initial condition, foot by foot

I_b = the bottom slope of the channel, foot by foot

Bs = the surface width of the river, feet

These equations were rewritten in a form suitable for application of fourth order techniques. The variation of Q and H was evaluated by finite differences and the integrations in time were performed using a fourth order Runge-Kutta method. The initial boundary conditions were:

a. The initial discharge in the river at time = 0 was taken as a constant for all stations (x) along the river.

b. The initial surge height in the river (H_0) was zero at $t = 0$ for all stations.

c. The input surge at $x = 0$ was taken as a prescribed stage hydrograph ($H_0(t)$).

A surge hydrograph at Nairn was generated using isovels from Hurricane Betsy, and hydrograph was extrapolated across the Pointe-a-la-Hache Relief Outlet to Bohemia, LA, and West Pointe-a-la-Hache. The hydrograph at West Pointe-a-la-Hache was compared to a partial stage record at the location that also included the peak stage.

Maximum water surface elevations for the Mississippi River between Venice and Baton Rouge were developed for three hurricane conditions, Hurricane Betsy, SPH, and PMH, and for four different Mississippi River water surface profiles. The water surface profiles corresponded to stages of 2.7, 6.0, 13.0, and 20.0 ft MSL at Carrollton. A moderate speed of translation of 5 to 15 knots was used to generate additional surge hydrographs.

Combining stage-frequency data for the Mississippi River and the hypothetical parameters for different frequency hurricanes, a hypothetical hurricane isovel pattern based on 96 percent of the SPH wind speeds was derived. The isovel pattern was transposed, rotated, and moved along three tracks considered critical to five points along the river: mile 49, mile 40, mile 25, mile 15, and mile 10 Above Head of Passes. Using the new winds with the SPH forward speed and radius to maximum winds, hurricane surge elevations were computed at five points along the river.

Waves. For Reach A, Reach B1, Reach B2, and Reach C, wave runup was calculated using the methodology described in Orleans East Bank. In this case, the deep water wave is slightly larger than the significant wave. For the lateral levee portion of Segment 1, Reach A, no design runup was considered; approximately 2 ft of freeboard was added to the surge elevation to achieve the design elevation. For the St. Jude to City Price levee, the designers indicated that the wave runup was calculated using the criteria contained in the 1984 *Shore Protection Manual*.

A large expanse of marsh was present between the Mississippi River and Breton Sound. The marsh ranged in elevation from near 0.0 ft at Breton Sound to almost 10 ft along the alluvial ridge adjacent to the Mississippi River. Although the ridge and marsh would be submerged during the design storm, the decrease in water depth would have a pronounced effect on the characteristics of the waves propagating from Breton Sound. Wave heights and periods would be attenuated in the lesser depths over the marsh and ridge and grow in height as they propagate across the river toward the Mississippi River Levee on the west bank.

The wave characteristics for Breton Sound were determined according to procedures outlined in the Corps *Shore Protection Manual*. To determine the characteristics of the wave at the river levee, a method outlined in the 1984 *Shore Protection Manual* was used. Wave runup was computed using model study data developed by Saville that is presented in the *Shore Protection Manual*.

The MR&T levee on the west bank above mile 44 is protected from significant wave activity by the east bank MR&T and Reach C levees; therefore, a wave berm was not required. Above mile 44, the presence of small wind-generated waves in the river necessitate the addition of 2 ft of freeboard to the existing levee above the design still-water level.

Information regarding wave characteristics for the MR&T levee on the east bank was not available at the time of preparation of this report.

Summary. Table 33 contains maximum surge or wind tide level, wave, and design elevation information. The Mississippi River West Bank Levee requirement is for the levee to slope in a straight line from elevation 17 at mile 44 into the existing MR&T levee height at mile 48. Information on the design elevation for the Mississippi River East Bank Levee was not available at the time of preparation of this report.

Interior Drainage. The protection system in Reach A would have an impact on the interior drainage of two small areas totaling 115 acres. The first area, 75 acres, would be drained by an existing drainage facility. The second area, 40 acres, would be drained into the Plaquemines Parish Drainage Canal. Local interests would determine exactly how water from the second area would reach this canal.

In Reach B1, in the vicinity of Empire, the protection system would intercept drainage of an area of about 365 acres. To meet the requirement of navigation, a floodgate would be constructed with an 84-ft width and a sill elevation of -14 ft MSL. This floodgate would be adequate to dispose of runoff from intense storms.

In Reach B2, the discharge pipes of the Venice pump station would require modification to accommodate construction of a floodwall at the site. In addition, the flotation channel to the Venice pump station would serve as an outfall to allow drainage flow into open water.

In Reach C, five gravity structures with flap gates were constructed by the local interests prior to the construction of the federal protection system. The system was determined to have sufficient capacity to dispose of runoff from a 25-year, 24-hour storm with an average stage of 0.5 ft on the gulf side so that the sump pool elevation remains below 2.0 ft MSL and storage equivalent to about 3 in. of runoff below elevation 2.0 ft MSL would be available within 24 hours after cessation of runoff.

Table 33
Wave Runup and Design Elevations (Transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of Fetch, ft	Significant Wave Height, Hs, ft	Wave Period, T, sec	Deep Water Wave Height ft	Maximum Surge or Wind Tide Level ft	Runup Height ft	Free-board ft	Design Elevation Protective Structure, ft
Segment 1, Reach A, 0+00 – 4+50	DM01, Sup 05, November 1987	6.6	3.20	4.35	3.28	8.9 NGVD	0.0	2.1	11.0 NGVD
Segment 1, Reach A, 4+50 – 83+30	DM01, Sup 05, November 1987	6.6	3.20	4.35	3.28	8.9 NGVD	3.5	-	12.5 NGVD
Segment 2, Reach A, 83+30 – 315+00	DM01, Sup 05, November 1987	6.9	3.26	4.40	3.35	9.2 NGVD	3.8	-	13.0 NGVD
Segment 3, Reach A, 315+00 – 477+00	DM01, Sup 05, November 1987	7.3	3.49	4.50	3.60	9.6 NGVD	3.7	-	13.5 NGVD
Segment 4, Reach A, 477+00 – 613+00	DM01, Sup 05, November 1987	7.8	3.65	4.65	3.76	10.1 NGVD	3.8	-	14.0 NGVD
Segment 5, Reach A, 613+00 – 681+67.45	DM01, Sup 5, November 1987	8.0	3.71	4.70	3.82	10.3 NGVD	4.0	-	14.5 NGVD
Reach B1, levee	DM01, August 1971	6.7	3.1	4.2	3.2	12.0 MSL	3.0	-	15.0 MSL
Reach B1, floodwall	DM01, August 1971	6.7	3.1	4.2	3.2	12.0 MSL	6.5 – 7.7 [1]	-	18.5 – 20.0 [1] MSL
Reach B2, levee	DM01, Sup 04, August 1972	7.2	3.3	4.4	3.41	11.5 MSL	3.5	-	15.0 MSL
Reach B2, floodwall	DM01, Sup 04, August 1972	7.2	3.3	4.4	3.41	11.5 MSL	7.5	-	19.0 MSL
West Pointe a la Hache Back Levee	None – memo dated 21 Feb 1991	NA	3.53	3.89	NA	8.1 NGVD	3.9	-	12.0 NGVD
Return Levee to Mississippi River Levee	None – memos dated 21 Feb 1991 and 20 June 1991	NA	NA	NA	NA	NA	-	NA 2 est.	12.0 – 10.0 NGVD
Mississippi River Mile 10.8 – 20.0	DM01, Sup 06, March 1987	15.6 [2]	5.2	4.7	5.7	12.6 NGVD	3.4	-	16.0 NGVD
Mississippi River Mile 20.0 – 30.0	DM01, Sup 06, March 1987	14.9 [2]	5.4	4.5	-	13.5 NGVD	3.5	-	17.0 NGVD
Mississippi River Mile 30.0 – 44.0	DM01, Sup 06, March 1987	13.7 [2]	5.1	4.5	-	13.5 NGVD	3.4	-	17.0 NGVD
Mississippi River Mile 44.0 – Mile 48.0	None – memo dated 30 May 1991	NA	NA	NA	NA	14.0 – 13.5 NGVD	-	2.0	17.0 – 15.5 NGVD
Reach C, Phoenix to Davant, Levee	DM01, Sup 04, May 1972	11.5	5.45	5.35	5.70	13.0 MSL	4.0	-	17.0 MSL
Reach C, Phoenix to Davant, Floodwall	DM01, Sup 03, May 1972	11.5	5.45	5.35	5.70	13.0 MSL	7.0	-	20.0 MSL
Reach C, Davant to Bohemia, Levee	DM01. Sup 04, May 1972	12.5	5.45	5.65	5.68	14.0 MSL	3.0	-	17.0 MSL
Reach C, Davant to Bohemia, Floodwall	DM01, Sup 04, May 1972	12.5	5.45	5.65	5.68	14.0 MSL	6.0	-	20.0 MSL

[1] Height of floodwall would be dependent on levee configuration on floodside of structure.
[2] In Breton Sound.

Geotechnical.

Reach A - City Price to Tropical Bend. Approximately 12.8 miles of Hurricane Protection Levees and Floodwalls.

Geology (Reference 52). Map of area shown on following page. The project area is located within the Gulf Coastal Plain. More specifically, the area is located on the deltaic plain of the Mississippi River in a region of extremely low relief. The dominant physiographic features are the natural levees of the Mississippi River and its abandoned distributaries, and the marshlands and bodies of water that lie between the natural levees. Elevations range from approximately 6 ft along the natural levees to 0.0 ft in the area between the natural levees.

Foundation Conditions.

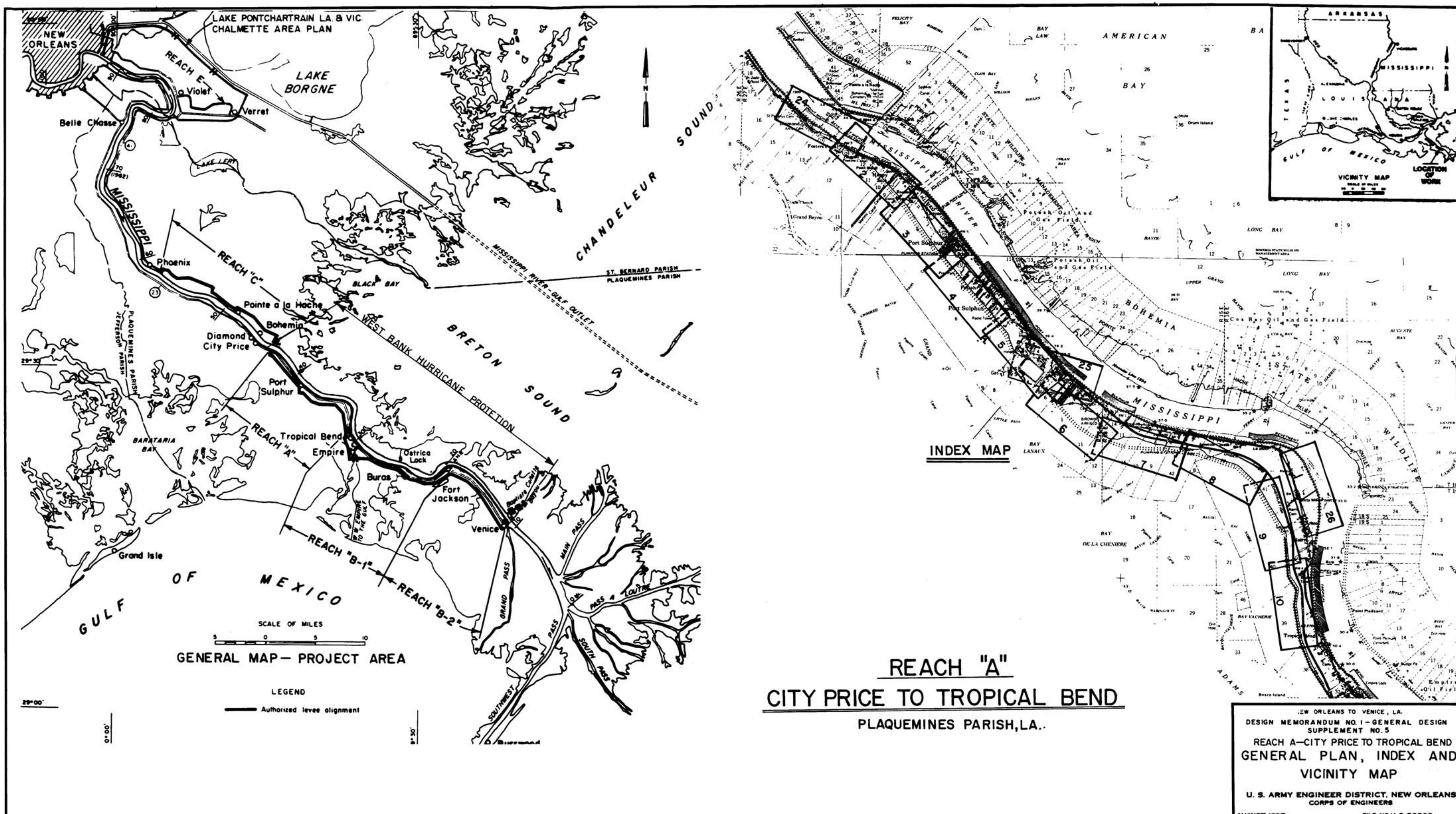
The subsurface consists of Holocene deposits of variable thickness underlain by Pleistocene material. Generally, the Holocene deposits consist of a surface layer of natural levee and/or marsh deposits underlain by interdistributary, intradelta, prodelta, and abandoned distributary deposits.

The marsh deposits, which range in thickness from 2 to 12 ft, consist of very soft to soft clays with peat and organic matter. Natural levee deposits overlie the marsh deposits between: Station 24+90 and Station 113+64, Station 234+00 and Station 329+53, and Station 445+00 and Station 501+12. These natural levee deposits vary in thickness up to 12 feet and consist of fat and lean clays, silts, and silty sands.

Abandoned distributary deposits are located in the vicinity of Stations 5+00, 70+00, 140+00, 170+00, 290+00, and 385+00. These abandoned distributary deposits consist of very soft to soft clays, silts, silty sands, and sands. The depths of the distributary deposits cannot be determined from available boring data; however, depths of 40 to 100 ft are indicated.

Underlying the marsh deposits between the abandoned distributaries are interdistributary deposits. These interdistributary deposits range in thickness from 25 to 65 ft and consist predominantly of fat clays. Occasionally lean clay, silt, silty sand, and sand lenses are found within the interdistributary deposits.

Field Investigation. A total of 30 general type and 36 undisturbed soil borings were made for design in association with the Reach A project. The bottom elevations of these borings range from -40 to -189 ft MSL. In addition to the above, borings P2-U, PI-G, and I-2-U were taken for the geotextile reinforced levee test section. No changes were made to the geologic profile or the shear strength lines to reflect the information from these borings. These borings are shallow and were taken to determine the local conditions and to find what type of materials the piezometers were going to be installed in.



LAKE PONTCHARTRAIN LA. & VIC.
CHALMETTE AREA PLAN
NEW ORLEANS
LAKE BORGNE
REACH E
Violet
Verret
Belle Chasse
CHANDELEUR SOUND
MISSISSIPPI
MISSISSIPPI RIVER-GULF OUTLET
Phoenix
REACH "C"
ST. BERNARD PARISH
PLAQUEMINES PARISH
BLACK BAY
Pointe a la Hache
Bohemia
Diamond
City Price
WEST BANK HURRICANE PROTECTION
BRETON SOUND
PLAQUEMINES PARISH
JEFFERSON PARISH
Port Sulphur
BARATARIA BAY
REACH "A"
Tropical Bend
Empire
Ostrica Lock
Buras
Fort Jackson
Venice
Grand Isle
GULF OF MEXICO
REACH "B-1"
REACH "B-2"
GRAND PASS
MAIN PASS
PASS A LOUTRE
SOUTH PASS
SOUTHWEST PASS
SCALE OF MILES
GENERAL MAP— PROJECT AREA
LEGEND
Authorized levee alignment
AMERICAN BAY
INDEX MAP
BOHEMIA
STATE WILDLIFE
MISSISSIPPI RIVER
Port Sulphur
LONG BAY
BAY LANAUX
BAY DE LA CHENIERE
ADAMS BAY
VICINITY MAP
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
GULF OF MEXICO
LOCATION OF WORK
REACH "A"
CITY PRICE TO TROPICAL BEND
PLAQUEMINES PARISH, LA.
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. I – GENERAL DESIGN
SUPPLEMENT NO. 5
REACH A—CITY PRICE TO TROPICAL BEND
GENERAL PLAN, INDEX AND
VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1987
FILE NO. H-2-30250

PLATE I

Levee Improvements.

a. *Existing Levee*. Throughout the project area, the existing hurricane levee has a FS slightly above 1 against a slope failure into the drainage canal which is on the protected side of the levee. Using conventional construction techniques, an enlargement of the existing levee gulfward from its present toe would result in a levee having a FS of 0.80 for a protected side analysis and 0.85 for a floodside analysis. This analysis applies where the levee crown elevation is 14.5 ft MSL. The critical slip surface for a failure into the canal is much deeper than the slip surface for a gulf side failure; it ranges from elevation -25 to -40 ft MSL. The design reaches are described in Table 34.

Table 34
Reach Geometry

Reach Number	Levee Stations	Crown Elevations ft MSL	Top of Berm ft MSL.	Berm Slope ft MSL	Bottom of Berm ft MSL
1*	0+00 to 4+00	11.0	not req'd	not req'd	
1**	4+50 to 83+30	12.5	7.5	IV on 11H	4.5
2	83+80 to 314+50	13.0	7.5	IV on 12H	5.0
3	315+00 to 477+00	13.5	8.0		5.0
4	477+50 to 613+00	14.0	8.5		5.5
5	613+50 to 681+90.791	14.5	8.5		5.5

* Pertains to upper return levee.
** Pertains to back levee.

b. *Geotextile Reinforced Levee Option*. Recent developments in high-molecular-weight polymers and weaving techniques have made it possible to enlarge the present levee in place. Geotextiles are textiles in a traditional sense, but consist of synthetic fibers rather than natural ones like cotton, wool, and silk. Thus, biodegradation is not a problem. The fibers are made into a flexible, porous fabric by standard weaving machinery. Geotextiles are designed to provide a wide range of porosity. The new generation of reinforcing geotextiles are made from polyester, nylon, aramid, or fiberglass fibers. An extremely strong single layer fabric can be manufactured from these fibers with tensile strain characteristics that are compatible with soft clay soils. Geotextiles made from other fibers generally exhibit excessive creep properties under a lower percentage of their ultimate load strength. Excessive creep can destroy any reinforced soil structure. In order for reinforcement to be effective, it must provide the required tensile force at levels of strain that are compatible with the soils at the site. The soft clay soils throughout this site reach maximum deviator stresses between 3 and 5 percent strain. A polyester geotextile is recommended, and the maximum recommended strain is 5 percent. Polyester is presently the most economical geotextile within the high strength group. At present, geotextiles provide the most viable alternative for raising the existing levee in place to design grade.

Pile Foundation.

The T-wall will be supported by piling, battered as required, to provide stability against the unbalanced lateral water loads. In compression, a FS of 2.0 was applied to the shear strength and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths and coefficients of $K_0 = 0.7$ (S-Case) and $K_0 = 1.0$ (Q-Case) were used. Design of the T-wall pile foundation was performed for both the Q- and S-Cases. In these two designs, the Q-Case shear strengths governed. Pile design loads versus tip elevations, and subgrade moduli versus tip elevations were computed. Settlement of the piles due to consolidation during maximum loading is not expected since the major loads are caused by hurricane-induced stages of insufficient duration for consolidation of the foundation clays to ensue.

It is recommended that pile load tests be performed at the Homeplace (Gainard Woods) Pumping Station prior to preparation of the plans and specifications. A minimum of two piles would be load tested: one at the design tip elevation, and another 10 ft below the design tip elevation. The piles would be tested in both compression and tension, allowing a minimum of 14 days between tests.

Slope Stability.

a. *Geometry*. The design section consists of a 1 vertical on 3 horizontal slope on the protected side, an 8-ft. crown, and a 1 vertical on 3 horizontal slope from the crown to the wave berm. The slope from the lower berm elevation to the existing ground is 1 vertical on 3 horizontal. Specifics for each reach are presented in Table 34.

b. *Factors of Safety*. The geotextile requirements to develop FSs of 1.3 or 1.5, as appropriate, were computed. A FS of 1.5 is used in the vicinity of pipelines and other structures. Two layers of geotextile will be used in reaches where a FS of 1.5 is required.

c. *Construction Fill.* A sand core is used in the gulf side enlargement of the existing levee. Sand has several advantages in this type of construction, especially in this area:

(1) Sand improves the frictional resistance between the geotextile and the fill.

(2) Sand provides a more stable foundation to place clay fill on, and sand reduces the chances of a failure within the fill.

Sand helps relieve the pore pressure at the soil/fabric interface caused by the foundation loading.

Clay will be placed over the sand blanket to provide a seepage barrier, erosion control, and a medium in which grass will grow. A minimum of 2 ft of clay will be placed on the sand blanket over the wave berm. A much thicker clay cover will be placed under the centerline of the new levee. An impermeable core is provided by the existing levee, which will effectively prevent flow through the section.

d. *Geotextile Design Methodology.* Stability analyses were performed using the sliding wedge method and the results were compared to values obtained from circular arc analyses for the section to elevation 14.5 ft MSL design grade. For this job, at shallow depths, the wedge method of analysis is more conservative and requires a stronger fabric to achieve the same FS; at greater depths, the geotextile requirements are approximately the same for both methods. The geotextile will provide the required tensile force to reinforce the soil and increase the FS to 1.3 or 1.5 where required against failure.

A geotextile that provides the necessary tension for a chosen factor of safety meets the following requirements:

(3) Tensile Requirements. Tensile requirements were computed using the following equation:

$$T = \mathrm{FS}\,(D) - (D) - R$$

$$D = D_a - D_p$$

$$R = R_a + R_b + R_p$$

FS = required factor of safety

Since it is customary to report fabric strength in pounds per inch, the T value is divided by 12.

Sufficient embedment length is available to develop the necessary tensile force.

(4) Embedment Length. The embedment length required to provide the frictional (cohesive) components to develop T, is calculated by combining the contributions from the top and bottom surfaces of the geotextile strip.

$$L = \frac{T}{[\gamma h \tan\phi + c]* + [\gamma h \tan\phi + c]**}$$

where

L = feet

T = pounds per foot

ϕ = friction angle between soil and geotextile

* = top surface

** = bottom surface

A length equal to or greater than L has to be available from the intersection of the active wedge and geotextile and into the stable portion of the slope.

I-Walls. The stability and required penetration of the steel sheet piling below the ground surface was determined by the Method of Planes. The long-term S shear strengths (c = 0) governed for design. A FS of 1.5 was applied to the friction angle as follows:

ϕ_d (developed friction angle) = $\tan^{-1} \underline{(\tan \phi_a)}/\mathrm{FS}$

This developed angle was used to determine $K_a = \tan^2 (45° - \phi_d/2)$, and $K_p = 1/K_a$. Using the two resulting developed shear strengths and net horizontal static water pressure, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using. these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetration required for stability was determined as that elevation at which the summation of overturning moments about the bottom of the sheet piling approached zero.

Floodwalls. Floodwalls are proposed for use in areas where an earth levee cannot be economically built. A new levee to elevation 10 ft will be used to make the transition from the sand core levee alignment to the existing back levee. This new levee is designed for an initial FS of 1.2 and a final FS of 1.3 (after settlement to elevation 7 ft). The existing levee I-wall composite section is designed for a FS of 1.3 against shear failure. For the high water hurricane loading case, with water to still-water level, the I-wall sections are designed for a FS of 1.2 against shear failure. In all cases, the penetration of the sheet pile is designed for a FS of 1.5. The wave effect was applied as a line force acting at the centroid of the wave pressure diagram. At the site of the two pumping stations, the existing levee across each station will be degraded to elevation 4 ft and an inverted T-wall construction on top of it. The I-wall will be tied into each end of the T-wall.

Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion protection, other than sodding, is considered necessary on the levee slopes along most of the levee alignment. However, foreshore protection will be constructed on the floodside levee toe in areas where damages could occur from waves generated by other than hurricane winds. This will be any berm or levee slope which is constructed into the open bays and bayous. The foreshore protection will consist of 24 in. of riprap on a 9-in.-thick shell bedding. At the pumping stations, protection against erosion will consist of 18 in. of riprap over a 9-in.-thick shell bedding.

Reach B1 to Tropical Bend to Fort Jackson – Approximately 12 miles of levee built by hydraulic dredge with shape-ups (Reference 53). Images of area shown on following pages.

Geology. The project area is located within the Central Gulf Coastal Plain. Specifically, the area is located on the modern subdelta which projects gulfward from the deltaic plain of the Mississippi River. It is a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from +5 ft along the crests of the natural levees to 0.0 ft MSL in the marshlands between the natural levee ridges. The numerous inland bodies of water vary in depth from 1 to 10 ft. The Mississippi River channel varies in depth from 70 to 190 ft below sea level.

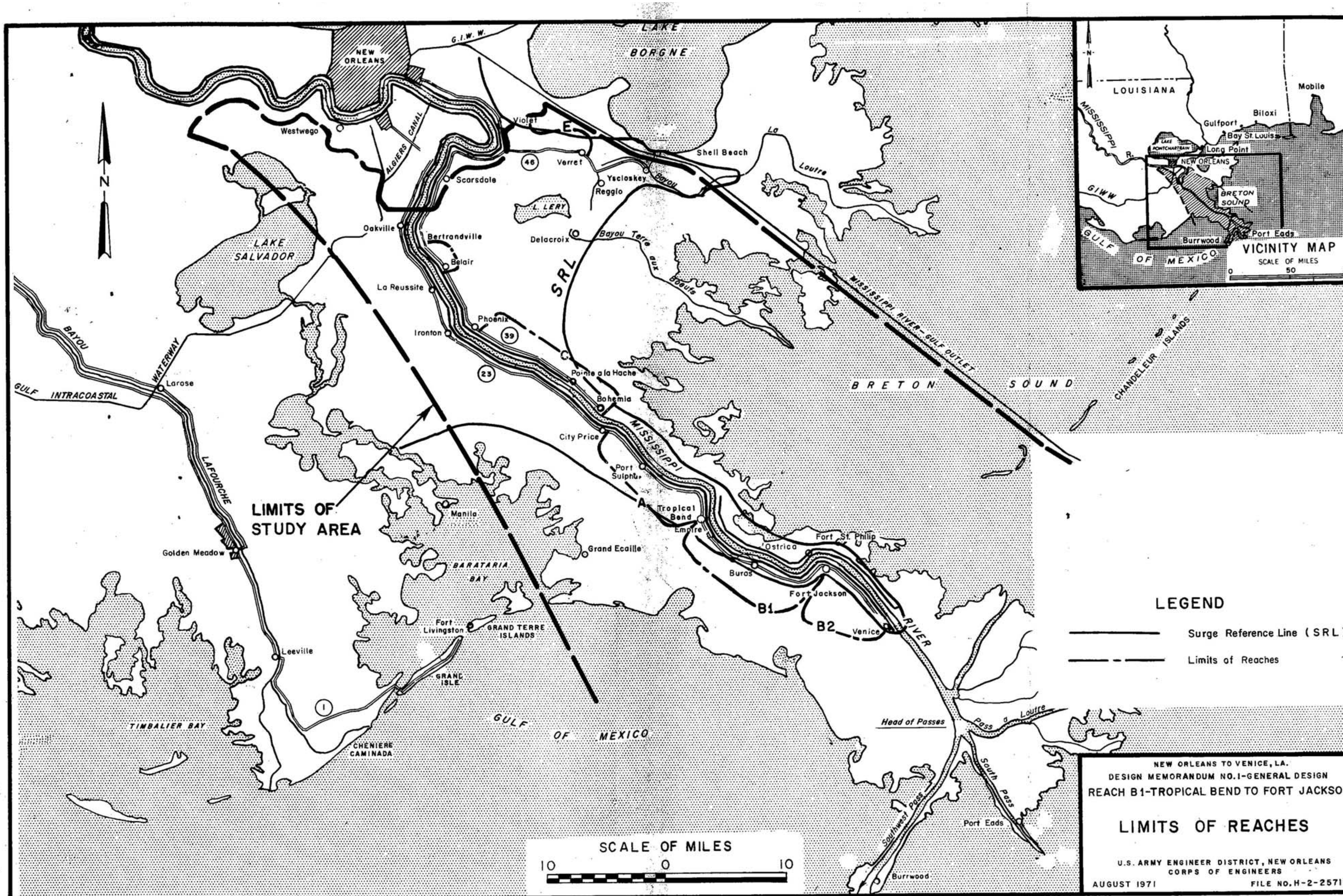
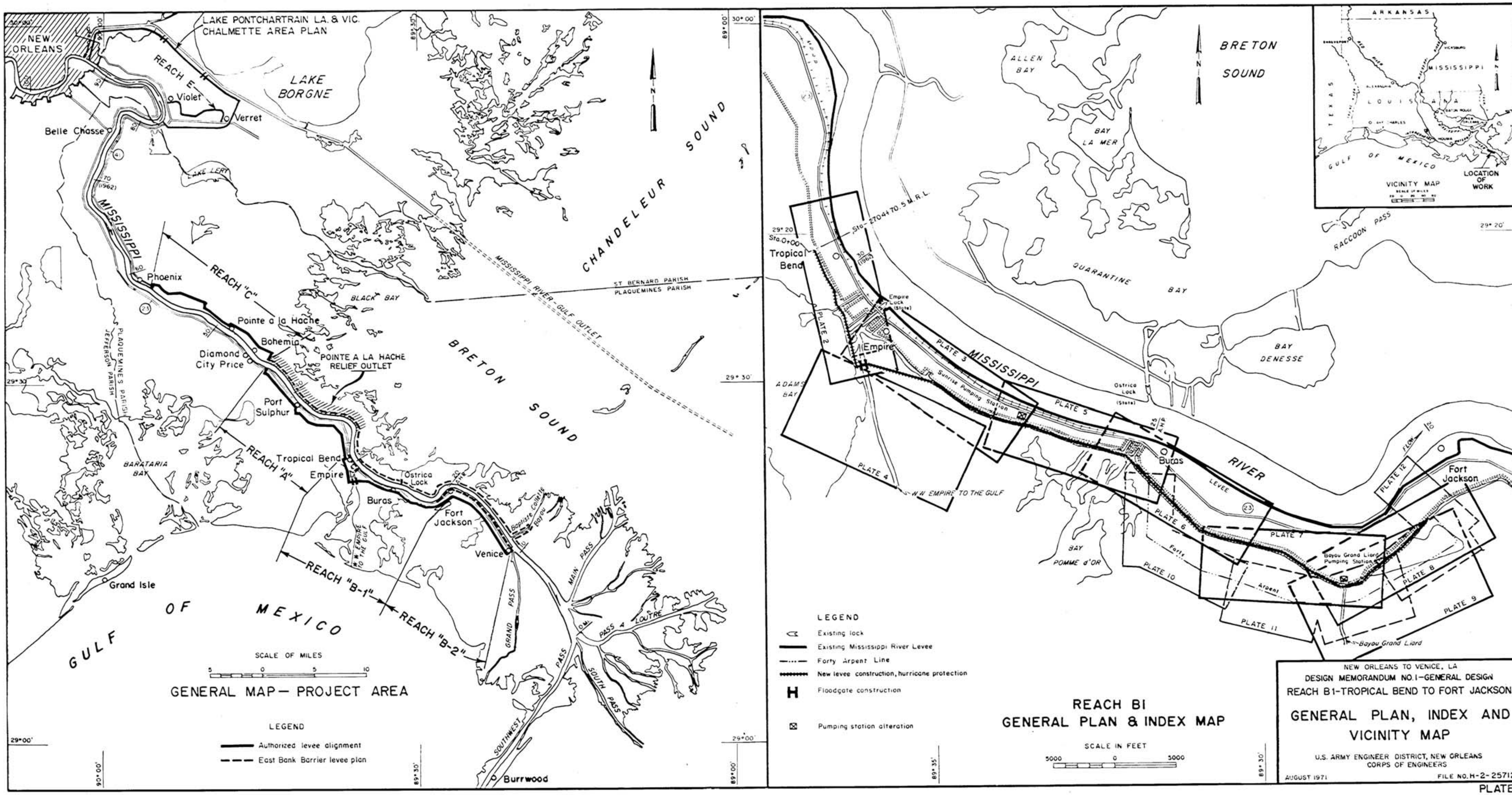

NEW ORLEANS
LAKE BORGNE
G.I.W.W.
Westwego
ALGIERS CANAL
Violet
Verret
Shell Beach
Ycloskey
Reggio
Scarsdale
L. LERY
Oakville
Bertrandville
Belair
Delacroix
Bayou Terre aux Boeufs
LAKE SALVADOR
La Reussite
SRL
Phoenix
Ironton
MISSISSIPPI RIVER–GULF OUTLET
BAYOU
WATERWAY
Larose
GULF INTRACOASTAL
Pointe a la Hache
Bohemia
BRETON SOUND
CHANDELEUR ISLANDS
City Price
MISSISSIPPI
Port Sulphur
LAFOURCHE
LIMITS OF STUDY AREA
Manila
Tropical Bend
Empire
Ostrica
Fort St. Philip
Grand Ecaille
Golden Meadow
BARATARIA BAY
Buras
Fort Jackson
B1
B2
Venice
RIVER
Fort Livingston
GRAND TERRE ISLANDS
Leeville
GRAND ISLE
TIMBALIER BAY
GULF OF MEXICO
CHENIERE CAMINADA
Head of Passes
Pass a Loutre
South Pass
Southwest Pass
Port Eads
Burrwood
SCALE OF MILES
10 0 10
LOUISIANA
MISSISSIPPI
Mobile
Biloxi
Gulfport
Bay St. Louis
Long Point
NEW ORLEANS
GIWW
BRETON SOUND
Burrwood
Port Eads
GULF OF MEXICO
VICINITY MAP
SCALE OF MILES
0 50 100
LEGEND
Surge Reference Line (SRL)
Limits of Reaches
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. 1-GENERAL DESIGN
REACH B1-TROPICAL BEND TO FORT JACKSON
LIMITS OF REACHES
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1971
FILE NO. H-2-25712

PLATE A-1


LAKE PONTCHARTRAIN LA. & VIC.
CHALMETTE AREA PLAN
NEW ORLEANS
REACH E
Violet
Verret
LAKE BORGNE
Belle Chasse
LAKE LERY
CHANDELEUR SOUND
MISSISSIPPI
Phoenix
REACH "C"
ST BERNARD PARISH
PLAQUEMINES PARISH
MISSISSIPPI RIVER-GULF OUTLET
BLACK BAY
Pointe a la Hache
Bohemia
POINTE A LA HACHE RELIEF OUTLET
Diamond
City Price
BRETON SOUND
PLAQUEMINES PARISH
JEFFERSON PARISH
Port Sulphur
BARATARIA BAY
REACH "A"
Tropical Bend
Empire
Ostrica Lock
Buras
Fort Jackson
Venice
Grand Isle
REACH "B-1"
REACH "B-2"
GRAND PASS
MAIN PASS
PASS A LOUTRE
SOUTH PASS
SOUTHWEST PASS
GULF OF MEXICO
SCALE OF MILES
GENERAL MAP— PROJECT AREA
LEGEND
Authorized levee alignment
East Bank Barrier levee plan
Burrwood
BRETON SOUND
ALLEN BAY
BAY LA MER
QUARANTINE BAY
RACCOON PASS
BAY DENESSE
Tropical Bend
Sta.0+00
Sta.2704+70.5 M.R.L.
Empire Lock (State)
Empire
ADAMS BAY
PLATE 2
PLATE 3
PLATE 4
PLATE 5
PLATE 6
PLATE 7
PLATE 8
PLATE 9
PLATE 10
PLATE 11
PLATE 12
Sunrise Pumping Station
Ostrica Lock (State)
MISSISSIPPI RIVER
Buras
Fort Jackson
W.W. EMPIRE TO THE GULF
BAY POMME d'OR
Forty Arpent
Bayou Grand Liard Pumping Station
Bayou Grand Liard
LEGEND
Existing lock
Existing Mississippi River Levee
Forty Arpent Line
New levee construction, hurricane protection
Floodgate construction
Pumping station alteration
REACH BI
GENERAL PLAN & INDEX MAP
SCALE IN FEET
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
GULF OF MEXICO
VICINITY MAP
LOCATION OF WORK
NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 1—GENERAL DESIGN
REACH B1—TROPICAL BEND TO FORT JACKSON
GENERAL PLAN, INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1971
FILE NO. H-2-25712
PLATE 1

Foundation Conditions. A generalized soil profile delineating the subsurface conditions along the project alignment shows that the subsurface consists of Recent deposits of very soft to medium clay soils with peat, silt, and sand layers. The upper 10 to 20 ft of marsh deposits generally consist of very soft organic clays, clays, and peat. Between Stations 0+00 and 399+00 the marsh deposits are underlain by interdistributary deposits of approximately 8 to 20 ft of layers of silt, silty sand, and sand. Below these layers is fat clay with layers of silt, silty sand, and sand. Between Stations 417+00 and 635+72 the marsh deposits are underlain by predominantly fat clay with intermittent thin layers of silt, sandy silt, and sand. Four abandoned distributaries are located below the marsh deposits between the following stations: 92+20 to 109+60, 398+50 to 417+50, 532+40 to 551+90, and 610+50 to 615+30. These abandoned distributaries are composed of alternate layers of clay, silt, silty sand, and sand. The dominant feature in the design of all the levee sections is the very soft foundation condition between elevation 0 and -12 ft.

Field Exploration. A total of 112 general type and 17 undisturbed borings was made in conjunction with the project. Eight general type borings were made by the Louisiana Department of Highways to locate a source of sand for borrow in the Mississippi River. Twenty-seven general type and two 3-in.-diameter undisturbed borings were made by the Louisiana Department of Public Works along the authorized levee alignment at the request of the Commission Council. Seventy-seven 1⅞-in.-ID core barrel and fifteen 5-in.-diameter undisturbed borings were taken by the Corps. The bottom elevations of the general type and undisturbed borings range from -40 to -50 ft and -77 to -242 ft, respectively.

Levee Improvements. In general, the protection will consist of a levee. Between Stations 0+00 and 98+81, the protection will consist of a conventional hydraulic clay fill levee. From Station 104+81 to Station 635+72, the protection will consist of a hydraulic clay fill levee with a core composed of sand. A floodgate will be located where the Empire Waterway crosses the project alignment between Stations 98+71 and 104+91. Cantilever I-type and T-type walls will be used in the vicinity of the Sunrise (Station 232+31 to Station 242+41) and Grand Liard (Station 532+76 to Station 539+81) Pumping Stations to avoid relocations or major modifications to these facilities.

Pile Foundation.

The T-walls will be supported by piling, battered as required, to provide stability against the unbalanced lateral water loads. The inverted T-type floodwalls will be used in lieu of the I-type for reasons mentioned above. In compression, a FS of 1.75 was applied to the shear strength and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths and a coefficient of $K_O = 0.7$ was used. One design was performed for both the Q- and S-Cases for the Bayou Grand Liard Pumping Station and is applicable to the Sunrise Pumping Station since it was considered more conservative. The Q-Case governed. Settlement of the piles due to consolidation will not be a problem since the major loads are caused by hurricane waterheads of insufficient duration for consolidation of the foundation clays to ensue.

During construction, one 12-in.-square concrete pile will be driven at the Bayou Grand Liard Pumping Station. The pile will be tested in compression to twice the design load (35 tons). If the pile fails before this load is reached, the spacing will be adjusted accordingly. Since spacings of greater than 10 ft on the tension piles are not desirable, the tension piles will be working well below the design load, and no pile test will be performed in tension. Because of the small number of piles at the Sunrise Pumping Station, there will be no test piles at this site. In the interest of avoiding a tension pile test and having only one form for casting concrete piles, tension piles will be the same length as compression piles and spaced a maximum of 10 ft on centers, thus reducing the design load to well below the theoretical allowable tension load.

Levee Stability.

Levees and dikes. In the interim between the publication of the GDM dated March 1967 and the GDM dated August 1971, plans and specifications were prepared for the first lift construction on two reaches of the project from Stations 0+00 to 98+55.3 and Stations 104+70 to 340+20. An additional set of plans and specifications were prepared by an A-E for the Commission Council and approved by the District Engineer for a reach of levee from Stations 340+20 to 377+50. Plans and specifications for the remaining section between Stations 377+50 and 635+72 will be prepared after approval of the general design memorandum. Stability plates 89 through 116 are divided to reflect the above segments as follows:

Stations	Segments
0+00 to 98+81	Tropical Bend to Empire
104+81 to 337+72	Empire to Buras
337+72 to 635+72	Buras to Fort Jackson

Q shear stability analyses were performed for these segments using four different shear strength criteria as shown on plates 87 and 88. Using sections and Q shear strengths representative of the existing conditions along the alignment, the slopes and minimum berm distances for the levee and dike sections were determined, and the minimum FS of the levee with respect to shear failure in the levee and foundation was 1.3 and 1.5 for failure into the adjacent borrow pit. The retaining and ponding dike sections were designed for a minimum FS of 1.2 for failure into the sand core trench and interior dike borrow, respectively, and a minimum FS of 1.3 for failure into the ponding area and borrow area, respectively. Borings 1-DU-1 and 2-DU-1 which were taken for design of the second lift showed no gain of shear strength. However, the second lift sections from 0+00 to 46+00 are somewhat larger than the first lift sections because the spoil from the first lift is serving as a floodside berm for the retaining dike. Since there was no spoil from 46+00 to 98+71, the second lift section is the same as the first lift section.

I-Walls. The stability and required penetration of the steel sheet piling below the fill surface was determined by the Method of Planes. The long-term S shear strengths (b = 0) governed the design. Prior to the preparation of plans and specifications for the I-type floodwall tying the final levee section to the I-wall in the existing back levee at Sunrise and the T-wall at Grand Liard, additional borings and analyses will be performed. A FS of 1.25 was applied to the friction angle as follows:

$$\phi_d \text{ (developed friction angle)} = \tan \frac{-1(\tan \phi_A)}{FS}$$

This developed angle was used to determine K_a and K_p lateral earth pressure coefficients as follows: $K_a - \tan_2 (45° - \phi_d/2)$ and $K_p - 1/K_a$.

Using the resulting shear strengths and net horizontal static water, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetrations required for stability were determined as those where the summation of moments approached zero.

T-Wall. A steel sheet-pile cutoff will be used beneath the T-walls to provide protection against seepage. The recommended tip elevations of the cutoff below the T-walls are shown on plates 25 and 26. No sheet-pile analysis was performed for the Sunrise Pumping Station since the unbalanced water load is negligible. The analysis for the Bayou Grand Liard Pumping Station is shown on plate 120 and was analyzed under the following design assumptions:

a. Conventional Q shear stability analyses utilizing a FS of 1.5 applied to the soil strength parameters were performed at 1-ft intervals.

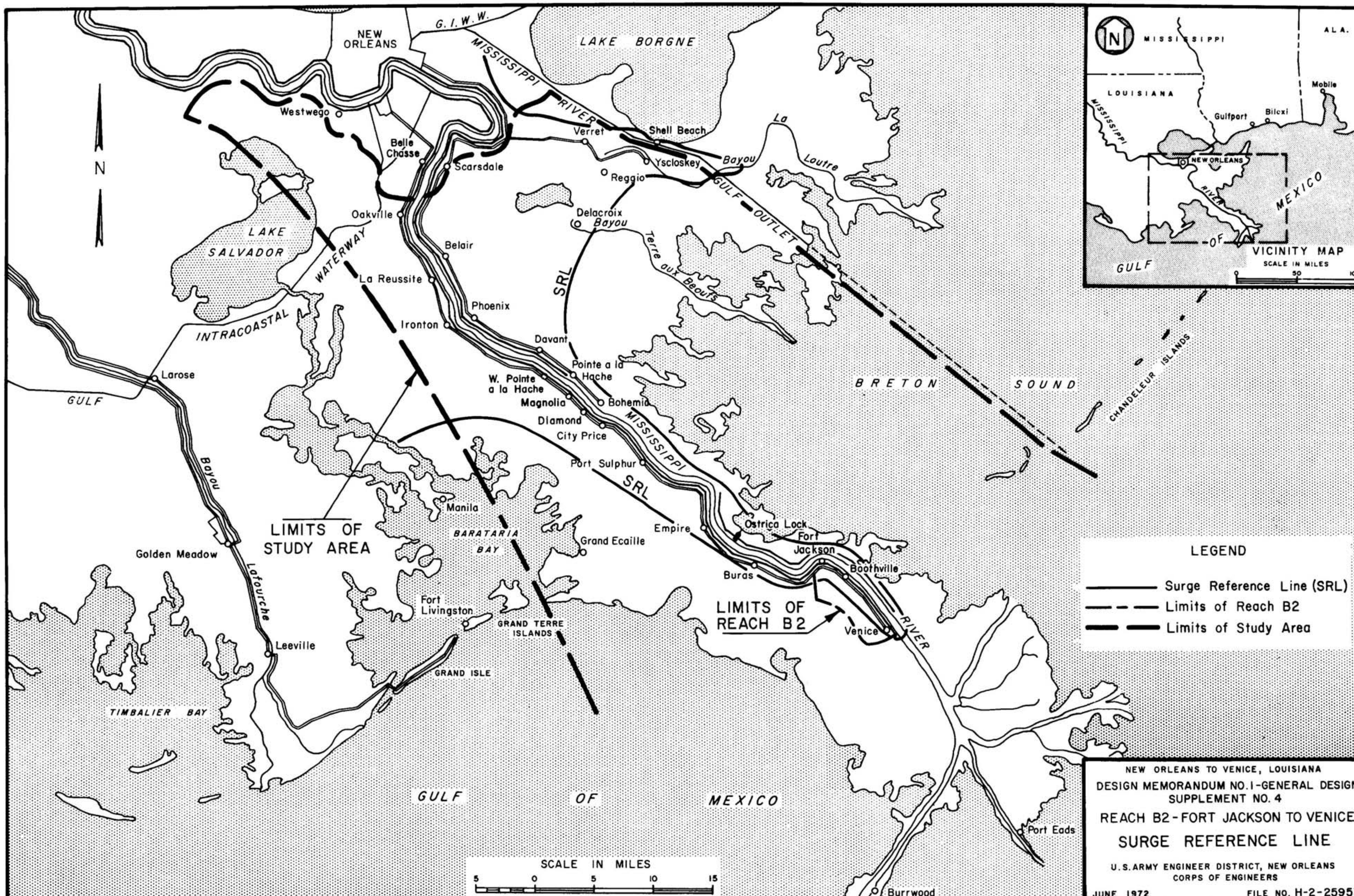
b. Net driving force = $D_p + R_A + R_B + R_p - D_A$.

c. The driving force above the base of the structure and the horizontal hydrostatic load were carried by the structure.

d. If the net driving force is positive, there is available horizontal soil resistance in excess of the unbalanced water load and, therefore, the bearing piles are not required to carry any additional lateral load acting on the sheet-pile cutoff.

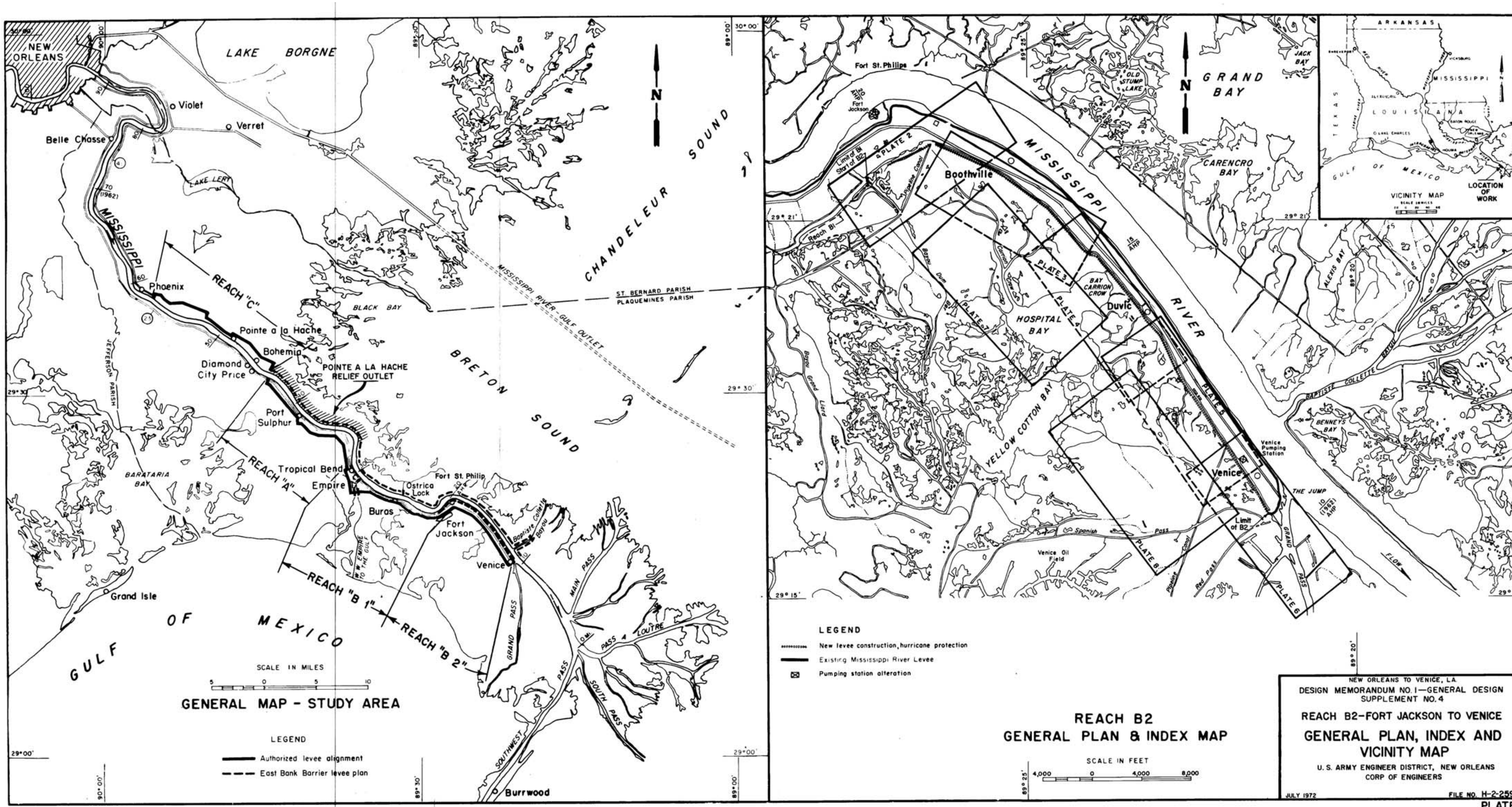
Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion protection is considered necessary on the levee slopes along most of the levee alignment other than sodding. However, foreshore protection will be placed along the bank of Adams Bay from Station 57+50 to the Empire floodgate, along the bank of the Empire to Gulf Waterway from Station 62+00 to the Empire floodgate, and on the landside and floodside of the canal closures between Stations 46+50 and 87+00 to protect the levee from damages which could occur from waves generated by other than hurricane winds. Design sections for the foreshore protection are shown on plate 22. At the Sunrise and Bayou Grand Liard Pumping Stations the erosion protection will consist of 18 in. of riprap over a 6-in.-thick shell bedding. Erosion protection at the Empire floodgate will consist of 2 ft of riprap on a minimum 1-ft blanket of clamshell.

<u>Reach B2 – Ft. Jackson to Venice.</u> Nine miles of levee constructed by hydraulic lifts and shaping to elevation 50 ft with an 8-ft crown (Reference 50). Images of area shown on following pages.



NEW ORLEANS
G.I.W.W.
LAKE BORGNE
MISSISSIPPI RIVER
GULF OUTLET
Westwego
Belle Chasse
Scarsdale
Verret
Shell Beach
Ycsloskey
Bayou
La Loutre
Reggio
Delacroix
Bayou
Terre aux Beoufs
Oakville
LAKE SALVADOR
WATERWAY
INTRACOASTAL
Belair
La Reussite
Phoenix
Ironton
SRL
Davant
Pointe a la Hache
W. Pointe a la Hache
Magnolia
Bohemia
Diamond
City Price
Port Sulphur
MISSISSIPPI
Larose
GULF
Bayou
Lafourche
Golden Meadow
Leeville
LIMITS OF STUDY AREA
Manila
BARATARIA BAY
Grand Ecaille
Empire
Ostrica Lock
Fort Jackson
Boothville
Buras
LIMITS OF REACH B2
Venice
RIVER
Fort Livingston
GRAND TERRE ISLANDS
GRAND ISLE
TIMBALIER BAY
BRETON SOUND
CHANDELEUR ISLANDS
GULF OF MEXICO
Port Eads
Burrwood
SCALE IN MILES
5 0 5 10 15
N
MISSISSIPPI
ALA.
LOUISIANA
Mobile
Gulfport
Biloxi
MISSISSIPPI
NEW ORLEANS
RIVER
MEXICO
OF
GULF
VICINITY MAP
SCALE IN MILES
0 50 100
LEGEND
Surge Reference Line (SRL)
Limits of Reach B2
Limits of Study Area
NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. I-GENERAL DESIGN
SUPPLEMENT NO. 4
REACH B2-FORT JACKSON TO VENICE
SURGE REFERENCE LINE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JUNE 1972
FILE NO. H-2-25953

PLATE A-10


NEW ORLEANS
LAKE BORGNE
Violet
Verret
Belle Chasse
LAKE LERY
MISSISSIPPI
CHANDELEUR SOUND
MISSISSIPPI RIVER-GULF OUTLET
ST. BERNARD PARISH
PLAQUEMINES PARISH
REACH "C"
Phoenix
BLACK BAY
Pointe a la Hache
Bohemia
POINTE A LA HACHE RELIEF OUTLET
Diamond
City Price
JEFFERSON PARISH
BRETON SOUND
Port Sulphur
REACH "A"
BARATARIA BAY
Tropical Bend
Empire
Ostrica Lock
Fort St. Philip
Buras
Fort Jackson
EMPIRE TO THE GULF
Baptiste Collette Bayou
Venice
REACH "B 1"
REACH "B 2"
Grand Isle
GULF OF MEXICO
GRAND PASS
MAIN PASS
PASS A LOUTRE
SOUTH PASS
SOUTHWEST PASS
Burrwood
SCALE IN MILES
GENERAL MAP - STUDY AREA
LEGEND
Authorized levee alignment
East Bank Barrier levee plan
Fort St. Philips
Fort Jackson
GRAND BAY
OLD STUMP LAKE
MISSISSIPPI RIVER
Boothville
CARENCRO BAY
Reach B1
PLATE 2
PLATE 3
PLATE 4
PLATE 5
PLATE 6
PLATE 7
PLATE 8
BAY CARRION CROW
Duvic
HOSPITAL BAY
YELLOW COTTON BAY
ALEXIS BAY
BAPTISTE COLLETTE BAYOU
BENNEYS BAY
Venice Pumping Station
Venice
THE JUMP
Limit of B2
Spanish Pass
Venice Oil Field
Red Pass
GRAND PASS
FLOW
ARKANSAS
MISSISSIPPI
LOUISIANA
TEXAS
GULF OF MEXICO
VICINITY MAP
LOCATION OF WORK
LEGEND
New levee construction, hurricane protection
Existing Mississippi River Levee
Pumping station alteration
REACH B2
GENERAL PLAN & INDEX MAP
SCALE IN FEET
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO. I—GENERAL DESIGN
SUPPLEMENT NO. 4
REACH B2-FORT JACKSON TO VENICE
GENERAL PLAN, INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORP OF ENGINEERS
JULY 1972
FILE NO. H-2-25953
PLATE I

Geology. The project area is located within the Central Gulf Coastal Plain. More specifically, the area is situated on the deltaic plain of the Mississippi River, a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and its abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from approximately 5 ft along the crests of the natural levees to 0.0 ft MSL in the marshlands between the natural levee ridges.

Foundation Conditions. The subsurface consists of recent deposits of very soft to medium clay soils with peat, silt, and sand layers. The upper 5 to 18 ft of marsh deposits generally consist of very soft organic clays, clays, and peat. Between Stations 0+00 and 4+50 the marsh deposits are underlain by interdistributary deposits of soft clay with layers of silt. Between Station 4+50 and 480+31 the marsh deposits are underlain by 20 to 40 ft of intradelta deposits consisting primarily of very soft to medium clays with alternating lenses and layers of silt, sand, and silty sand. These deposits are in turn underlain by interdistributary deposits consisting of soft to medium clays with very few lenses and layers of silt. Two abandoned distributaries are located below the marsh deposits – one between Stations 457+60 and 460+60 and the other between Stations 466+00 and 469+00. These abandoned distributaries are composed of alternate layers of clay, silt, silty sand, and sand. The dominant feature in the design of all the levee sections is the very soft marsh deposits in the upper 5 to 18 ft of the foundation.

Field Exploration. A total of 39 general type and nine undisturbed borings were made in association with the Reach B2 project. Twenty general type borings were made by the Louisiana Department of Highways to locate a source of sand for borrow between mile 12 and mile 18.5 in the Mississippi River. Nineteen 1⅞-in.-ID core barrel and nine 5-inch diameter undisturbed borings were taken by the Corps, New Orleans District. The bottom elevations of the general type and undisturbed borings ranged from -45 to -79 ft and -71 to -237 ft, respectively. Prior to the preparation of plans and specifications, additional general type borings will be taken in the sand and clay borrow areas.

Levee Improvements.

The Reach B2 project will consist of a sand core hydraulic clay fill levee, and extend from a junction with the terminus of the proposed Reach B1 project levee in the vicinity of Fort Jackson (Station 0+00) for about 9 miles southeast to a junction with the proposed highway ramp for relocation of Louisiana Highway 23 at Venice (Station 475+33). The proposed realigned Mississippi River levee will join the opposite side of the highway ramp to complete the Reach B2 project. The Reach B2 levee, the realigned river levee, and the highway ramp will be constructed to elevation 15 ft MSL, thereby forming a uniform net grade for the Reach B2 levee system. The Reach B2 levee centerline will be approximately 190 ft marshward and generally parallel to the existing non-federal back levee. Minor changes in levee centerline location will be permitted in the field where the changes will result in a more favorable alignment.

Floodwalls at Venice Pumping Station. The Venice Pumping Station is located on the protected side of the existing back levee with discharge pipes passing through the levee just below the road surface on the levee crown. To provide continuous protection at minimum cost, the new levee will tie into the existing back levee approximately 100 ft to each side of the discharge pipe crossings. Inverted T-type floodwall in the existing levee and I-type floodwall in the tie-in levees will be constructed to elevation 19 ft MSL. The tie-in levees will have an 8-foot crown width at elevation 10 ft MSL. Stability of the existing levee requires that it be degraded to elevation 5 ft MSL and the slopes be regraded to 1 on 3. Where the discharge pipes pass through the floodwall, provisions to accommodate settlement or deflection of the wall or any small movements of the pipes will be provided.

Pile Foundations.

The T-wall will be supported by piling, battered as required, to provide stability against the unbalanced lateral water loads. In compression, a FS of 1.75 was applied to the shear strength, and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths, and a coefficient of $K_0 = 0.7$ was used. Design of the T-wall pile foundation was performed for both the Q- and S-Cases. The Q-Case governed. Pile design loads versus tip elevations, and subgrade moduli versus tip elevations are shown on plate 72. Settlement of the piles due to consolidation is not expected since the major loads are caused by hurricane-induced stages of insufficient duration for consolidation of the foundations' clays to ensue.

During construction, one 12-in.-square concrete pile will be driven to the design tip elevation (-50.4 ft) in the vicinity of the Venice Pumping Station. The test site will be located in the vicinity of boring 28-B2UC. The pile will be tested in compression to 78 tons (twice the design load). If the pile fails before this load is reached, the spacing will be appropriately adjusted. To eliminate a tension pile test and have only one form for casting concrete piles, tension piles will be the same length as compression piles (60 ft) and spaced a maximum of 10 ft on centers thereby reducing the design load to 22.5 tons, which is well below the theoretical allowable tension load of 30 tons. If the spacing of compression piles has to be reduced, the spacing of tension piles will be reduced by the same ratio.

Slope Stability.

a. *Levees and Dikes.* Using levee sections and Q shear strengths representative of conditions along the project alignment, slopes and minimum berm distances for the levee and dike sections were determined by the Method of Planes. Levee sections were designed for a minimum FS of 1.3 with respect to shear failure in the levee and foundation and 1.5 for failure into the adjacent borrow pit. The retaining dike sections were designed for a minimum FS of 1.3 for failure into either the sand core trench or the retaining dike borrow pit. The pending dike sections were designed for a minimum FS of 1.2 for failure into the interior dike borrow pit, and a minimum FS of 1.3 for failure into the marsh borrow area.

b. *Floodwalls.* A combination of I-type and inverted T-type floodwalls will be used at the Venice Pumping Station. The use of I-wall along the existing back levee at this location was not feasible because a minimum levee crown elevation of 10.0 ft would be required to prevent excessive deflection of the wall. A stability analysis was performed with the levee crown at elevation 10.0 ft MSL and the I-wall in place. In order to maintain the required FS of 1.30, large stability berms would be necessary in both the landside and floodside drainage pits resulting in either relocation or major modifications to the pumping station. Therefore, a 365-ft length of T-wall with the levee degraded to elevation 5.0 ft MSL will be used along the existing back levee with I-wall joining the T-wall to the full earthen levee section. For the stability analyses, the wave effect was applied as a line force acting at the centroid of the wave pressure diagram. The water pressure diagram resulting from the wave action alone was considered effective only to the levee crown.

I-Walls.

Cantilever I-wall. The stability and required penetration of the steel sheet piling below the fill surface was determined by the Method of Planes. The long-term S shear strengths (c = 0) governed for design. Prior to the preparation of plans and specifications for the I-wall tying the full earthen levee section to the T-wall at the Venice Pumping Station, additional borings and analyses will be performed. A FS of 1.25 was applied to the friction angle as follows:

$$\phi_d \text{ (developed friction angle)} = \tan{-1}\left(\frac{\tan\phi_A}{FS}\right)$$

This developed angle was used to determine K_a and K_p (lateral earth pressure coefficients) as follows:

$$K_a = \tan^2\left(45° - \frac{\phi_d}{2}\right) \text{ and } K_p = \frac{1}{K_a}.$$

Using the resulting shear strengths and net horizontal static water pressure, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetration required for stability was determined as that elevation at which the summation of overturning movements about the bottom of the sheet piling approached zero.

T-Walls.

Steel sheet-pile cutoff. A steel sheet-pile cutoff will be used beneath the T-wall to provide protection against seepage. The stability analysis of the T-wall was based on the following:

Conventional Q shear stability analyses were performed at 1-ft intervals from the bottom of the structure base to the sheet-pile tip, utilizing a FS of 1.3 applied to the soil strength parameters.

The value of R_b at the bottom of the base of the structure was assumed equal to zero.

The net force equals D_a - ($D_p + R_a + R_b + R_p$) and was determined at each increment of depth.

The driving force above the base of the structure and the horizontal hydrostatic load are carried by the structure.

The algebraic difference in the net forces at the top and bottom of each 1-ft interval was used to develop the pressure diagram.

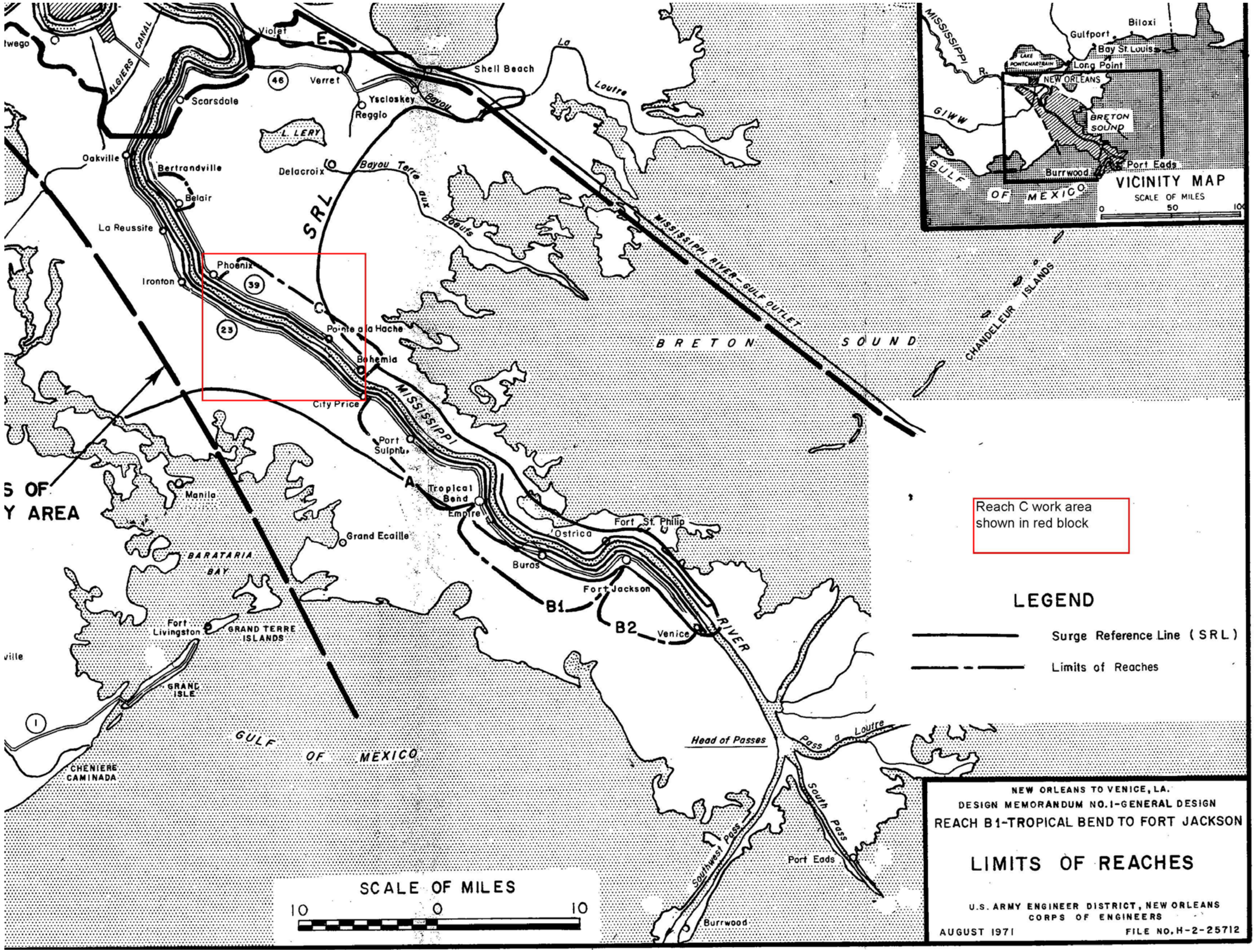
If the algebraic difference is negative, the available horizontal soil resistance is in excess of the unbalanced water load, and the bearing piles are not required to carry any additional lateral load acting on the sheet-pile cutoff.

Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion protection, other than sodding, is considered necessary on the levee slopes along most of the levee alignment. However, foreshore protection will be constructed on the floodside levee toe in the Bay Carrion Crow area from Station 232+00 to Station 263+00 to protect the levee from damages which could occur from waves generated by other than hurricane winds. The foreshore protection will consist of 21 in. of riprap on a 9-in.-thick shell bedding. At the Venice Pumping Station, protection against erosion will consist of 18 in. of riprap over a 9-in.-thick shell bedding.

Reach C Phoenix to Bohemia. The existing 16 miles of interior earth fill will be enlarged and raised from an elevation of approximately 14 ft MSL to a net elevation of 17 ft MSL (Reference 51). Image of area shown on following page.

Geology. The project area is located within the Central Gulf Coastal Plain. More specifically, the area is situated on the deltaic plain of the Mississippi River, a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and its abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from approximately 5 ft along the crests of the natural levees to 0.0 ft MSL in the marshlands between the natural levee ridges. The numerous inland bodies of water range in depth from 1 to 6 ft. The Mississippi River channel in the vicinity of the project area ranges in depth from 70 to 190 ft below mean sea level.

Foundation Conditions. The subsurface consists of Recent deposits that range in depth from approximately 112 ft at the upstream end of the project to about 131 ft at the downstream end. The Recent deposits are underlain by Pleistocene (Prairie Formation) deposits. Generally, the Recent consists of a 6- to 12-ft surface layer of very soft to soft marsh deposits with organic material and peat. The marsh deposits consist generally of clays with organic matter and peat, underlain by interdistributary deposits of very soft to soft clays containing lenses and layers of silt and silty sands. The interdistributary deposits range in thickness from 38 ft in the vicinity of Station 10+00 to about 50 ft at approximate Station 650+00. Underlying the interdistributary clays at elevations ranging from -110.0 ft MLS in the vicinity of Station 10+00, to -119.0 ft MSL near Station 800+00 are medium to stiff prodelta clays. The prodelta clays overlie a thin wedge of nearshore sands with shell and shell fragments which thickens from a minimum of about 2 ft at Station 10+00 to a maximum of about 12 ft near Station 800+00. The entire sequence of Recent sediments is underlain by stiff to very stiff Pleistocene clays at elevations ranging from -112.0 ft at Station 10+00 to -131.0 ft at Station 800+00.


Reach C work area shown in red block
LEGEND
Surge Reference Line (SRL)
Limits of Reaches
VICINITY MAP
SCALE OF MILES
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO.1-GENERAL DESIGN
REACH B1-TROPICAL BEND TO FORT JACKSON
LIMITS OF REACHES
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1971
FILE NO. H-2-25712
SCALE OF MILES
BRETON SOUND
GULF OF MEXICO
MISSISSIPPI RIVER
MISSISSIPPI RIVER-GULF OUTLET
CHANDELEUR ISLANDS
BARATARIA BAY
SRL
Phoenix
Pointe a la Hache
Bohemia
City Price
Port Sulphur
Tropical Bend
Empire
Buras
Fort Jackson
Fort St. Philip
Ostrica
Venice
Head of Passes
Port Eads
Burrwood
Grand Ecaille
Manila
Fort Livingston
GRAND TERRE ISLANDS
GRAND ISLE
CHENIERE CAMINADA
Oakville
Bertrandville
Belair
La Reussite
Ironton
Scarsdale
Violet
Verret
Ysclaskey
Reggio
Delacroix
Shell Beach
L. LERY
ALGIERS CANAL

Field Investigation. Two 5-in.-diameter undisturbed borings approximately 110 ft in depth were made. Nine additional undisturbed borings approximately 100 ft in depth were made. The nine additional undisturbed borings were equally divided between three locations, each location having a levee centerline boring and a boring at the levee toe on each side of the centerline. A total of 31 general type core borings, 1⅞-in.-ID were made. Twenty-four of the general type borings extended approximately 50 ft in depth. The remaining seven borings extended to 80 ft in depth. Two general type borings were made in the recommended borrow area in the Pointe a la Hache Relief Outlet.

Seepage. Approximately 10 ft of clay cover above the sand core will be provided on the floodside of the levee. Due to the relatively short duration of hurricane headwaters, this is considered sufficient to prevent seepage.

Pile Foundations. Not used.

Sliding Stability.

a. *Levees.* Based on varying soil conditions, the Reach C levee was divided into three sub-reaches - Station 0+00 to Station 159+00, Station 159+00 to Station 495+00, and Station 495+00 to Station 834+85.0 (end of project). Undisturbed borings were made at Stations 14+06 (borings 30 CUT, 30 CU, and 30 CUTP), 303+05 (borings 8 CUT, 8 CU, and 8 CUTP), and 687+50 (borings 21 CUT, 21 CU, and 21 CUTP). Stability of the proposed levee sections was investigated for each subreach using soil properties and strengths derived from the appropriate set of undisturbed borings. Stability was determined by the Method of Planes based on a minimum FS of 1.3 with respect to shear strength. Stability was investigated at various depths in the foundation, and FSs with respect to shear strength were determined for various assumed failure planes. Berms on the floodside of the levee are not necessary for levee shear stability, but are provided as a means of dissipating a portion of the wave energy and thus reducing the required levee grade.

b. *Bohemia spillway canal.* Between Stations 801+09.5 and 834+85 at the lower end of Reach C, the spillway canal parallels the levee. Because of this condition, a stability computation was performed in addition to the ones calculated as generally representative of the subreach between Stations 495+00 and 834+85.0. The results shown on plate 42 indicate FS in excess of the required minimum of 1.3. The assigned foundation stratification and design shear strengths were those determined from the set of undisturbed borings at Station 687+50.

c. *Gravity drainage structures.* Five drainage structures have been constructed by local interests along the Reach C alignment. Conventional stability computations were made for three structures, one in each subreach which represented the most critical condition in the respective subreach. The stability analyses were based on the foundation stratifications, design shear strengths, and water conditions appropriate for the respective sub-reaches. In addition, a surcharge was added to the passive wedges. The surcharge was computed by dividing the total weight of concrete and sacked riprap above the soil by an assumed failure width. Routine stability computations were made assuming the culverts were non-existent, which is a conservative assumption. Two of the structures, one at

Station 89+50 and the other at Station 425+50, satisfied the requirement for a 1.3 FS. The third structure, at Station 548+60, did not meet the 1.3 criteria by conventional stability computations, assuming an infinite width. It was necessary, therefore, to perform a mass stability analysis at this location. The result of this analysis indicates a FS in excess of 1.3 with respect to shear stability (see plate 45).

d. Pumping Stations. Two pumping stations have been recently constructed along the project alignment by local interests. One is located at Station 241+77 near Bellevue, the other at Station 551+38 near Pointe a la Hache. Stability of the levee sections adjacent to the connecting cantilever I-type floodwalls was analyzed. The most critical condition was found at the Pointe a la Hache Pumping Station. Computations for this location indicate a FS of 1.42, assuming levee failure into the discharge canal. The assigned foundation stratification and design shear strengths used in the above computations are those appropriate to the subreaches containing the pumping stations.

I-Walls. The floodwalls between Stations 240+72 and 242+82 and Stations 550+33 and 552+43 are I-type cantilever sheet-pile walls consisting of P2-27 steel sheet pile capped with concrete. The sheet piling extends from elevation 13.0 to -20.0 ft NGVD, and the concrete cap is provided between elevation 10.0 and 20.0 ft NGVD. In order to provide a seepage cutoff, the sheet piling was extended below the elevation required for stability purposes. Seepage cutoff is also provided under the discharge basin side and backwalls to obtain a continuous diaphragm. A wave berm has been constructed along the floodwall alignment to dissipate hurricane wave forces on the floodwalls. The floodwalls are designed to withstand loading from an 8-ft broken wave.

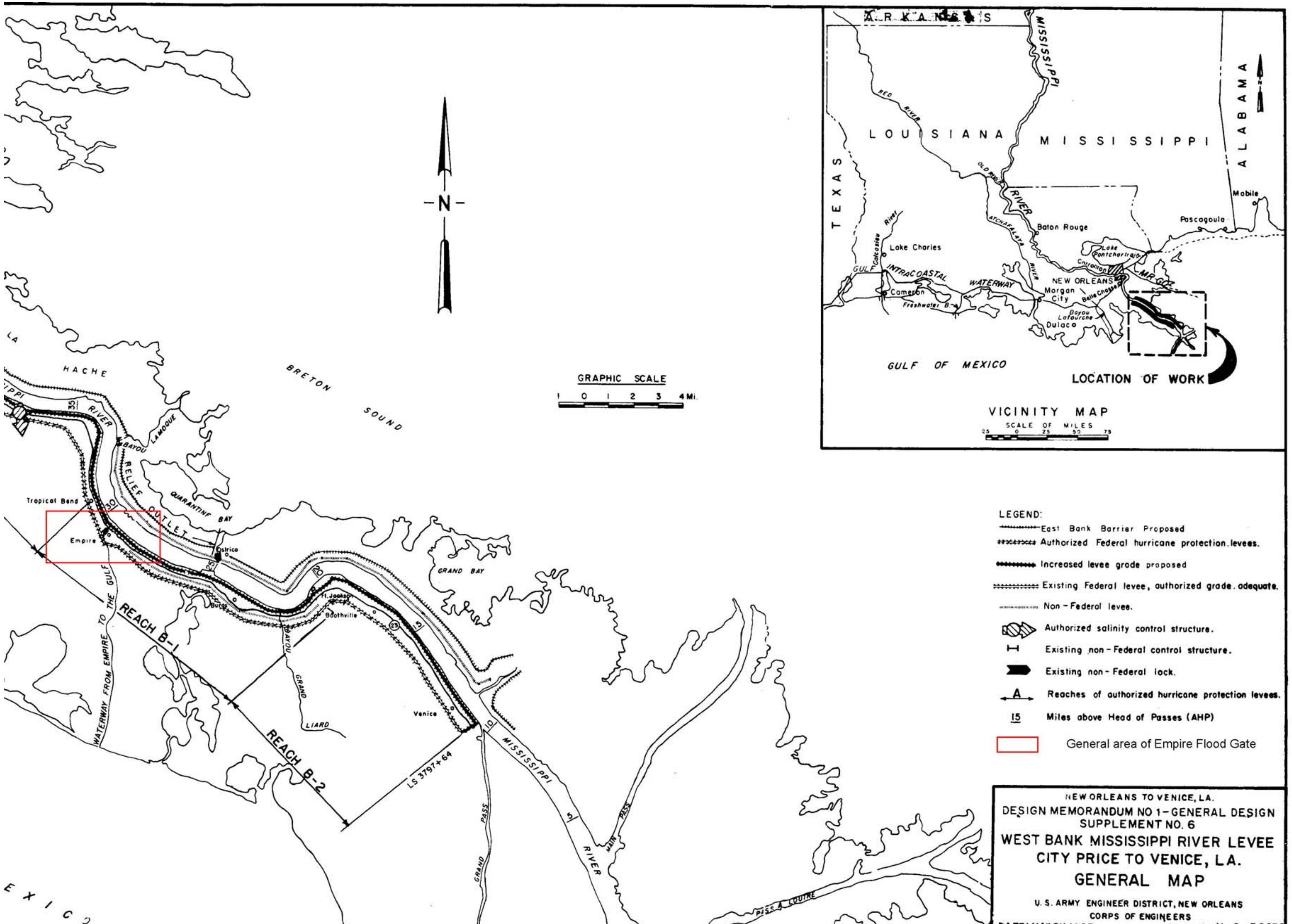
T-Walls. Not used.

Erosion Protection. Because of the relatively short duration of hurricane flood stages and the resistant nature of the clayey soils, erosion protection other than sodding is not considered necessary along the major length of the levee. In the vicinity of the two pumping stations, adequate erosion protection is provided by riprap along the floodside of the cantilever I-type floodwalls. Erosion protection at the existing gravity drainage structures consists of sacked concrete riprap.

Empire Floodgate New Orleans to Venice (Reference 69). Image of area shown on following page.

Geology. The geology within the general area of the Empire Floodgate is presented in Reference 50.

Project Foundation Conditions. The foundation soils consist predominantly of Recent backswamp clays having soft to medium consistencies, and extending to depths of approximately 90 feet below the natural ground surface. The Recent clays contain 3- to 5-ft thick layers of silts and sands approximate at elevations -20, -30, and -50 ft. The 5- to 10-ft thick clay layer extending from the ground surface contains organic matter with some peat.



VICINITY MAP
LOCATION OF WORK
GULF OF MEXICO
LOUISIANA
MISSISSIPPI
ALABAMA
TEXAS
ARKANSAS
NEW ORLEANS
Baton Rouge
Lake Charles
Morgan City
Pascagoula
Mobile
GRAPHIC SCALE
BRETON SOUND
GRAND BAY
QUARANTINE BAY
Tropical Bend
Empire
Ostrica
Boothville
Venice
REACH B-1
REACH B-2
LS 3797+64
WATERWAY FROM EMPIRE TO THE GULF
MISSISSIPPI RIVER
LEGEND:
East Bank Barrier Proposed
Authorized Federal hurricane protection levees.
Increased levee grade proposed
Existing Federal levee, authorized grade adequate.
Non-Federal levee.
Authorized salinity control structure.
Existing non-Federal control structure.
Existing non-Federal lock.
Reaches of authorized hurricane protection levees.
Miles above Head of Passes (AHP)
General area of Empire Flood Gate
NEW ORLEANS TO VENICE, LA.
DESIGN MEMORANDUM NO 1-GENERAL DESIGN
SUPPLEMENT NO. 6
WEST BANK MISSISSIPPI RIVER LEVEE
CITY PRICE TO VENICE, L.A.
GENERAL MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE: MARCH 1987
FILE NO. H-2-30032

PLATE B-1

Field Exploration. One 5-in.-diameter undisturbed boring and four general type disturbed core borings were made for this investigation. The undisturbed and general-type borings extended in depth to approximate elevations of -90 and -80 ft, respectively.

Unwatering and Hydrostatic Pressure Relief During Construction. In order to construct the floodgate and floodwalls in the dry and to insure stability of the structure excavation during construction, hydrostatic pressure relief was to be provided in the silt and sand layers within the soil foundation. The pressure relief was to be accomplished by vertical sand drains and well-points. To allow time for pore pressure relief, the rate of unwatering of the working area was to be maintained at a maximum of 2 feet per day for the first 10 ft, and 1 ft per day thereafter until completely unwatered. Temporary construction piezometers were to be installed in the pervious layers to monitor the pore pressure during the unwatering and pressure relief period. After the structure was complete and operating, the sand drains were expected to discharge into the shell backfill and provide a degree of permanent pressure relief. Conventional sumps and pumps were to maintain the area free of surface water during construction.

Slope Stability.

a. *Construction Slopes.* The stability of the excavation, dike and closures, existing first lift levee, and berm distances were determined by the Method of Planes based on a minimum FS of 1.3 with respect to shear strength and the Q design shear strengths. Stability was investigated at various depths in the foundation, and FS with respect to shear strength were determined for various assumed failure planes. The relief facilities were to provide the required pressure reduction in the pervious layers for stability.

b. *Final Slopes.* The Q stability governed for design of the final slopes. The final slopes were to be constructed by clamshell backfilling. In the vicinity of the structure, the inclinations of the rebuilt slopes were determined by the requirement that the length of the floodwalls be as short as possible without sacrificing stability of the tie-in levee into the inlet and outlet channels. The remaining rebuilt slopes were designed to be stable with a minimum of backfilling.

Cantilevered I-Wall. The results of tidal hydraulic analyses indicated that the I-wall would be subjected to the pressure and forces imparted by breaking waves. In the stability analyses, the dynamic wave effect was applied as a line force acting through the centroid of the dynamic wave pressure distribution diagram. The static water pressure diagram resulting from wave action was considered effective only to the top of the impervious clay, inasmuch as the period of time the wave would exist was considered too short to allow water pressure to become effective in the impervious clays. The stability and required penetration of the steel sheet piling below the fill surface were determined by the Method of Planes. The long-term S shear strengths (C = 0) governed for design. A FS of 1.25 was applied to the friction angle. This developed angle was used to determine K_a and K_p lateral earth pressure coefficient values.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. The depths of penetration required for stability were determined as those where the summation of moments was equal to zero.

Control Structures and T-Walls.

a. *Steel sheet-pile cutoff.* A steel sheet-pile cutoff was to be used beneath the floodgate and T-walls to provide protection against hazardous seepage. The net pressure diagram along the sheet-pile cutoff was determined as follows:

(1) Conventional stability analysis by the Method of Planes, utilizing a FS of 1.3 incorporated in the soil strength parameters, was performed to determine the stability against rotational failure. The analysis was performed at 1-ft vertical intervals with the active wedge located at the floodside edge of the structure and the passive wedge located at the protected side edge of the structure.

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

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

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

(5) Considering driving (D_A) positive and all resistance negative (Dp, Rp, R_E, and R_A) in the expression $D = D_A - D_p - R_p - R_B - R_A$, using the Method of Planes stability analyses, $\sum D$ was determined by assuming failure at the bottom of the base of the structure and at each foot in depth thereafter. The value of the algebraic difference in $\sum D$, between 1-ft intervals, was used to develop the pressure diagram. If the incremental difference were negative, the pressure diagram indicated an available horizontal resistance in excess of that required, and if the incremental difference were positive, the pressure diagram indicated an unbalanced horizontal pressure in excess of the available soil resistance. It was considered that such an excess must be carried by the sheet-pile cutoff.

(6) The net pressure diagrams indicated that the total available horizontal resistance was in excess of the total horizontal water load. Therefore, the analyses indicated that the bearing piles were not required to carry any additional lateral load acting on the sheet-pile cutoff.

b. *Bearing pile foundations.*

(7) The floodgate and T-walls were to be supported by piling, battered as required, to provide stability against the unbalanced lateral water loads. The inverted T-type floodwalls were to be used in lieu of I-type floodwalls where the height of the I-wall above ground and the magnitude of the dynamic wave force rendered the I-type floodwall impracticable. In compression, a FS of 1.75 was applied to the shear strengths, and a lateral earth pressure coefficient $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths and a coefficient $K_0 = 0.7$ were used. Settlement of the piles due to consolidation was not indicated to be a problem since the major loads were caused by hurricane water heads of insufficient duration for consolidation of the foundation clays to ensue.

(8) During construction, three 12-in.-diameter class B untreated timber piles of different lengths were to be driven. The intermediate pile was to be tested in compression. If test results showed that the pile could safely carry twice the design load, the pile would be tested in tension. If the intermediate pile failed before the required capacity was attained in compression, the long pile would be tested in compression and in tension. If the intermediate pile safely carried compression loads significantly in excess of that required, the short pile would be tested in compression and in tension. Pile test loads were to be 15 tons in tension and 40 tons in compression.

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

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

e. *Erosion protection.* To protect against loss of channel and backfill material due to erosion and subsequent undermining of the floodgate and floodwalls, 2 ft of riprap on a minimum 1-ft blanket of clamshell was to be provided.

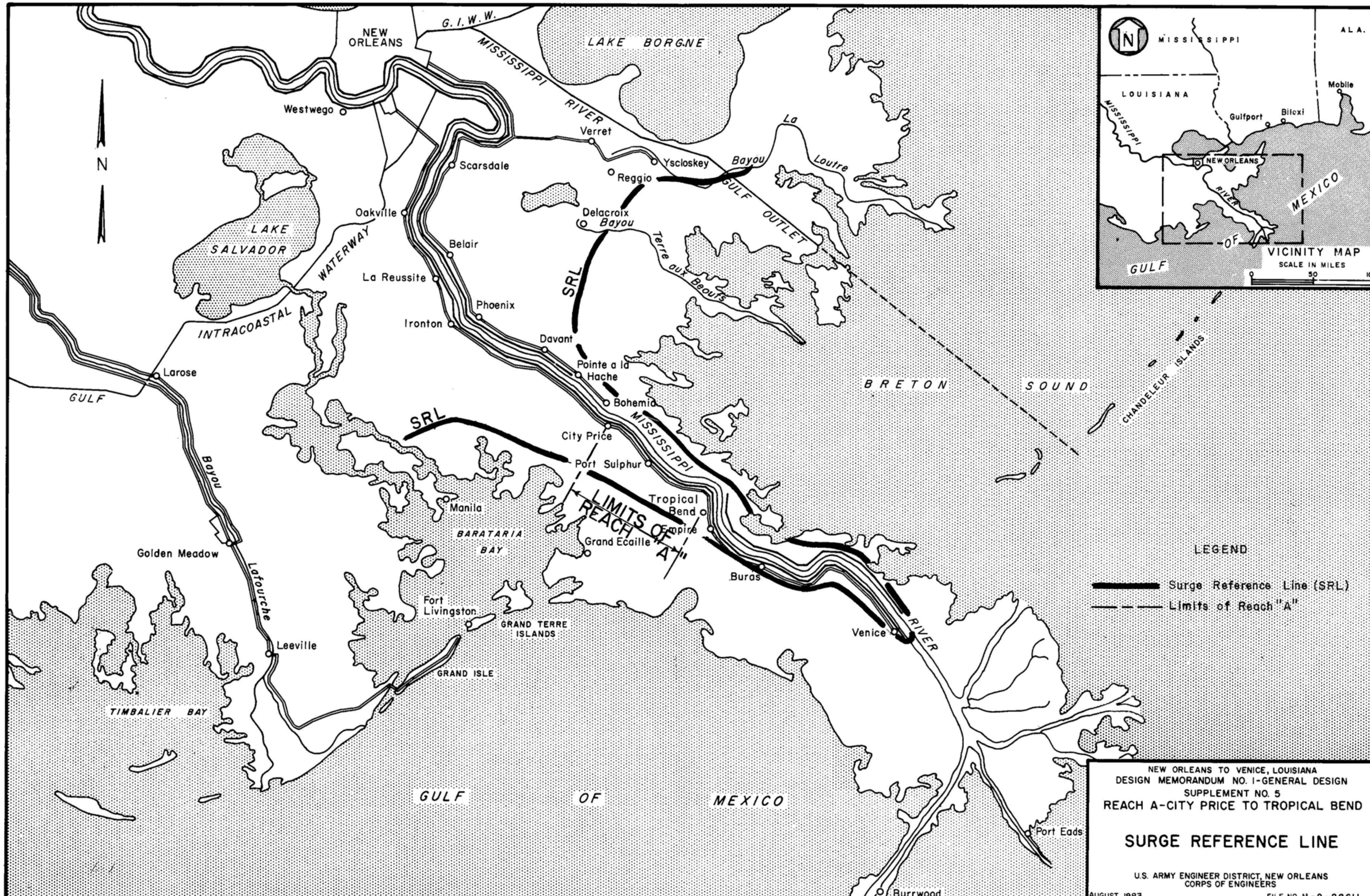
f. *Settlement observations.* Settlement observations were to be made along the structure and floodwalls promptly after construction and yearly thereafter.

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

Structural.

Reach A - City Price to Tropical Bend (Reference 52). Image of area shown on following page.

General. The project plan consists of protective levees and appurtenant features. The levee system is approximately 12.8 miles in length, with a net elevation ranging from 12.5 ft NGVD at the beginning near City Price to 14.5 ft NGVD at the lower end near Tropical Bend. Structural features include floodwalls at the City Price drainage structure, Hayes Canal pumping station, Freeport Sulphur unloading dock, and Gainard Woods pumping station. The pumping station discharge pipes will pass through the floodwall, but will be modified to prevent potential backflow during high outside stages.



NEW ORLEANS TO VENICE, LOUISIANA
DESIGN MEMORANDUM NO. 1-GENERAL DESIGN
SUPPLEMENT NO. 5
REACH A-CITY PRICE TO TROPICAL BEND
SURGE REFERENCE LINE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1983
FILE NO. H-2-28611
LEGEND
Surge Reference Line (SRL)
Limits of Reach "A"
VICINITY MAP
SCALE IN MILES
0
50
100
MISSISSIPPI
ALA.
LOUISIANA
Mobile
Gulfport
Biloxi
NEW ORLEANS
GULF OF MEXICO
N
NEW ORLEANS
G.I.W.W.
LAKE BORGNE
MISSISSIPPI RIVER GULF OUTLET
Westwego
Verret
Yscloskey
Reggio
Bayou La Loutre
Scarsdale
Oakville
Delacroix
Bayou Terre aux Beoufs
Belair
La Reussite
Phoenix
Ironton
Davant
Pointe a la Hache
Bohemia
LAKE SALVADOR
INTRACOASTAL WATERWAY
Larose
GULF
SRL
City Price
Port Sulphur
LIMITS OF REACH "A"
Tropical Bend
Empire
Grand Ecaille
Buras
Venice
MISSISSIPPI RIVER
BRETON SOUND
CHANDELEUR ISLANDS
Manila
BARATARIA BAY
Golden Meadow
Bayou Lafourche
Fort Livingston
GRAND TERRE ISLANDS
Leeville
GRAND ISLE
TIMBALIER BAY
GULF OF MEXICO
Port Eads
Burrwood

PLATE A-11

Structure Elevations		
Location	**Top of Wall ft NGVD**	**Design Water Surface ft NGVD**
City Price Floodwall	12.5 16.0	8.9 8.9
Hayes Canal Pumping Station Floodwall	16.0	9.2
Freeport Sulphur Floodwall	16.0	9.2
Gainard Woods Pumping Station Floodwall	17.0	9.6

I-Type Floodwalls. I-type floodwalls are constructed at Hayes Canal and Gainard Woods Pumping Stations, City Price, and at Freeport Sulphur. The load case which controls design is water load to the still-water level (see above) plus the wave loads computed from guidelines outlined in *Shore Protection Manual*, Volume II, 1971. The required FS is 1.5, S-Case soil conditions.

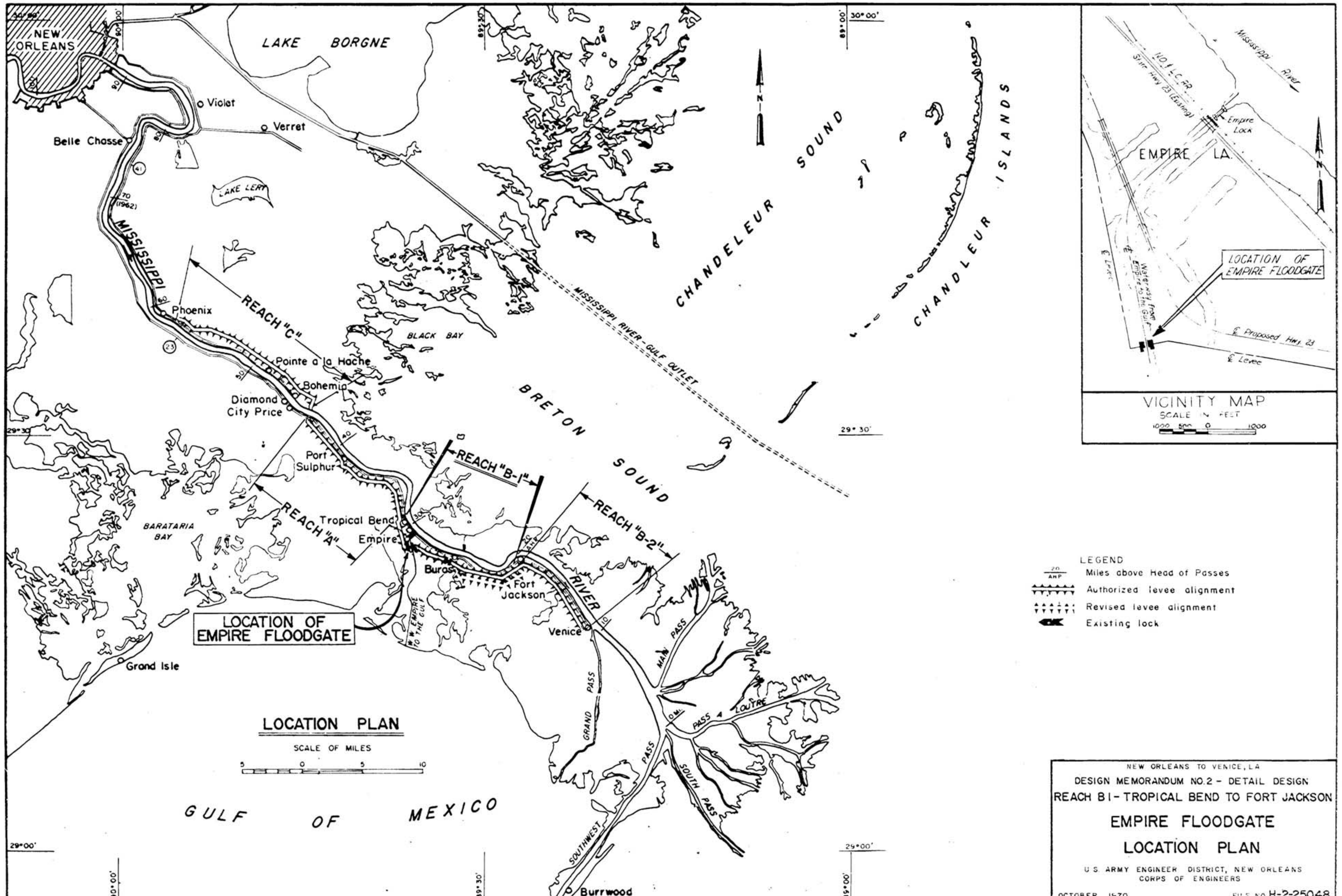
T-Type Floodwall. T-type floodwalls are constructed at the Hayes Canal Pumping Station and at the Gainard Woods Pumping Station. Load cases for the T-walls are as follows:

Load Case		Symbol
I	Dead load	DL & WL
II	Water load and impervious uplift	UI
III	Pervious uplift	UP
IV	Wave load	WL

For pile design, no load factors were used (working stress) and the following load cases were considered:

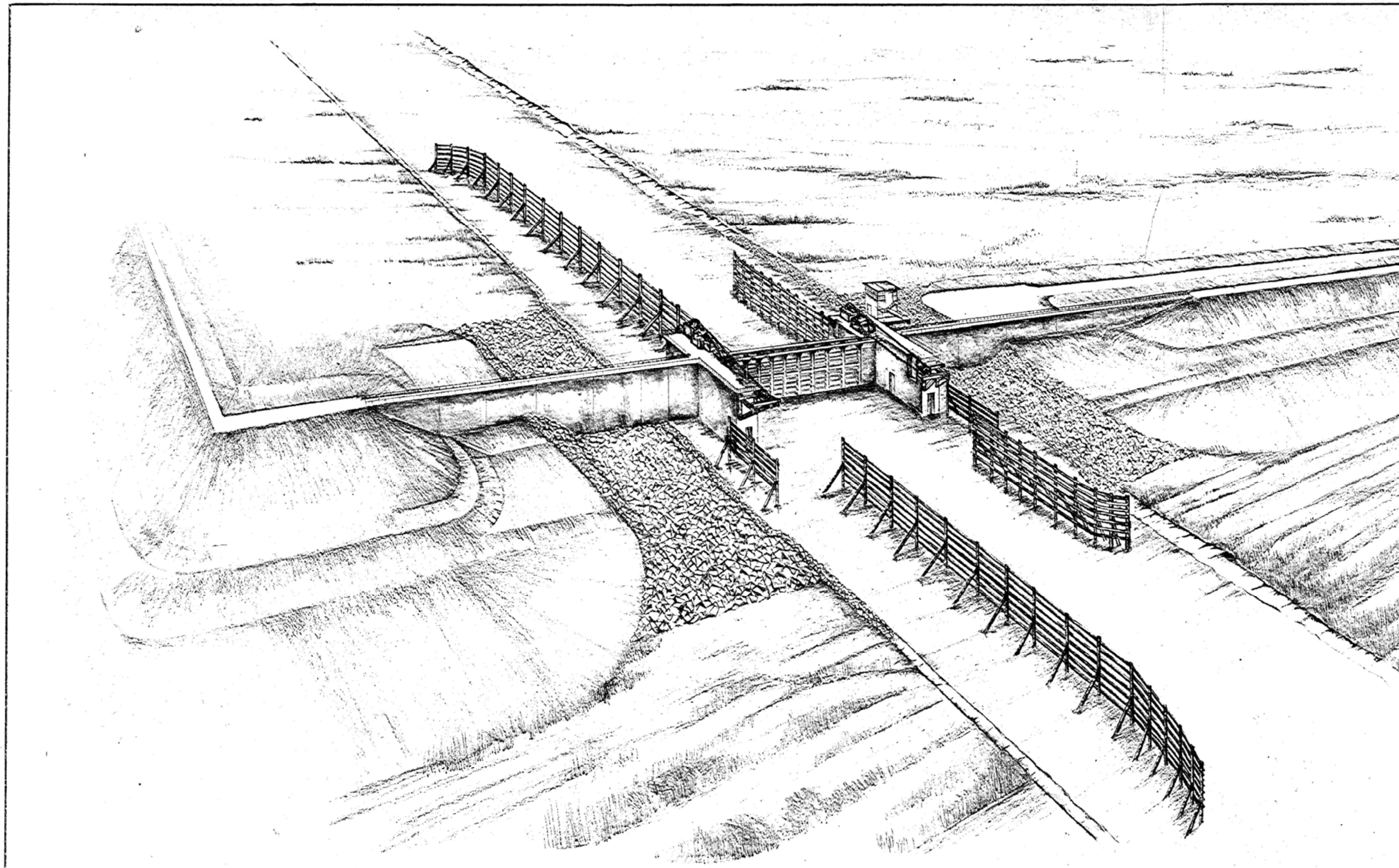
No.	LD Combination
1	DL + WL + UI
2	DL + WL + UP
3	0.75 (DL + WL + UI + WL)
4	0.75 (DL + WL + UP + WL)

Reach B1 Tropical Bend to Fort Jackson – Floodgate at Empire - (Reference 54). Images of area shown on following pages.



LAKE BORGNE
NEW ORLEANS
Violet
Verret
Belle Chasse
LAKE LERY
MISSISSIPPI
RIVER
Phoenix
REACH "C"
Pointe a la Hache
Bohemia
Diamond City Price
BLACK BAY
Port Sulphur
REACH "B-1"
REACH "B-2"
REACH "A"
Tropical Bend
Empire
Buras
Fort Jackson
Venice
BARATARIA BAY
LOCATION OF EMPIRE FLOODGATE
Grand Isle
CHANDELEUR SOUND
CHANDELEUR ISLANDS
BRETON SOUND
MISSISSIPPI RIVER - GULF OUTLET
MAIN PASS
PASS A LOUTRE
GRAND PASS
SOUTHWEST PASS
SOUTH PASS
Burrwood
GULF OF MEXICO
LOCATION PLAN
SCALE OF MILES
VICINITY MAP
SCALE IN FEET
EMPIRE LA.
Empire Lock
Mississippi River
LOCATION OF EMPIRE FLOODGATE
LEGEND
Miles above Head of Passes
Authorized levee alignment
Revised levee alignment
Existing lock
NEW ORLEANS TO VENICE, LA
DESIGN MEMORANDUM NO. 2 - DETAIL DESIGN
REACH B1 - TROPICAL BEND TO FORT JACKSON
EMPIRE FLOODGATE
LOCATION PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
OCTOBER 1970
FILE NO H-2-25048

PLATE I-1



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

PLATE I-2

General. The floodgate consists of a reinforced concrete U-frame gate bay with a clear opening of 84 ft and sill elevation of -14.0 ft NGVD. A steel gate is hinged at the bottom of the structure. The entire structure is supported on untreated timber piling. The total structure width is 106 ft and the top of the walls are at elevation 15.0 ft NGVD. A control house is provided above one wall for operation of the gate, and needle dams are provided for unwatering the gate while the gate is in the closed position.

A 300-ft timber guide wall and a 100-ft long timber fender are located on each side of the gate structure. The guide wall is on the west side of the channel and the fender is on the east side of the channel.

An inverted T-type reinforced concrete floodwall abuts the structure wall and extends for a distance of 150 ft on each side of the structure, at which point I-type reinforced concrete floodwalls extend an additional 105 ft on each side of the structure. The top of the floodwalls will be at elevation 15.0 ft MSL.

Design Water Elevations (ft MSL)		
	Gulf Side	**Landside**
Direct head from hurricane	+12.1	+2.0
Reverse head from hurricane	-2.0	+6.3
Direct head for maintenance	+5.0	-1.0
Reverse head for maintenance	-2.0	+5.0

Structure Elevations (ft MSL)	
Top of Wall	**+15.0**
Top of timber guide walls and fenders	+ 9.5
Top of sill	-17.5/-14.0
Centerline of gate hinges	-15.54
Centerline of hoist wildcat	+17.75
Centerline of cwt, wildcats	+15.0/+21.0
Centerline of needle girders	+5.0
Bottom of channel outside limits of riprap	-12.0

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

Lateral pressures, (lb/sq ft/ft)	**Submerged**	**Saturated**
Earth	25.8	54.0
Shell	13.5	41.6
Riprap	28.4	56.5

Uniform live loads	**lb/sq ft**
Walkways and stairs	100

Control building floor	200
Control building roof	20

Wind loads. Wind loads on exposed vertical surfaces and projected area of sloped surfaces. (Allowable stresses increased one-third) - 30 lb/sq ft.

Wave loads. Net wave pressures have been computed from the hurricane design wave data in accordance with recommendations of "Shore Protection, Planning and Design," Technical Report No. 4, Third Edition, 1966, by the Coastal Engineering Research Center, Corps of Engineers. The hurricane design wave was assumed to approach the structure at a 90° angle.

Allowable Working Stresses - The allowable working stresses for structural steel and concrete are in accordance with those recommended in "Working Stresses for Structural Design," EM 1110-1-2101, dated 1 November 1963.

Application of Working Stresses.

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.

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 with a general allowance of an increase of 33 1/3 percent over allowable stresses:

Wind loads
Wave loads

Pile Foundation and Stability Analysis. The pile foundations were designed in accordance with EM 1110-2-2906, July 1969, "Design of Pile Structures and Foundations." Computed pile loads were determined from the rational method of pile foundation analysis (method developed by A. Hrennikoff).

Design loading conditions for the Concrete U-Frame Gate structure.

- Case I – Operating conditions. Maximum direct head (hurricane). Gate closed; floodside water at elevation +12.1 ft NGVD, protected side water at elevation +2.0 ft NGVD; uplift with sheet-pile cutoff considered impervious; no wave force
- Case II – Same as Case I, except uplift with sheet-pile cutoff considered pervious

- Case III – Maximum direct head with wave forces (hurricane). Gate closed; floodside water at elevation +12.1 ft NGVD, protected side water at elevation +2.0; uplift with sheet-pile cutoff considered impervious
- Case IV – Same as Case III except uplift with sheet-pile cutoff considered pervious
- Case V – Maximum reverse head. Gate closed; floodside water at elevation -2.0 ft NGVD, protected side water at elevation +6.3; uplift with sheet-pile cutoff considered impervious
- Case VI – Same as Case V except uplift with sheet-pile cutoff considered pervious.

Non-operating conditions

- Case VII – Gate dewatered. Gate removed; needle beams and girders in place; floodside water at elevation +5.0ft NGVD; protected side water at elevation +5.0 ft NGVD; full uplift
- Case VIII – Construction condition. Gate closed; no uplift.

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

West Bank Mississippi River Levee – City Price to Venice (Reference 69).

General. The project plan provides for enlargement of the west bank Mississippi River levees and construction of levee setbacks and floodwalls. The only structure in the project is the existing Empire Lock at Empire, LA. It was determined that no modification was necessary to the lock gates. Wave overtopping would be allowed; however, the 30 ft of existing I-wall on both sides of the structure was found to be inadequate to withstand the projected hurricane wave force if capped to project height. The length of the existing sheet piling is inadequate. For structural and constructability reasons, the existing sheet piling shall be removed and replaced with adequate lengths of new PZ-27 sheet piling.

Basic data relevant to water surface elevations, structure elevations, and dimensions are summarized below:

Structure Elevations					
Top of Wall ft NGVD	**Design swl Elevation ft NGVD**	**Wave Loads**			
		fm, lb/sq ft	**hc, ft**	**psw, lb/sq ft**	**ds, ft**
21.5	13.5	336	7.3	448	0

Wave forces were computed from guidelines outlined in *Shore Protection Manual*, Volume II, 1971.

Strength Design Criteria. The concrete structures are designed in accordance with ETL 1110-2-265, "Strength Design Criteria for Reinforced Concrete in Hydraulic Structures," dated 15 September 1981, and ACI 318-77, "Building Code Requirements for Reinforced Concrete." Design values used are listed below:

f'_c	3,000 psi
fy (reinforcement)	40,000 psi
P	0.25 P_b
Pmin (flexure)	200/fy or 1/3 greater than required by analysis
minimum temperature steel	0.002 bt - (half in each face)
v_c	$2\ (f'_c)^{½}$
Sheet pile	ASTM-A328 (19,500 psi allowable)

Design of I-Type Floodwalls. The load case which controls design is water load to the still-water level (see above) plus the wave loads computed from the information given. A FS equal to 1.5 was used in design of the sheet piling assuming a cantilever design under an S-Case soil condition.

Reach B2 Fort Jackson to Venice (Reference 50).

General. The structural features of the project consist of I- and T-walls at the Venice Pumping Station. The Venice Pumping Station is located on the protected side of an existing back levee with discharge pipes passing through the levee just below the road surface on the levee crown. To provide continuous protection at minimum cost, the new levee ties into the existing back levee approximately 100 ft to each side of the discharge pipe crossings. Inverted T-type floodwall in the existing levee and I- type floodwall in the tie-in levees were constructed to provide the continuous line of protection.

Criteria for Structural Design. The structural design of the floodwall complies with standard engineering practice and criteria set forth in Engineering Manuals for Civil Works Construction published by the Office of the Chief of Engineers, Wave forces were computed using guidelines outlined in Technical Report No. 4, third edition, 1966, "Shore Protection Planning and Design," published by the U.S. Army Coastal Engineering Research Center with the exception that breaking waves were not considered to act on the total structures (see WES Research Report H-68-2, dated September 1968, "Shock Pressures Caused by Waves Breaking Against Coastal Structures").

Basic Data

Still-water level (swl), floodside	11.5
Assumed water elevation landside of floodwall	-5.0 ft NGVD
Unit weight of water	62.5 lb/cu ft
Unit weight of reinforced concrete	150.0 lb/cu ft

Allowable Working Stresses. The a1lowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM 1110-1-2101, dated 1 November 1963, and amendment 1, dated 14 April 1965.

I-Type Floodwall. The I-wall consists of sheet piling driven into the final levee sections and capped with concrete. For design of the I-wall, two loading cases were considered:

- Case I – Static water to the swl, elevation 11.5 ft NGVD; 1.5 FS in the soil; and no wave force
- Case II – Static water to swl, elevation 11.5 ft NGVD; 1.25 FS in the soil; and wave load from non-breaking wave.

T- Type Floodwall. A reinforced concrete T-wal1 section will be supported by battered prestressed concrete piles driven into the levee section. The sheet-pile cutoff wall below the T-wall base is assumed to be self-supporting and, therefore, does not cause or resist any load on the T-wall. The T-wall was designed assuming the following loading cases:

- Case I – Static water to swl, elevation 11.5 ft NGVD; no wave force; and impervious sheet-pile cutoff.
- Case II – Static water to swl, elevation 11.5 ft NGVD; no wave force; and pervious sheet-pile cutoff.
- Case III – Static water to swl, elevation 11.5 ft NGVD; wave load from non-breaking wave; impervious sheet-pile cutoff; and 33 1/3 percent increase in allowable stresses.
- Case I V – Static water to swl, elevation 11.5 ft NGVD; wave load from non-breaking wave; pervious sheet-pile cutoff; and 33 1/3 percent increase in allowable stresses.

Reach C Phoenix to Bohemia (Reference 51).

General. Two pumping stations, one near Bellevue, the other near Pointe a la Hache, were recently constructed by local interests; these include provisions for protection from design hurricane tides. A continuous protective system is provided by the floodwalls between the discharge basins and the adjacent levees; the discharge basin sidewalls; and the backwalls of the discharge basins. The critical structure loadings resulting from design hurricane induced stage differentials are transmitted from the discharge basin backwalls through longitudinal shear walls and ultimately distributed to all the structural components of the pumping stations. Therefore, virtually the entire pumping station is used to resist these loads.

In addition, floodwalls between Stations 240+72 and 242+82 and Stations 550+33 and 552+43 are I-type cantilever sheet-pile walls consisting of PZ-27 steel sheet pile capped with concrete. The sheet piling extends from elevations 13.0 to -20.0 ft NGVD, and the concrete cap is provided between elevations 10.0 and 20.0 ft NGVD. In order to provide a seepage cutoff, the sheet piling was extended below the elevation required for stability purposes. Seepage cutoff is also provided under the discharge basin side and backwalls to obtain a continuous diaphragm. A wave berm has been constructed along the floodwall a1inement to dissipate hurricane wave forces on the floodwalls. The floodwalls are designed to withstand loading from an 8-ft broken wave.

Structural Design Criteria. There are no design analyses prepared by the Corps for these structures. However, since the pumping stations and connecting floodwalls are an integral part of the Reach C hurricane protective system, the structural designs prepared by local interests were coordinated with and approved by the Corps, New Orleans District.

Sources of Construction Materials.

Sheet Pile. Generally, the sheet-pile sections specified during advertisement were used for construction. However, sheet-pile section substitution conforming to the minimum required section modulus was allowed. Below is a table of sheet-pile sections for New Orleans to Venice.

New Orleans to Venice	**DM**
West Bank	
St. Jude to City Price	
Diamond Pump Station Tie-In	**
Reach A (City Price to Empire)	
TN Gas Pipeline	PZ-22
Hayes Pump Station Tie-In	unknown cold-rolled sheet pile
Gainard Woods Pump Station Tie-In, Upstream	Frodingham*
Gainard Woods Pump Station Tie-In, Downstream	PZ-22*
Homeplace Marina	unknown cold-rolled sheet pile
Reach B1 (Empire (Tropical Bend) to Ft. Jackson)	
Empire Floodgate Tie-In	PZ-32
Sunrise Pump Station Tie-In	PZ-27*
Bayou Grand Liard	PZ_27*
Reach B2 (Ft. Jackson to Venice)	
Duvic Pump Station Tie-In	unknown cold-rolled sheet pile
East Bank	
Reach C (Phoenix to Bohemia)	
Point a la Hache Pump Station Tie-In	**
Bellview Pump Station Tie-In	**
* As advertised – Not confirmed as-built. ** Information not located at the time of publication.	

Levee material.

Sources of Borrow (Reach A City Price to Tropical Bend). Sand fill will be pumped from the river and stockpiled in the batture to be hauled to its final location. The clay will be hauled from local borrow pits. There are three primary sources of borrow for constructing the levees. Two sites are located within the protected area of Reach A and a third just north of City Price.

Sources of Construction Materials (Reach B1, Tropical Bend to Fort Jackson). Since the levees will be constructed primarily of hydraulic fill with sand and shell core, building materials should present no problems. Hydraulic fill can be pumped from areas immediately adjacent to the proposed alignment; sand can be secured from the Mississippi River nearby; and shell, aggregate, and riprap can be barged and hauled in as required. Suitable materials for topping out the levees can be obtained from the existing earth-fill levee.

Sources of Borrow (Reach B2 – Fort Jackson to Venice, Louisiana). Reach B2 will consist of a sand core hydraulic clay fill levee. A sand core trench will be excavated. Material excavated from the sand core trench will be spoiled in spoil and pending area No. 1, and in the temporary area diked off in spoil and pending area No. 3. Sand will then be pumped from the Mississippi River borrow areas, into the sand core trench and retaining dike base area. Sand will be pumped to elevations that will provide sufficient material for shapeup of the sand core and retaining dike base. A floodside hydraulic clay fill retaining dike will then be constructed from adjacent borrow. Hydraulic clay fill from the clay borrow areas, which will be stripped of the upper 10 ft of poor quality cover material, will then be pumped between the existing back levee and the floodside retaining dike over the shaped sand core fill. When the hydraulic clay fill has sufficiently dried, approximately 2 years after placement, undisturbed borings and shear tests will be made to more accurately design the final levee sections. Where a second lift is not required, the hydraulic clay fill will be shaped to the net section plus some overbuild to compensate for settlement. After the major settlement is virtually complete, approximately 1 year after the first shaping, the levee will be reshaped and the back levee degraded and used as topping material to overbuild the net levee section to allow for any additional settlement. A second hydraulic clay fill lift will be provided where it is anticipated that sufficient material will not be available for the first shaping. Shapeups following the second lift will be the same as those previously described. It is estimated that ultimately, due to settlement, a clay cover of at least 10 ft will be provided on the floodside slope of the levee, including the wave berm.

Sources of Borrow Materials (Reach C, Phoenix to Bohemia). The levee will be enlarged primarily with hauled fill obtained from the Pointe a la Hache Relief Outlet. In addition, excess material above the levee design section will be used for construction of the final levee section.

Source of Borrow (West Bank MRL, City Price to Venice, LA). The hurricane protection levee will consist of semi-compacted clay fill in the levee embankment and uncompacted fill in the berms. The borrow material will be obtained almost entirely from the east bank batture directly across from the construction area; this area will be under water when the borrow material is removed. The material will be barged across the river to the construction site. A small portion of material will come from degrading the existing MR&T levee in setback areas.

As-Built Criteria – Construction Documents

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation).

DACW29-99-C-0052. Narrative Completion Report, New Orleans to Venice, LA, Reach A, Vicinity of Port Sulphur Hurricane Protection Levee, B/L 238+00.4 to 298+00, Final Levee Enlargement & Freeport Canal Closure 2nd Lift, Plaquemines Parish, LA.

Reviewed as-builts; no major modifications or changes were found.

DACW29-96-C-0030. New Orleans to Venice, LA, Reach A and B1, Hurricane Protection Levee, (Homeplace to Empire Floodgate) B/L Station 0+00 to B/L Station 467+00, Levee Enlargement, Plaquemines Parish, LA.

Reviewed narrative completion report; no applicable modifications or changes were found.

DACW29-98-C-0039. New Orleans to Venice, LA, Reach A, Hurricane Protection Levee, Hayes Pumping Station to Port Sulphur, Second Enlargement, Plaquemines Parish, LA.

Reviewed narrative completion report and modification log report. No applicable modifications or changes were found. Contractor did provide his own borrow pit for uncompacted fill material.

DACW29-01-C-0025. Hurricane Protection Project, New Orleans to Venice, LA, Reach B1, Foreshore Dike at Empire, Plaquemines Parish, LA.

Reviewed modification log report and modification documents. The area of the West Harbor Dike between WH C/L Stations 0+00 and 5+13.95 was realigned so that it could be constructed using hauling equipment rather than floating plant.

Inspection During Original Construction, QA/QC, State What Records Are Available.

See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

DACW29-96-C-0030. New Orleans to Venice, LA, Reach A and B1, Hurricane Protection Levee, (Homeplace to Empire Floodgate) B/L Station 0+00 to B/L Station 467+00, Levee Enlargement, Plaquemines Parish, LA.

Attached are moisture analysis reports, percent complete, borrow pit elevations, and daily trucking reports.

DACW29-98-C-0039. New Orleans to Venice, LA, Reach A, Hurricane Protection Levee, Hayes Pumping Station to Port Sulphur, Second Enlargement, Plaquemines Parish, LA.

Attached are records of preparatory inspections/meetings.

DACW29-99-C-0052. Narrative Completion Report, New Orleans to Venice, LA, Reach A, Vicinity of Port Sulphur Hurricane Protection Levee, B/L 238+00.4 to 298+00, Final Levee Enlargement & Freeport Canal Closure 2nd Lift, Plaquemines Parish, LA.

Records of preparatory inspections/meetings were found.

Inspection and Maintenance of Original Construction.

Annual Compliance Inspection (i.e., trees, etc.).

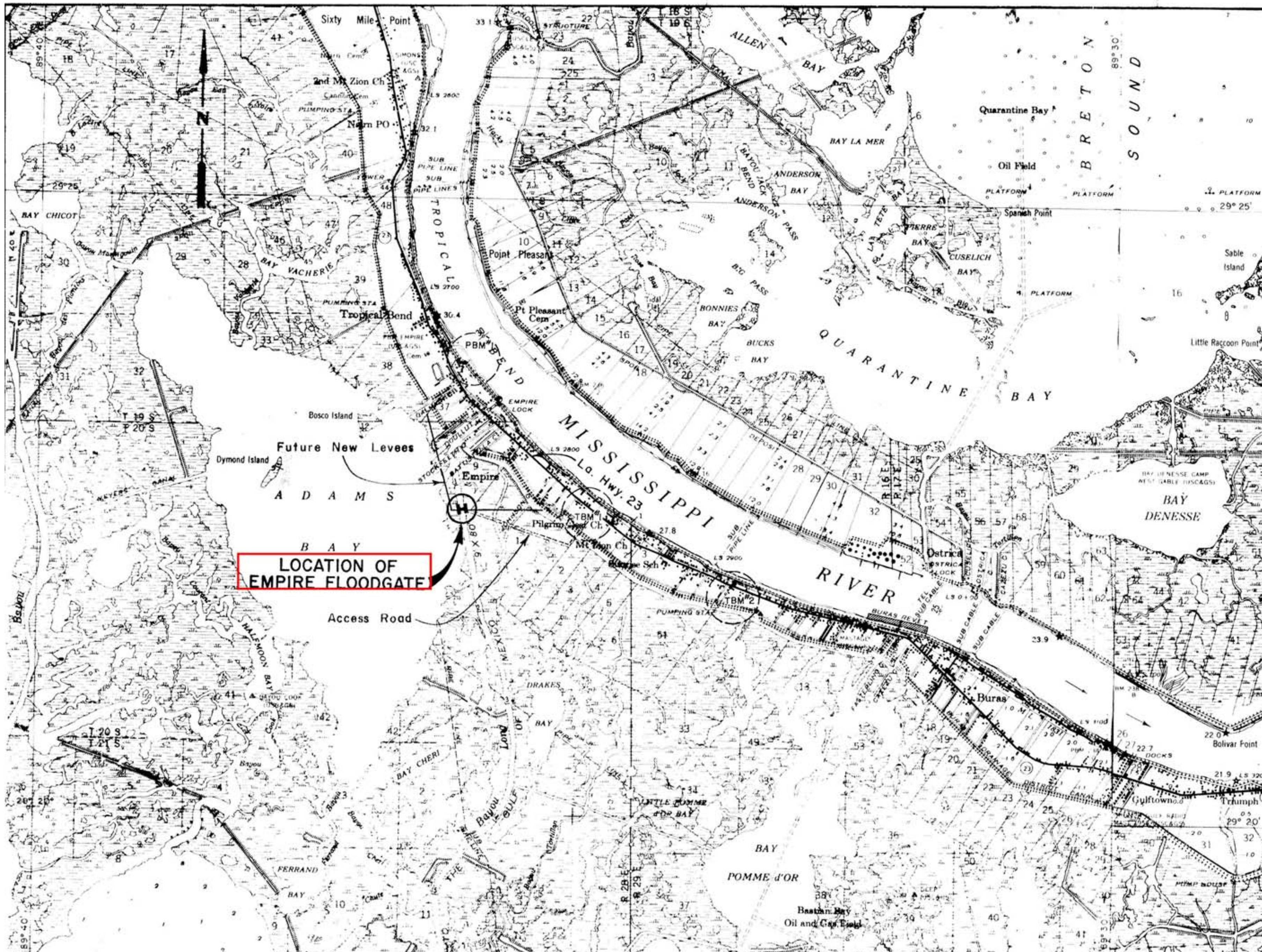
Annual Compliance Inspections were conducted for the New Orleans to Venice Project in conjunction with the Grand Prairie Levee District, the Plaquemines Parish West Bank Levee District, and the Buras Levee District.

These inspections, which were general in nature, primarily defined the status of existing project work, and provided a general condition rating.

For the last 6 years, 1998 through 2004, the ratings for the Orleans Levee District, which includes the New Orleans East polder, were “Outstanding” through year 2001, and “Acceptable” each year thereafter, at which time there was a change in the project rating scale. The project rating scale was then redefined, and “Acceptable” became the highest rating.

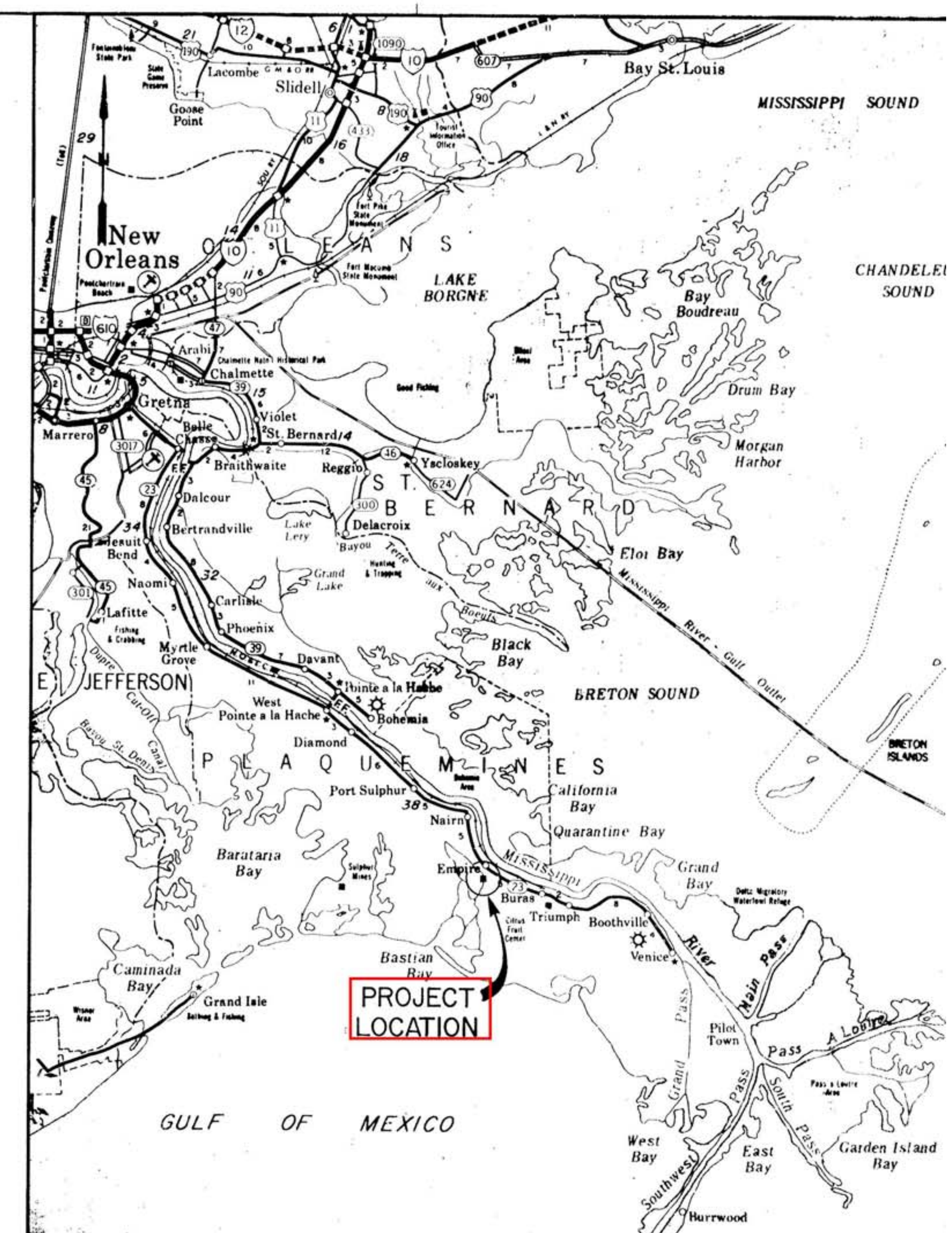
There was no specific mention of deficiencies for the hurricane protection system.

Periodic Inspections Empire Floodgates (Reference 62). Images of area shown on following page.



LOCATION MAP

SCALE 1:31,680



VICINITY MAP

SCALE IN MILES

The following information presents a summary of the inspection and corrective actions associated with those inspection deficiencies for the Empire Floodgate:

Historical Deficiencies Reported During and Related to Periodic Inspections.

Date	Description of Observations
28 August 1975	Geologist inspected in-place riprap and determined that it did not meet the original specification. Contracting Division had approved an alternate gradation on 15 March 1974. The riprap did not appear to meet the revised gradation.
4 September 1975	The following items were noted during Periodic Inspection No. 1: (1) Separation at expansion joint between T-4L and T-3L (RM-3 & RM-2); (2) Minor shrinkage and temperature cracks on tops of floodwalls and gate bay monolith; (3) Lower parts of flap gate including the hinge brackets were rusting and needed touch-up painting; (4) Several nuts on hinge anchor bolts were loose; (5) One anode missing on gate; (6) Electrical gate control panels not yet operational; (7) West sheet-pile wall settled up to 2 ft; (8) Reference marks not installed on east side of structure; (9) Reaction grillages slightly recessed and skewed horizontally with respect to bearing plate on gate; and (10) Locking device retainer plates on each end of the flap gate plate girder were out of position.
5 November 1975	A problem was encountered when lowering the flap gate. The gate was lowered to just below the waterline where it stopped and the hoist chain became slack. After jogging the motors, the gate did go down approximately another foot.
24 November 1975	An underwater inspection revealed that the hoist chain shackle pin was interfering with the concrete chamber wall.
8 December 1975	The modification provided adequate clearance between the lifting chain and the chamber wall. However, the flap gate descended slightly lower than on previous tests before stopping. Next, the counterweights were dogged off and the gate was nearly lowered to the fully open position. Contractor was instructed to remove some lead weights from the counterweights.
13 January 1977	Field inspection to check on mechanical and electrical features was conducted. Upon arrival, it was noticed that the eastern counterweight chain had failed. The counterweight was not dogged off, but suspended by its chain. When the chain failed, the counterweight fell to the bottom of the slot. The chain failed in the fourth link above the swivel. This link did not pass over the wildcat.

24 January 1977	The western counterweight chain had failed. The chain failed in the eighth link above the swivel. This link did not pass over the wildcat. Tests indicated failure was not caused by faulty material.
8 December 1977	Counterweight chain tests were conducted by WES. The forces acting on the gate hoist and counterweight chains were found to be affected by the wave height within the structure and the position of the gate when a boat traveled through the structure.
FY 1978	Recurring malfunction in the electrical panel that was generally associated with the limit switches.
4 October 1978	The following items were noted during Periodic Inspection No. 2: (1) Enlarged joint between RM-2 and RM-3, RM-4 and RM-5 and RM-18 and RM-19; (2) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls was observed; (3) Wall armor on the gate bay had extensive corrosion; (4) A small crack with efflorescence was noted on the floodside vertical face of the gate bay monolith - west side; (5) Sand deposits in needle beams appeared to be causing corrosion; (6) Settlement of sheet-pile I-wall had caused the water-stop to no longer have continuous contact with the T-wall; (7) Paint coating on underside of the gate was discolored; (8) Large amount of soil was trapped on the gate's walkway; (9) Riprap erosion protection was missing from channel slope south of the east wall - shell backfill exposed; and (10) The hoist and counterweight chains were rusted.
29 July 1981	The following items were noted during Periodic Inspection No. 3: (1) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls; (2) Continued movement of joints between RM-2 and RM-3, RM-4 and RM-5 and RM-18 and RM-19; (3) Ladders and wall armor on gate bay had extensive corrosion; (4) Counterweight recesses contained entrapped water – indicating that drain holes at elevation -9.5 ft were not functioning properly; (5) Entrapped water in east counterweight recess was flowing through eight small holes on the north face of the concrete wall just above the pump platform; (6) Spalled area of concrete was noted underneath a handrail anchor plate on the northeast corner of the west side channel wall; (7) Cracked area of concrete was noted underneath a handrail anchor plate on southeast corner of the west side channel wall; (8) Water was noted leaking through form bolt holes on both sides of the T-wall tie-in to the gate bay monolith on the west side of the structure; (9) Sand deposits in the needle beams were causing corrosion; (10) Decaying timber blocking was noted underneath the needle girder; (11) Settlement of the sheet-pile I-wall had caused the water stop to no longer make continuous contact with the T-wall; (12) Corroded sheet-pile I-wall was noted; (13) A large amount of soil trapped on the gate's walkway; (14) Two dents on the skin plate at the top of the flap gate was noted

near each of the two vertical center ribs; and (15) Missing/corroded handrail anchor bolts were noted.

3 May 1982 — Plaquemines Parish inspected the structure gate several days after it was noted that a boat had hit the gate. Soundings indicated the top of the gate was at elevation 10.5 ft. Upon raising the gate, there was a dent in the middle. The damage was confined to the top beam and face plate.

31 January 1984 — The following items were noted during Periodic Inspection No. 4: (1) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls since last inspection; (2) Joint readings showed no substantial change between RM-2 and RM-3 and RM-4 and RM-5; (3) Minor concrete popout on top surface of west side wall was noted; (4) Missing handrail post anchors were noted; (5) Decaying timber blocking was noted underneath the needle girder; (6) No change was noted in the two dents on the skin plate at the top of the flap gate near each of the two vertical center ribs; (7) Breakwater dike on west bank of south approach channel had lost all riprap; (8) Staff gage number was illegible; and (9) Continued widening of joint between RM-18 and RM-19 (monoliths T-3R and T-4R) was noted.

29 January 1987 — The following items were noted during Periodic Inspection No. 5: (1) No substantial change was noted between RM-2 and RM-3 and RM-4 and RM-5; (2) Entrapped water in east counterweight recess was flowing through a hole on the north face of the concrete wall just above the pump platform; (3) Ladders and wall armor on the gate bay had extensive corrosion; (4) Large amount of soil trapped on the gate's walkway; (5) Moderate damage to the skin plate and channel beam across the top of the gate on the three central interior spans; (6) Section of handrail adjacent to the ladder recess on the west side of the structure was not anchored; (7) No pile caps on the majority of the timber piles; (8) Breakwater dike on the west bank of south approach channel had lost all riprap; (9) Riprap protection was deficient all around the structure; (10) Top of gate resting at approximate elevation of -10.5 ft in lieu of elevation -14.0 ft and this was due to the accumulation of silt in the gate recess; (11) Cathodic anodes needed to be replaced; and (12) Excessive settlement of the eastern sheet-pile I-wall had occurred.

August 1989 — At the request of Plaquemines Parish, the structure was inspected by New Orleans District mechanical and electrical engineers. The following were found: (1) A motor casing for ream drive had fractured and loose parts jammed the motor, and (2) Electrical problems existed with the transmitter for the synchronizer for the lift motors.

30 January 1990	The following items were noted during Periodic Inspection No. 6: (1) A ½-in. gap was noted between L-type water stop and T-wall; (2) Entrapped water in east counterweight recess was flowing through a 2-in. hole on the north face of the concrete wall – recently drilled to drain ponded water; (3) Ladders, corner plates and wall armor on gate bay had extensive corrosion; (4) Sacrificial anodes were partially eaten away; (5) The previously reported area of moderate damage to the skin plate and channel beam across the top of the gate on the three central interior spans had sustained further damage since last inspection; (6) South handrail on eastern half of structure had been hit – caused some spalling of concrete adjacent to anchor bolts, but still securely attached; (7) Handrailing on the north side of the access ladder was loose and missing anchor bolts; (8) No pile caps were on the majority of the timber piles; (9) Sand deposits in needle beams were causing corrosion; (10) Rotted timber blocking was noted underneath the needle girder; (11) Bottom stair tread to boat dock had rusted completely through; (12) Spall was noted at the expansion joint between T-wall monoliths T-1R and T-2R; (13) Joint filler material was missing from most of the expansion joints; (14) Continued rotation settlement of T-4R away from T-3R was occurring – stretching the water stop; (15) Breakwater dike on west bank of south approach channel had lost all riprap; (16) Riprap protection was deficient around the entire structure; (17) Channel bank lines were receding; (18) Staff gages needed cleaning – had barnacles and algae; and (19) Several chain links in splash zone were severely corroded.
28 January 1993	The following items were noted during Periodic Inspection No. 7: (1) Spalls were noted at handrail anchor plates; (2) Minor popouts were observed on top of T-walls; (3) Protective coating was peeling off east sheet piling, especially just above the ground surface; (4) The breakwater dike was underwater; (5) Lack of riprap on levee nose was noted; (6) ½-in. gap existed between L-type water stop and west T-wall; (7) 1-½- to 3-in. gap existed between L-type water stop and east T-wall; (8) Split in the water stop was noted at the joint between monoliths T-4L and T-3L; (9) Rodent holes along west I-wall, protected side, were noted; (10) Channel bank lines were continuing to recede; (11) Riprap protection was slightly deficient all around the structure even though additional riprap had been placed in January 1991; (12) Damage to the channel beam on the flap gate was noted; (13) No change was noted in damage to the south handrail on eastern half of the structure – spalling of concrete adjacent to anchor bolts, but still securely attached; (14) Pump platform ladder had rusted strings and rungs at the bottom; and (15) Handrailing on the north side of the access ladder on west side of structure was loose and missing anchor bolts.

17 January 1996 & 13 February 1996	The following items were noted during Periodic Inspection No. 8: (1) Gap of 1 in. between L type water stop and T-wall monolith T-4L; (2) Torn waterstop was noted between T-wall monoliths T-4L and T-3L and T-4R and T-3R; (3) Spalls on the gate bay noted from the last inspection had not changed; (4) Corner plates and wall anchor on gate bay had extensive corrosion; (5) Deterioration of paint noted on all exterior miscellaneous metals; (6) Handrails on the boat dock had deteriorated to a point that they no longer provided a safety function; (7) Bottom three treads and attachment plate on boat dock stairway were severely corroded; (8) Pump platform access ladder was badly corroded in the splash zone; (9) Siltation of gate recess continued to be a problem; (10) The top of the flap gate was damaged; (11) Exterior surface of gate operating machinery had deteriorated badly - corrosion was noted in many areas; (12) Chain links in splash zone were severely corroded and encrusted with oyster shells/ barnacle growth; (13) Steel counterweight cages were badly corroded; (14) Ratchet type load binders for counterweights were not in place; (15) Observed flow distribution in discharge manifold for gate recess flushing system is poor; (16) Pump engine muffler needed replacing; (17) Hydraulic system for safety latching of gate and gate shock absorption were not operable; (18) Emergency generator had old and loose fan belts, old cooling hoses, and oil leaks; (19) Diffuser lens needed on two east side pole lights; (20) Wires were exposed from removed light poles (one on east side and three on west side); (21) Wire was hanging out of junction boxes for gate latching device on east side; (22) Nondestructive testing of needle girders had not been performed; (23) Three timbers on north guide walls needed replacement in near future; (23) Rotted timber blocking was noted underneath the needle girders; (24) Deck board was missing from the boat dock; (25) Breakwater dike on west bank of south approach channel had lost riprap and was well below the design grade; (26) Channel bank lines were continuing to recede due to wave wash; (27) West sheet-pile I-wall was up to 3.4 ft below design grade; and (28) Coal tar epoxy protective coating had completely deteriorated off the sheet piling near the ground line.
17-18 February 1997	Performed electrical load tests on gate hoist motors.
23 March 1999 & 2 September 1999	The following items were noted during Periodic Inspection No. 9: (1) Spall was noted on the protected side of T-wall at the joint between monoliths T-2R and 3R; (2) Torn water stop between T-wall monoliths T-4L and T-3L and T-4R and T-3R noted from last inspection had not been repaired; (3) Complete deterioration of joint filler material was observed; (4) Spalls were noted at handrail anchor plates - noted at previous inspections; (5) Entrapped water in east counterweight recess was flowing through numerous small holes on the north face of the concrete wall just above the chambered construction joint - first noted during Periodic Inspection No. 3; (6) Four of the eight piles supporting the pump

platform had extensive vertical cracks that extend below the water surface sketches; were provided; (7) Crack in support beam of pump platform, southwest corner; sketches were provided; (8) Corner plates and wall armor on gate bay had extensive corrosion; (9) Top of flap gate was damaged; (10) Inoperative gate shock absorbers were noted; (11) HSS evaluation of gate and dewatering components was needed; (12) Wildcat sheaves had appreciable wear on load side; (13) Chains showing some corrosion and barnacle collection were noted at and below the splash zone; (14) Limit switches and synchronizing system were no longer maintained; manual operation of gate was preferred; (15) Lack of redundancy was noted in the event the gate hoist system failed; (16) Pump vanes and diffuser bowl had never been inspected; (17) Hand operated pump lubricating unit for pump bearings was inoperable; (18) Water leak was present below air vent valves indicating ruptured pipes; (19) Badly corroded attachment ring, nuts, and bolts for 24-in. check valve and expansion coupling; (20) Pump engine muffler needed replacing; it was badly corroded and leaking exhaust through the bottom; (21) Two small leaks in generator cooling system were noted; (22) Broken refractors were noted for the two lights on the east side; (23) Several deteriorated timbers on the guide walls needed replacement in the near future; (24) Breakwater dike on west bank of south approach channel had lost riprap and was well below the design grade; (25) Channel bank lines were continuing to recede due to wave wash; (26) A gap was noted between the east sheet-pile I-wall and earthen embankment; and (27) Reference marks on the top of the I-walls were obliterated when the coal tar epoxy was applied

17 August 2000 — Plaquemines Parish made a visual inspection of pump: (1) Needed to replace suction and piping between gate and pump; (2) Needed to replace universal joints between engine and pump angle drive; (3) Needed to replace/rebuild greasing system for pump and pump angle drive; (4) Needed to check components in gear box – replace seals as necessary; (5) Needed to replace/repair relief valves; (6) Needed to replace expansion joint coupling in discharge line; and (7) Needed to replace tie bolts for expansion joint.

14 October 2001 — Plaquemines Parish noted that water was passing through sides of gate. They stated that water stops should be changed at next dewatering.

8 September 2003 — Plaquemines Parish found a cracked link in the east side hoist chain.

Historical Repairs/Construction Work.

Date	Description of Observations
20 October 1975	The following were noted: (1) Lower portion of the gate was sandblasted and painted; (2) All nuts on the hinge anchor bolts were tightened; (3) Exposed form work anchor bolts were burned off; (4) Reinstalled anode that had broken loose; and (5) Concrete was chipped out adjacent to grillage bearing face to allow contact between gate bearing plate and grillage bearing face.
November – December 1975	The detachable link on the hoist chain was attached to the pickup clevis on the gate to provide adequate clearance away from the chamber wall.
2-3 May 1977	The broken counterweight chains were removed and new chains were installed by New Orleans District hired labor at a cost of $1,735 (excluding chain cost). The gate was operated to test all components of the system. While lowering the gate, the east chain slipped off the drive wildcat and caused the east end of the gate to drop abruptly (about a foot) before the chain caught again in the wildcat.
24 May 1977	The drive wildcats were ground down by New Orleans District hired labor. A permanent solution was considered to consist of replacing both weldment-type wildcats with steel casting type wildcats.
13-31 March 1978	Two steel casting type replacement wildcats were installed. This work involved the cutting of the pillow blocks to remove the old wildcats. New Orleans District hired labor performed the work for a cost of $26,811 (excluding wildcat cost).
Prior to August 1978	The idler drum diameter was increased from 8 to 14 in. and the gate locking device retainer plates were repositioned.
18 May 1979	Replaced two solar panels and repaired damage resulting from theft of panels at a cost of $1,009.
21 May 1979	Replaced parts and repaired damage done by vandals: broken glass, ripped screens, broken shutters, and stolen ratchet jack.
May–June 1982	Plaquemines Parish Commission Council had soil deposits in the gate recesses cleaned out by contract dredging and labor forces. They also replaced lights and shutters.
4 August 1982	Workers made repairs to damaged portion of the gates for a cost of $15,037.

September 1982	Workers sandblasted and repainted embedded steel on gate bay and pump pipe.
January 1983	Maintenance work was performed on the handrails, and miscellaneous cleaning and painting was done.

Other Features - New Orleans to Venice - Plaquemines

Brief Description.

The primary components of the hurricane protection system for the New Orleans to Venice reach basins are described above, namely the levees and floodwalls designed and constructed by the Corps. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section briefly describes and presents the criteria and pre-Katrina conditions of the interior drainage system, pump stations, non-Corps back levees, and the Mississippi River Flood Protection System. This section covers all the basins in Plaquemines Parish from New Orleans to Venice.

Pre-Katrina Conditions.

According to the local jurisdictions responsible for interior drainage, the storm drain systems, roadside ditches, interior canals, and outfall pump stations were in good condition and prepared for high inflows from rainfall prior to 29 August 2005.

The Corps and non-Corps back levees were in good condition prior to Katrina landfall.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

Interior Drainage System.

Overview. The New Orleans to Venice reach contains 11 separate basins totaling about 60 square miles. The land generally slopes away from the Mississippi River to marshland. It is sparsely developed except for areas along the Mississippi River, and there is considerable agricultural and petrochemical activity. Many features in the Belle Chase basin are typical of urban cities in the United States, and some features are unique because much of the area is below sea level. Surface runoff from yards and streets flows into roadside ditches or into inlets and storm sewers. Excess runoff flows down streets and/or overland to lower areas. The other basins have more rural type drainage features with many areas below sea level. Stormwater pump stations, located along the back levees, pump the water directly into outfall canals or sloughs in the marsh. No stormwater is pumped into the Mississippi River. The entity responsible for local drainage in the New Orleans to Venice reach is Plaquemines Parish.

System Components. Stormwater flow is influenced by the land topography, roadways, ditches, canals, and pump station layout. Figure 5 in Volume VI shows the topographic layout of the northern end of the reach. With the relatively narrow basins and agricultural influence, the interior drainage systems consist mainly of roadside ditches and canals. The ditches and canals not only collect stormwater from the land and roadways and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations. Because material for the levees came from the interior canals, there is considerably more storm water storage in the Plaquemines Parish canals than in the other urban areas. Based on land topography and the drainage system, the 11 basins were divided into 37 subbasins. The outfall pump stations are located along the back levees. Pump station information is presented below.

Design Criteria. The interior drainage systems in the older and rural areas have a capacity of about a 50% probability(2-year frequency) event, while new drainage projects and developments are required to accommodate a 10% probability (10-year frequency) event. Where canal or pump capacity is not available downstream, larger developments are required to put in stormwater detention facilities. The goal for new or upgraded pump stations is to pump one in./hr for the first 2 hours and ½ in./hr after that. The current functional capacity of the canals and pump stations is 0.25 in./hr. The level of protection is similar to the other New Orleans area basins that have a higher pumping capacity because of the additional storage in the Plaquemine Parish open canals.

There are no Southeast Louisiana (SELA) Urban Flood Control Projects in these basins.

Pumping Stations - Plaquemines Parish Summary.

Figure 29 is a map showing the Plaquemines Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips.

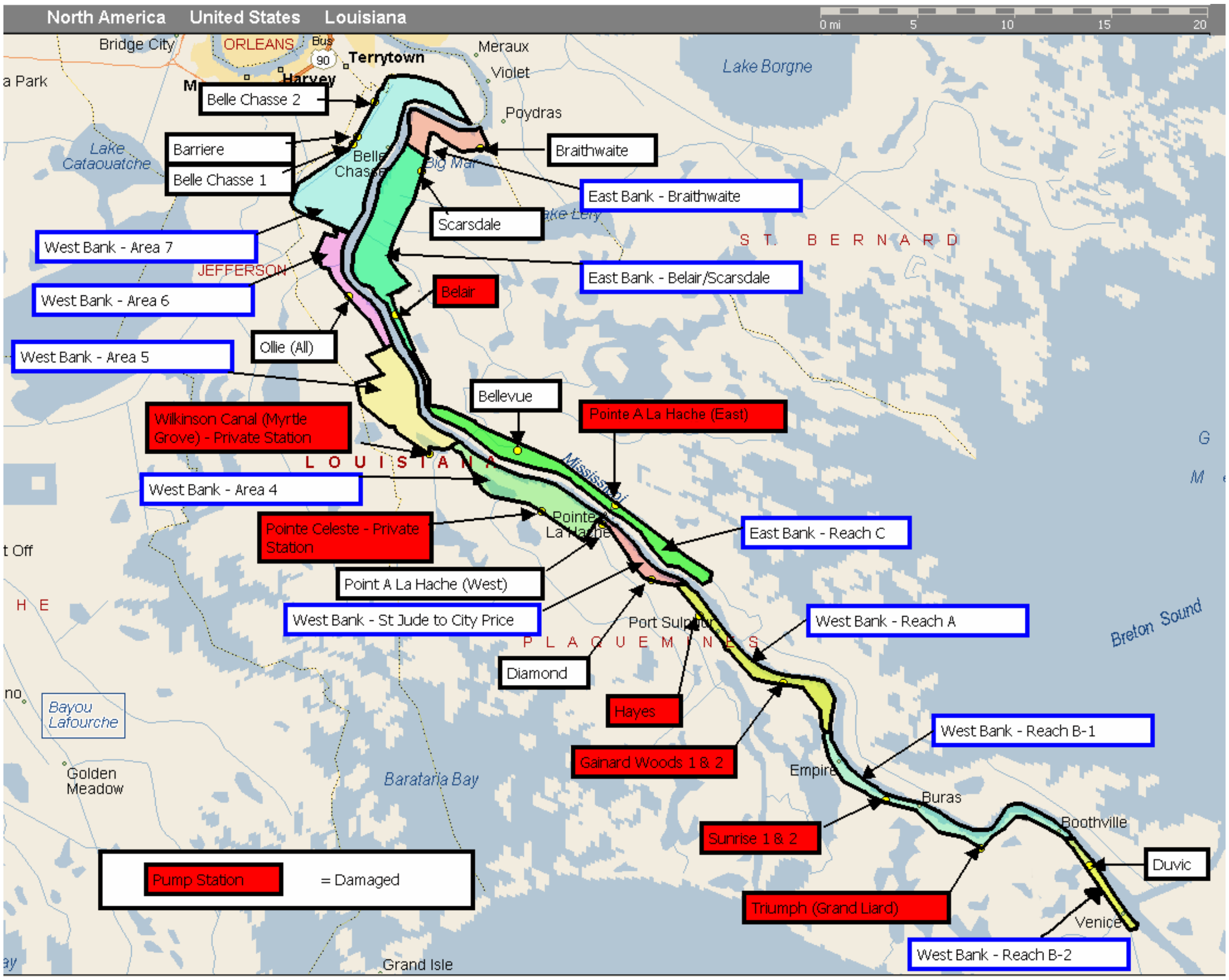


Figure 29. Plaquemines Parish pump station locations.

Drainage Basin. Plaquemines Parish consists of ten separate drainage basins. These basins have one or two pump stations, with the exception being the East Bank – Braithwaite, which has three pump stations. Plaquemines Parish borders the Mississippi River. The pump stations generally discharge into marshes, although there are exceptions. The pump stations predominantly use diesel driven vertical pumps. Details for each pump station are listed in Volume VI.

East Bank – Braithwaite

Braithwaite

Intake location: ....Braithwaite Pond
Discharge location: .... Marsh
Nominal capacity: ....105 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	40	1974	Diesel	Vertical
2	65	1974	Diesel	Vertical

East Bank – Belair/Scarsdale

Belair

Intake location: ....Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....130 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	130	1950	Diesel	Vertical

Scarsdale

Intake location: .... Scarsdale Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....1,784 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	446	1965	Diesel	Horizontal
2	446	1965	Diesel	Horizontal
3	446	1965	Diesel	Horizontal
4	446	1965	Diesel	Horizontal

East Bank – Reach C

Bellevue
Intake location: .... Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....516 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	258	1972	Diesel	Horizontal
2	258	1972	Diesel	Horizontal

East Point a la Hache
Intake location: .... Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....580 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	290	1972	Diesel	Horizontal
2	290	1972	Diesel	Horizontal

West Bank – Area 7

Belle Chasse 1
Intake location: .... Barriere Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: .... 3,556 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	800	1955	Diesel	Horizontal
2	800	1955	Diesel	Horizontal
3	150	1955	Diesel	Vertical
4	903	1963	Diesel	Horizontal
5	903	1963	Diesel	Horizontal

Belle Chasse 2
Intake location: ....Belle Chasse Drainage Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....1,050 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	350	n/a	Diesel	Vertical
2	350	n/a	Diesel	Vertical
3	350	n/a	Diesel	Vertical

Barriere Road
Intake location: .... Barriere Pond
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....25 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	25	n/a	Diesel	Vertical

West Bank – Area 6

Ollie Lower
Intake location: .... Ollie Canal
Discharge location: ....Ollie Outfall Canal
Nominal capacity: ....440 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	140	n/a	Diesel	Vertical
2	150	1981	Diesel	Vertical
3	150	1981	Diesel	Vertical

Ollie Upper

Intake location: .... Ollie Canal
Discharge location: ....Ollie Outfall Canal
Nominal capacity: ....140 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	n/a	1964	Diesel	Vertical
2	140	1964	Diesel	Vertical

West Bank – St. Jude to City Price

West Pointe a la Hache

Intake location: .... West Pointe A La Hache Canal
Discharge location: ....Jefferson Lake Canal
Nominal capacity: ....45 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	15	n/a	Diesel	Vertical
2	15	n/a	Diesel	Vertical
3	15	n/a	Electric	Vertical

Diamond

Intake location: ....Diamond Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....256 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	128	1976	Diesel	Vertical
2	128	1976	Diesel	Vertical

West Bank – Reach A

Hayes
Intake location: .... Hayes Drainage Canal
Discharge location: .... Marsh
Nominal capacity: .... 500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	250	1963	Diesel	Horizontal
2	250	1963	Diesel	Horizontal

Gainard Woods 1
Intake location: ....Gainard Woods Canal
Discharge location: .... Marsh
Nominal capacity: .... 570 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	205	1960	Diesel	Vertical
2	205	1960	Diesel	Vertical

Gainard Woods 2
Intake location: ....Gainard Woods Canal
Discharge location: .... Marsh
Nominal capacity: .... 570 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	285	1985	Diesel	Vertical
2	285	1985	Diesel	Vertical

West Bank – Reach B1

Sunrise 1
Intake location: .... Sunrise Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....180 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	90	1960	Diesel	Vertical
2	90	1960	Diesel	Vertical

Sunrise 2
Intake location: .... Sunrise Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....290 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	145	1979	Diesel	Vertical
2	145	1979	Diesel	Vertical

Grand Liard/Triumph
Intake location: .... Bural Drainage Canal
Discharge location: ....Grand Liard Marsh
Nominal capacity: ....840 cfs

West Bank – Reach B2

Duvic
Intake location: .... Venice Drainage Canal
Discharge location: .... Bayou Duvic
Nominal capacity: ....560 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	280	1976	Diesel	Vertical
2	280	1976	Diesel	Vertical

West Bank – Area 5

Wilkinson Canal (Myrtle Grove)
Intake location: ....Unnamed Canal
Discharge location: .... Marsh
Nominal capacity: ....980 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
101	223	n/a	Diesel	Vertical
102	223	n/a	Diesel	Vertical
103	267	n/a	Diesel	Vertical
104	267	n/a	Diesel	Vertical

West Bank – Area 4

Pointe Celeste (Upper and Lower)
Intake location: ....Unnamed Canal
Discharge location: .... Marsh
Nominal capacity: ....895 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
105	223	n/a	Diesel	Vertical
106	223	n/a	Diesel	Vertical
107	223	n/a	Diesel	Vertical
108	223	n/a	Diesel	Vertical

Levees and Floodwalls.

MRL West Bank Mississippi River Levee City Price to Venice, LA. Approximately 34 miles of levee (Reference Nos. 44, 71). Image of area shown on following page.

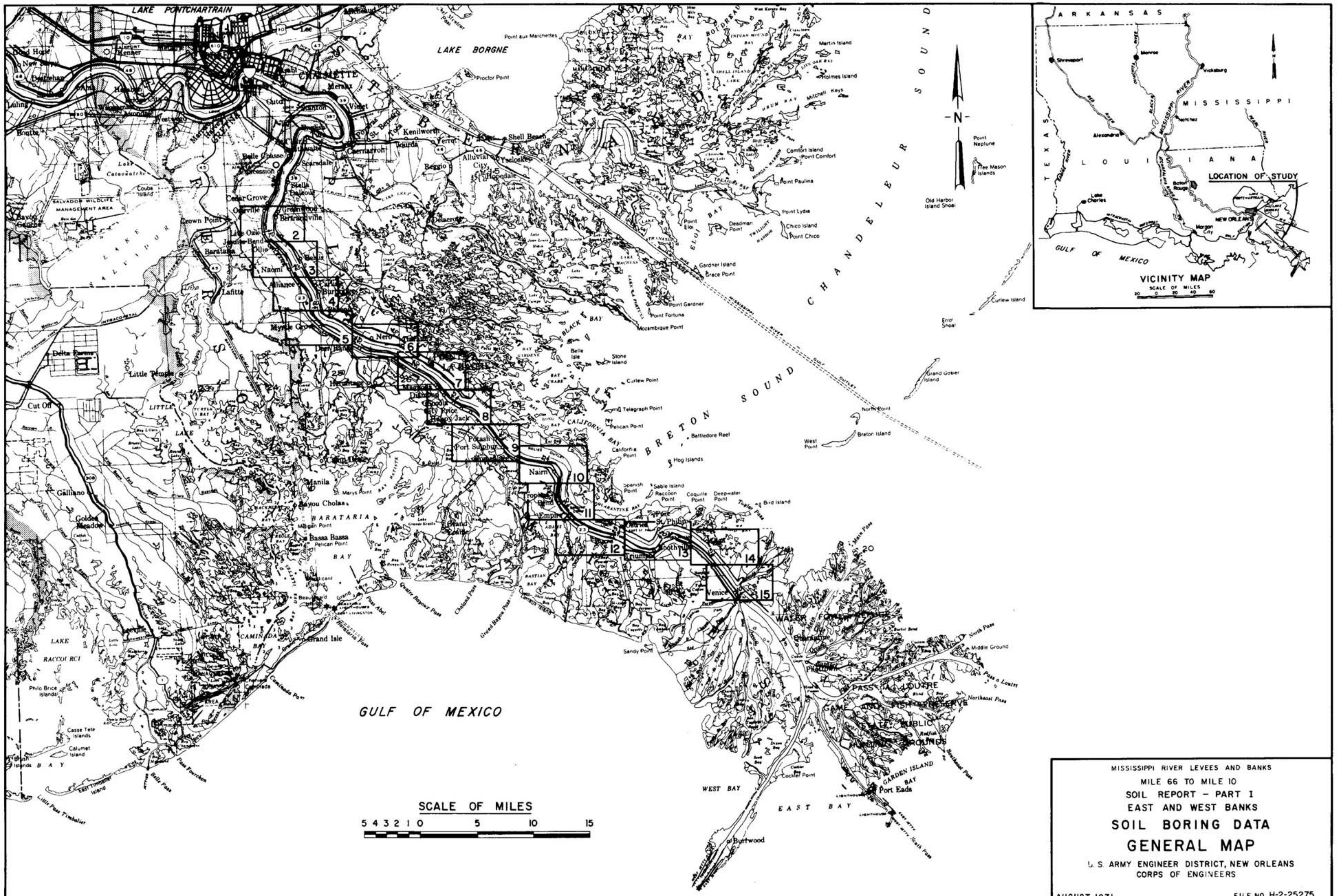

LAKE PONTCHARTRAIN
LAKE BORGNE
CHANDELEUR SOUND
BRETON SOUND
GULF OF MEXICO
BARATARIA BAY
CAMINADA BAY
LAKE RACCOURCI
WEST BAY
EAST BAY
SCALE OF MILES
5 4 3 2 1 0 5 10 15
N
VICINITY MAP
ARKANSAS
MISSISSIPPI
LOUISIANA
TEXAS
LOCATION OF STUDY
GULF OF MEXICO
NEW ORLEANS
SCALE OF MILES
20 0 20 40 60
MISSISSIPPI RIVER LEVEES AND BANKS
MILE 66 TO MILE 10
SOIL REPORT – PART I
EAST AND WEST BANKS
SOIL BORING DATA
GENERAL MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1971
FILE NO. H-2-25275

PLATE I

Geology. The study area is located within the Central Gulf Coastal Plain. Specifically, the area is located on the modern subdelta which projects gulfward from the deltaic plain of the Mississippi River. It is a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from about 5 ft along the crests of the natural levees to sea level or slightly lower in the marshlands between the natural levee ridges. The numerous inland bodies of water range in depth from 1 to 6 ft. The Mississippi River channel ranges in depth from 65 to 190 ft below sea level.

Foundation Conditions. The foundation conditions for these 34 miles of levee are very complex because of the geologic history of the area. The subsurface on the MRL West Bank consists of Recent deposits ranging in thickness from about 150 ft at mile 44 to 260 ft at mile 10AHP, the downstream end of the project. The Recent deposits are underlain by Pleistocene age materials. Generally, the Recent deposits consist of a surface layer of soft to stiff natural levee clays with layers and lenses of silt varying in thickness from 3 to 22 ft. The natural levee deposits are underlain by a discontinuous layer of very soft marsh clays with peat and organic matter. The marsh deposits range in thickness from 2 to 9 ft. The natural levee and marsh deposits are underlain by soft, alternating intradelta clays and silts with layers of silty sand and sand. The thickness of the intradelta deposits varies from 25 to 97 ft. Some of the natural levee and marsh deposits are underlain by very soft to soft interdistributary clays with lenses and layers of silt and silty sands. The interdistributary deposits range in thickness from 20 to 78 ft. The remaining reaches of natural levee and marsh deposits are underlain by point bar silts, silt sands, and sand with layers of clay.

Field Exploration.

Considerable and related data had been obtained in the past from the West Bank of the Mississippi River between miles 10 and 66 to determine the characteristics of the subsurface materials for use in levee and revetment designs. These data are presented in Reference No. 44. The report presents soil data for the West Bank including detailed boring logs, shear test data, consolidation test data, and soil stratification and shear strengths. Of the 247 borings included in Reference No. 44, ten general type and 42 undisturbed borings were considered in the design of the present study area.

Additionally, more recent borings considered in the present design are:

(1) Borings 1-UBU, 2-UBU, 3-UBU, 4-UBU, 5-UBU, 6-UBU, 7-UBU, and 8-UBU taken for the Upper Buras Levee Enlargement

(2) Borings R-25.1-U, R-25.1-UT, R-25.5-U, and R-25.5-UT taken for Buras Revetment.

Boring depths were dependent upon the project for which the borings were taken. Borings ranged from ground surface to as shallow as elevation -15 ft NGVD and as deep as elevation -219 ft NGVD. Undisturbed borings were taken with a 5-in.-diameter steel-tube, piston-type sampler and general types with a 1⅞-in.-ID core barrel sampler.

Levee Improvements. The most prevalent typical levee section is a landside enlargement of the existing MR&T levee; however, there are riverside enlargements, straddle enlargements, and landside enlargements with underwater rock beans and setbacks. The project basically will consist of raising the existing Mississippi River levee (MR&T) hurricane project height with an appropriate wave berm. The levee embankment will consist of semicompacted clay fill, while the berms will consist of uncompacted fill with riprap armor on the riverside face of the wave berm.

Settlement. Consolidation data from borings within a reach were combined and average parameters were determined. Analyses for the enlarged levee sections indicate that the gross grade levee crown will settle from 0.2 to 2.5 ft, depending on the location within the project area. This settlement includes a 10 percent allowance for shrinkage and lateral spread. Due to right-of-way and cost constraints, the enlarged levees will be constructed in multiple lifts in all reaches except Reach T, where a gross levee enlargement is possible. Multiple lift construction will involve enlarging the levee to the desired project net grade for intermediate lifts and grossing the levee in the final lift as required to achieve the desired project net grade. Four years was used as the time between lifts.

Based upon local experience and calculations performed using consolidation data taken from the undisturbed borings, settlement of the setback levees in the first 4 years due to consolidation, shrinkage, and lateral spread was estimated to be about 4 ft. Again, because of right-of-way and cost constraints, the setback levees will be constructed in multiple lifts.

Slope Stability. Protected side stability berms were designed, proposed levee and wave berth stability checked, and floodside stability control lines were developed for the following conditions: Water level to the still-water level on the floodside for failure toward the protected side; and low water on the floodside for failure toward the floodside. Minimum slopes were determined by the Method of Planes analysis, using design shear strengths and a minimum FS of 1.3.

I-Walls. The only structure in the project is the existing Empire lock at Empire, LA. It was determined that no modification was necessary to the lock gates. Wave overtopping would be allowed; however, the 30 ft of existing I-wall on both sides of the structure was found to be inadequate to withstand the projected hurricane wave force if capped to project height. The length of the existing sheet piling is inadequate. For structural and constructability reasons, the existing sheet piling shall be removed and replaced with adequate lengths of new PZ-27 sheet piling. All measurable settlement will be allowed to take place prior to I-wall capping. This period is estimated to be 3 years after completion of second enlargement embankment work in the area; however, the area will be monitored regularly to confirm this time period.

T-Walls. No T-walls are included on this project.

Erosion Protection. The berms will consist of uncompacted fill with riprap armor on the riverside face of the wave berm.

Non-Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3. West Bank and Vicinity

3.2.3.1. General Description

The project is located on the west bank of the Mississippi River in the vicinity of New Orleans and in Jefferson, Orleans, and Plaquemines Parishes.

The project will provide Standard Project Hurricane Protection to residents from storm surges from Lakes Cataouatche and Salvador, and waterways leading to the Gulf of Mexico.

The original project included 22 miles of earthen levee and 2 miles of floodwalls extending from the Harvey Canal to the V-levee near the Jean Lafitte National Historical Park and to the town of Westwego. The Lake Cataouatche area eliminated the west-side closure in Westwego, and added about 10 miles of levee and 2 miles of floodwalls to the project. The East of Harvey Canal area includes a sector floodgate in the Harvey Canal just below Lapalco Boulevard and about 25 miles of levee and 5 miles of floodwalls, including enlargement of the federal levees along the Algiers Canal.

Figure 30. Hurricane protection project, West Bank and Vicinity, New Orleans, LA.

3.2.3.2. History

The modern West Bank and Vicinity, New Orleans, Louisiana, Hurricane Protection Project emanated as an outgrowth of the original New Orleans to Venice Hurricane Protection Project. In 1965 and 1966, four resolutions—two adopted by the Senate Committee on Public Works and two by the House Committee on Public Works—authorized reviews of the project posited in the 1962 Chief of Engineers report on the Mississippi River Delta at and below New Orleans, (which had been authorized in the 1962 Flood Control Act and later renamed the New Orleans to Venice Hurricane Protection Project) to determine feasibility of modifying the project to provide improved hurricane protection and flood control to portions of Jefferson Parish lying between the Mississippi River, Bayou Barataria, and Lake Salvador.

In December 1986, the Corps completed a feasibility report that examined potential hurricane surge protection measures for the west bank of the Mississippi River in Jefferson Parish between Westwego and the Harvey Canal to the vicinity of Crown Point. The recommended plan included 22 miles of earthen levees and two miles of floodwalls extending from the canal to the V-levee near Jean Lafitte National Historical Park north to the town of Westwego. The 1986 Water Resources and Development Act (Public Law 99-662) authorized the Westwego to Harvey Canal Hurricane Protection Project and construction was initiated in 1991.

The Westwego to Harvey Canal Hurricane Protection underwent a post-authorization change in the mid 1990s. In February 1992, the Corps completed a reconnaissance study regarding hurricane protection for that portion of the west bank of the Mississippi River in Jefferson Parish between Bayou Segnette and the St. Charles Parish line, particularly at Lake Cataouatche where the existing non-federal levee had been deemed structurally unstable. The reconnaissance study recommended a plan based on combination of levees and steel sheet-pile floodwalls generally along the existing Lake Cataouatche levee alignment to provide protection from hurricane tidal surges up to a 100-year recurrence interval. This plan also eliminated the need for the west-side closure in Westwego authorized under the Westwego to Harvey Canal Hurricane Protection Project. The 1996 Water Resources and Development Act (Public Law 99-662) authorized the Lake Cataouatche area project.

That same act also authorized the East of Harvey Canal Hurricane Protection Project. In 1994, the Corps completed a feasibility report that examined additional hurricane surge protection to approximately 35,000 acres in portions of Jefferson, Orleans, and Plaquemine Parishes bounded by the Harvey Canal to the west, the Mississippi River to the north and east, the Hero Canal to the south, and divided by the Algiers Canal. For the area west of the Algiers Lock, the feasibility report recommended the construction of a navigable floodgate in the Harvey Canal just south of Lapalco Boulevard; a combination of levees and floodwalls on the east side of the canal from the floodgate to the Hero Pumping Station; raised protection from the Hero Pumping Station along both banks of the Algiers Canal to the Algiers Lock; and an outfall canal for the Cousins Pumping Station. For the area to the east of the Algiers Lock, the study recommended raising the existing protection along the Algiers and Hero Canals; and the construction of a new levee near Oakville to connect to the Hero Canal levee and to the existing Plaquemines Parish levee. By tying into the line of protection authorized under the Westwego to Harvey Canal project, the recommended plan presented a continuous line of protection for west bank residents from Westwego to Oakville.

The 1999 Water Resources and Development Act (Public Law 106-53) combined the Westwego to Harvey Canal project, the Lake Cataouatche project, and the East of Harvey Canal project into West Bank and Vicinity, New Orleans, LA, Hurricane Protection Project.

3.2.3.3. Datum - Subsidence and Vertical Datum Problems in New Orleans, LA

Because of technological gains, the Corps is able to more accurately track subsidence of projects – something that could not be done as reliably in the past. Based on a recent study, we can now estimate that the New Orleans area is subsiding at a rate of 6 to 17mm/yr or 2 to 5½ feet per century. In New Orleans itself, subsidence is about 3 feet per century. Subsidence is as much as 10 feet per century in Venice.

The IPET, an independent group activated by the Corps to study the response of the hurricane protection system during Hurricane Katrina, identified problems with using the previous vertical datum to which survey benchmarks were referenced. IPET's ability to accelerate analysis of this issue, which was ongoing by the Corps' New Orleans District and the NOAA's NGS, led to the identification of two major problems with elevations in the New Orleans area: subsidence and the use of the old vertical datum elevations as equal to local mean sea level, a common misunderstanding in the engineering community up until the 1990s.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), and structure and levee elevations. It has been known since 1985 that the elevations of benchmarks in and around New Orleans were inaccurate, due to subsidence, and needed to be updated. The exact amount of subsidence was not known until a 2004 survey conducted by the NGS in cooperation with the Louisiana Spatial Reference Center, the Corps and state and local governments was performed on 86 benchmarks in southern Louisiana.

The 2004 survey pointed out inaccuracies due not only to subsidence, but also to distortions and errors in elevations of benchmarks that were assumed to be stable in the past, but had in fact subsided themselves. Based on the 2004 survey, the Corps has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in southern Louisiana. Use of the 2004 survey assures consistency for all elevation surveys performed in the southern Louisiana area.

The IPET has developed a relationship between the current local mean sea level and the 2004 survey, which is referred to as the North American Vertical Datum of 1988 (2004.65 Adjustment). Local mean sea level in the city of New Orleans is about ½ foot above the 2004 datum. The Corps will use the 2004 elevations and their varied relationship to the local mean sea level throughout the area to precisely determine the elevations of levees and other critical flood protective structures. This datum will also be used by the construction industry and others in southern Louisiana for a variety of projects that rely on elevations relative to the local water surface.

More information can be found in the “Geodetic and Water Level Datum” report.

3.2.3.4. Design Hurricane

Because of the urban nature of the project area, the selected design hurricane is the standard project hurricane.

Standard Project Hurricane

The standard project hurricane (SPH) is one that may be expected from the most severe combination of meteorological conditions that are considered reasonably characteristic of the region. Guidance on the selection of site-specific storm meteorological parameters was initially given in National Hurricane Research Project Report No. 33 (U.S. Weather Bureau 1959). The Weather Bureau and Corps jointly derived the specifications, criteria, procedures, and methods. The specifications for SPH were reviewed several times after 1959, and the Weather Bureau issued updates. An additional update was published by NOAA in 1979 (September 1979). This update formed the basis for the SPH meteorological parameters used for this project.

As discussed in the section on the Lake Pontchartrain and Vicinity Project, a SPH storm was considered to have a recurrence interval of once in 100 years anywhere within Zone B. The probability of the SPH storm striking a smaller subzone, such as the Lake Cataouatche area, would be less. The frequency of the SPH at the site of a protective structure was assumed to be dependent upon its exposure and the direction of approach of the storm. Using observed high water mark and stage data, combined with computed wind tide elevations using different CPIs, a surge frequency curve was constructed that was representative of a reach of the hurricane protection system. The frequency curve also considered statistics on the critical direction of approach. The frequency of the computed wind tide elevations was adjusted based on the percentage of each direction followed by historic hurricanes. The probabilities of equal stages for both groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions.

Probable Maximum Hurricane

The probable maximum hurricane was not used in the analysis.

3.2.3.5. Lake Cataouatche (References 25 and 26)

Introduction

This area consists of approximately 10 miles of levee and 2 miles of floodwalls as shown in Figure 31. Vicinity map of area is shown on following page.

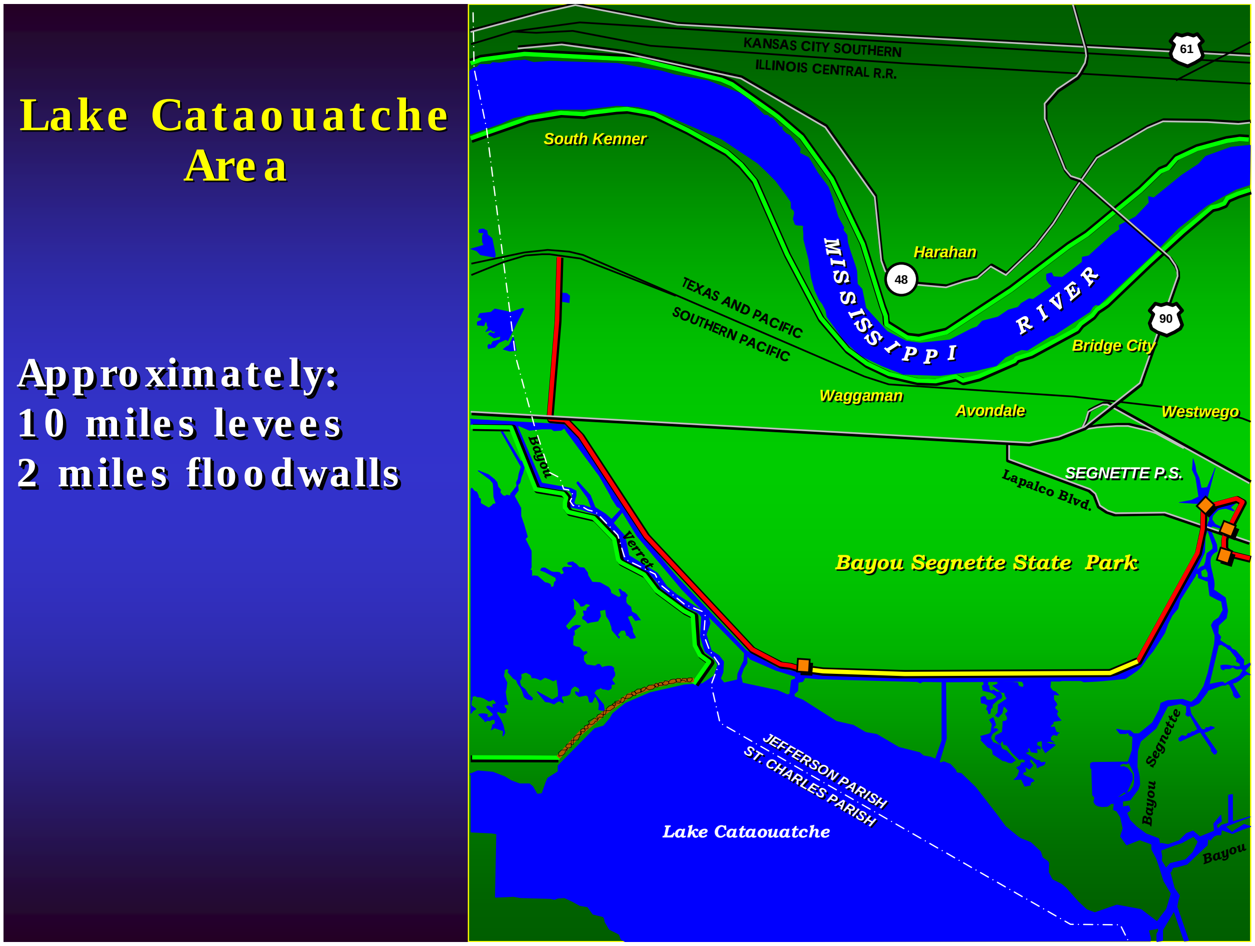


Figure 31. Lake Cataouatche project features.

Pre-Katrina

Construction in this area started in 2000. Before Hurricane Katrina, only one construction contract was completed in this area. Another one was under construction, but the contractor was terminated for default in 2005. Currently, the Corps of Engineers is waiting for the surety to determine their plans regarding the contract takeover. Remaining work in this area consists of first enlargement levee or floodwall contracts and future second enlargement levee contracts.

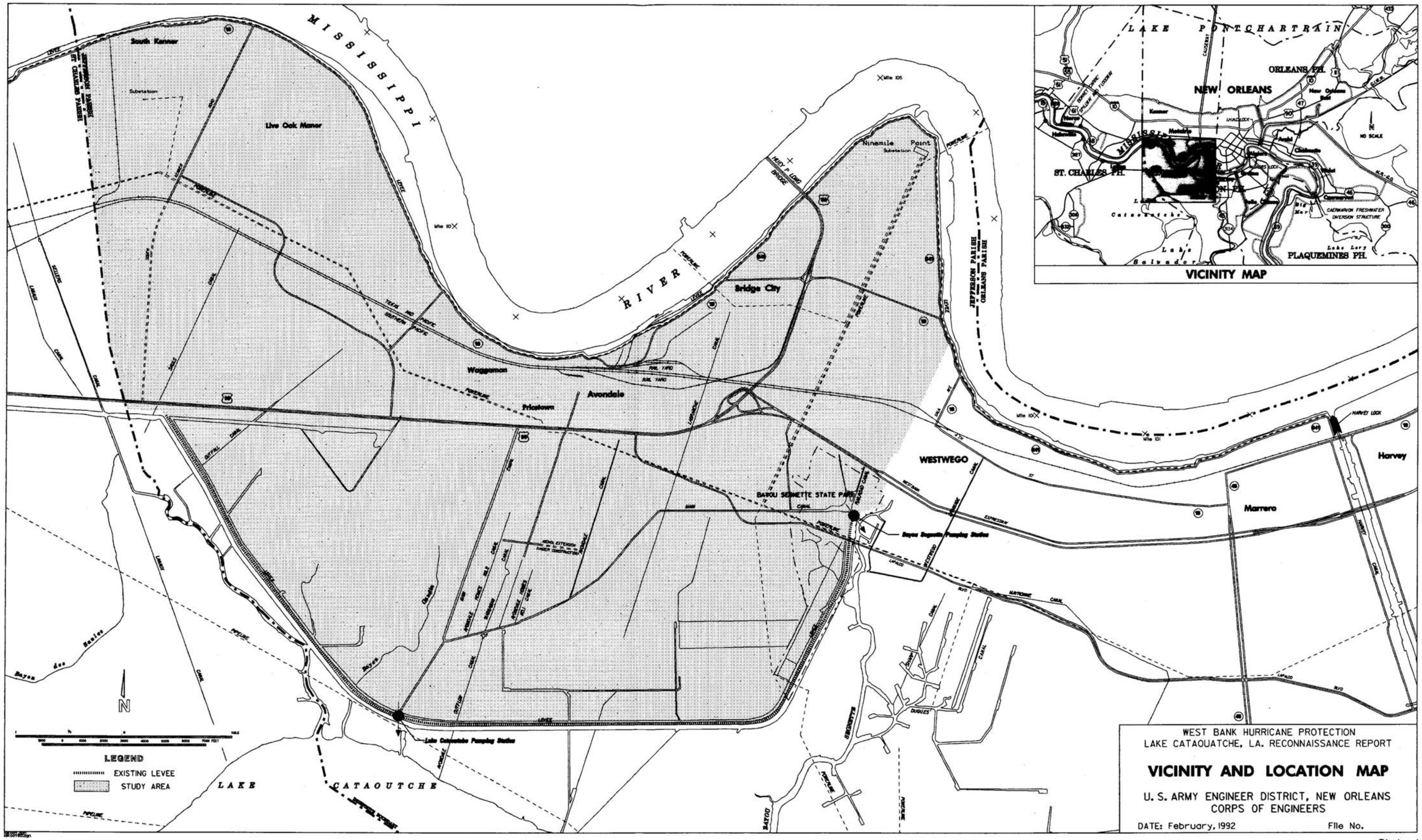

MISSISSIPPI
RIVER
South Kenner
Live Oak Manor
Ninemile Point
Bridge City
Waggaman
Avondale
Bridgetown
WESTWEGO
Harvey
Marrero
BAYOU SEGNETTE STATE PARK
Bayou Segnette Pumping Station
Lake Cataouatche Pumping Station
HARVEY LOCK
JEFFERSON PARISH
ORLEANS PARISH
LAKE CATAOUTCHE
LAKE PONTCHARTRAIN
NEW ORLEANS
ORLEANS PH.
ST. CHARLES PH.
PLAQUEMINES PH.
CAERNARVON FRESHWATER DIVERSION STRUCTURE
NO SCALE
VICINITY MAP
LEGEND
EXISTING LEVEE
STUDY AREA
N
WEST BANK HURRICANE PROTECTION
LAKE CATAOUATCHE, LA. RECONNAISSANCE REPORT
VICINITY AND LOCATION MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE: February, 1992
File No.

Plate 1

Hydrology and Hydraulics.

For Lake Cataouatche area, the design hurricane characteristics are shown in Table 35; the design tracks are shown on Figure 32. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 35
Design Hurricane Characteristics

Location	Track	CPI in.	Radius of Maximum Winds, nautical miles	Forward Speed, knots	Maximum Wind Speed[1] mph	Direction of Approach
Lake Cataouatche	C	27.4	30	11	100	South

[1] Wind speeds represent a 5-minute average 30 ft above ground level.

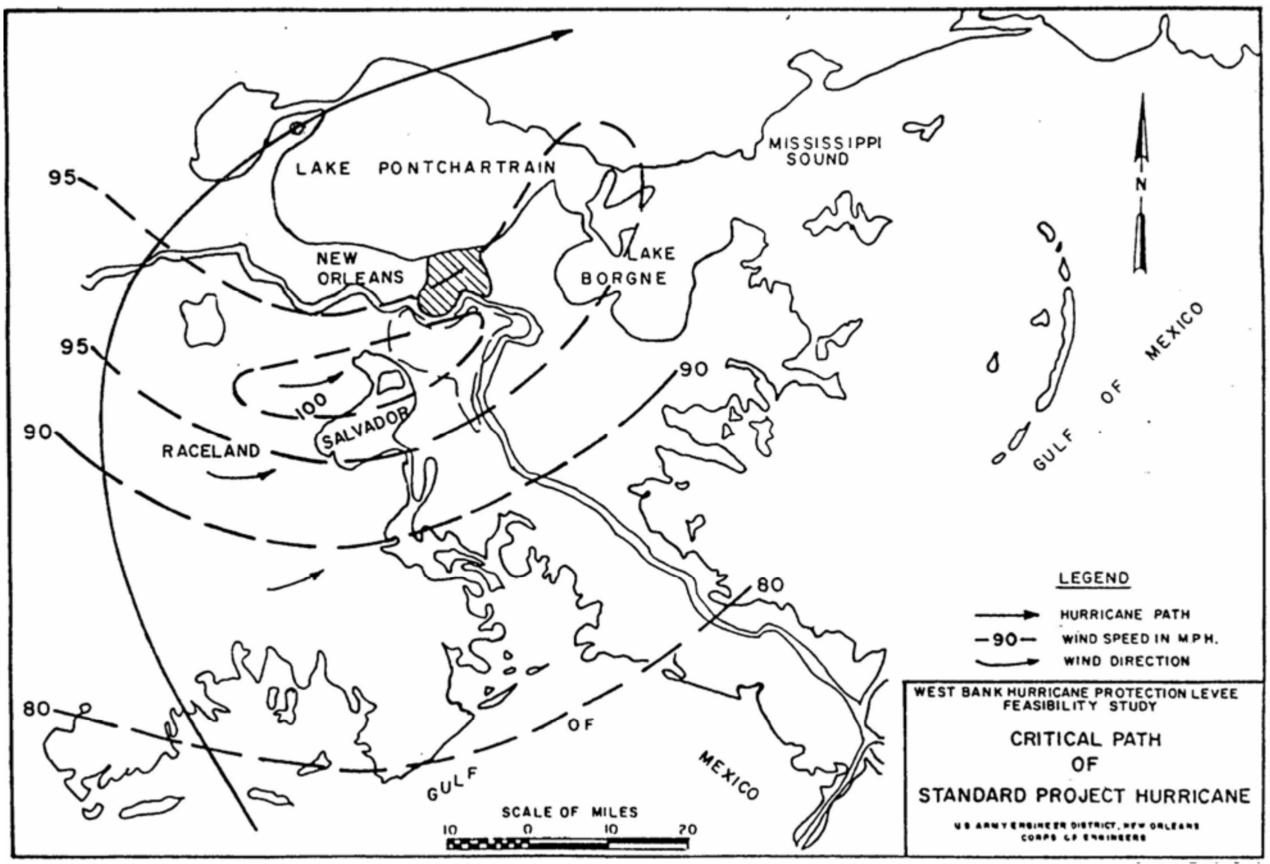


Figure 32. Critical path of standard project hurricane.

Surge. Wind tide levels for Lake Cataouatche area were computed using the same methodology as used for Lake Pontchartrain Lakefront for Orleans East Bank, plus several additional steps.

Where the coastline is characterized by a coastal bay separated from the gulf by an offshore barrier island, such as Grand Isle, or a shoal, an additional step was added to verify hurricane tides. Water surface elevation was transposed from the gulf side to the bay side of the island.

Table 36
Verification of Hurricane Surge Heights

Location	Surge Adjustment Factor, Z	September 1915		September 1947		September 1956	
		Observed ft MSL	Computed ft MSL	Observed ft MSL	Computed ft MSL	Observed ft MSL	Computed ft MSL
Grand Isle – front side	0.80	9.0	8.8	3.9	4.1	-	-
Grand Isle – back side	0.80	-	-	8.0	7.8	-	-
Manila Village	0.48	8.0	8.5	-	5.1	-	-

Major hurricane damage would result from induced effects on Lake Salvador and Lake Cataouatche. As with Lake Pontchartrain, the wind tide level for Lake Salvador and Lake Cataouatche is the sum of the surge, setup, tide, and runoff from rainfall. Hurricane surge values were routed from the bay side of the coast to Lake Salvador using the same methodology as that performed for routing surge from Lake Borgne into Lake Pontchartrain. Hourly incremental setup values were computed at Manila Village from general wind tide equation. Stage-area curves were developed for a schematized conveyance channel between Manila Village and Lake Salvador basin, which includes Lake Salvador, Lake Cataouatche, and adjacent marsh. Rainfall calculated by methods described in Weather Bureau reports (15, 16). A moderate rainfall of 8.5 in. over 24 hours was used for the Westwego to Harvey analysis. The surge from Hurricane Betsy was routed by the procedure as a verification of the procedure. Using this procedure resulted in a stage of 3.05 ft MSL, which compared favorably with 3.35 ft MSL, for the location Bayou Barataria at Lafitte. This location was assumed to be representative stage of Lake Salvador basin.

The average wind speed and average depth were determined from isovel and hydrographic charts furnished by NWS. The same setup and setdown equations for Lake Pontchartrain were used.

The methodology used for Chalmette Extension to route the wind tide level from the surge reference line to the location of the protection system was also used for the Lake Cataouatche portion of the West Bank and Vicinity project. A weighted mean decrease in surge heights inland at a rate of 1 foot per 2.75 miles was used. Table 37 shows the wind tide level at the surge reference line and at the Harvey Canal.

Table 37
Wind Tide Levels

Location	Wind Tide Level, Surge Reference Line, ft NGVD	Wind Tide Level at Levee Location, ft NGVD
Harvey Canal	9.0	7.5

Some additional modeling was also performed; four analytical models were used. HURWIN, Tropical Storm Planetary Boundary Wind Model, was used to generate wind-field input into ADCIRC. This model was run to obtain wind fields for Track C, the actual track of the 1915 hurricane. The model was then calibrated to actual results of the 1915 hurricane.

ADCIRC, Advanced Circulation Model, was used to obtain tidal and storm surge condition at the boundary of the WIFM model. PBLWIND, Planetary Boundary Layer Model, was used to interpolate wind fields from ADCIRC to WIFM. WIFM, WES Implicit Flooding Model, was used to compute tidal circulation and storm surge propagation. WIFM was used because ADCIRC did not have wetting and drying capability. WIFM was calibrated to Hurricane Betsy.

The WIFM model was adjusted to account for future land loss due to subsidence and an estimated sea level rise of 0.2 feet per 50 years. Model results indicated an increase in wind tide level of 1.0 ft by the year 2040.

Waves. For the Lake Cataouatche, some levees and floodwalls would be sheltered from storm generated runup; small locally generated waves could occur. These small waves would be likely to occur along Bayou Segnette, Bayou Verret, and South Kenner Road. Wave runup for all levees and floodwalls was calculated using methodology described in 1984 *Shore Protection Manual*.

Summary. Table 38 contains maximum surge or wind tide level, wave, and design elevation information.

Table 38
Wave Runup and Design Elevations (Transition zones not tabulated – Governing Report is listed)

Location	Report	Average Depth of Fetch, ft	Significant Wave Height, Hs ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level ft	Runup Height ft	Freeboard ft	Design Elevation Protective Structure, ft
Lake Cataouatche	PAC Study, December 1996	NA[1]	NA	NA	7.5 NGVD	3.0	-	10.5 NGVD
Bayou Segnette	PAC Study, December 1996	NA	1.0	2.7	7.0 – 6.0 NGVD	3.0	-	10.0 – 9.0 NGVD
Bayou Verret	PAC Study, December 1996	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
South Kenner Road	PAC Study, December 1996	NA	1.0	2.7	4.5 NGVD	2.0	-	6.5 NGVD

[1] Not presented in the report.

Geotechnical.

The projects that make up the West Bank and Vicinity Levee are the Lake Cataouatche area, Westwego to Harvey Canal, and the East of Harvey Canal.

Geology. The study area is located in Jefferson Parish, Louisiana between the Mississippi River and Lake Cataouatche. Surface elevations in the study area range from +10 ft NGVD along the natural levees of the Mississippi River to near 0 ft NGVD in the marshes bordering Lake Cataouatche. The surface and shallow subsurface in the study area is composed of natural levee, marsh, swamp, interdistributary, and prodelta deposits. Specifically, the borings show that the entire area is overlain by 8 to 22 ft of swamp deposits generally consisting of organic clays, wood, and peat, with occasional sand and silt layers. Underlying swamp deposits are interdistributary deposits located between elevations +2 and -22 ft NGVD and are up to 40 ft thick. Interdistributary deposits generally consist of very soft, fat clay with occasional lenses of lean clay, silt, and silty sand. Prodelta deposits underlie interdistributary deposits between Stations 149+50 and 455+00. Prodelta deposits are found between elevations -30 and -55 ft NGVD and extend to an unknown depth. Prodelta deposits generally consist of homogeneous, medium clays with occasional lenses of silt and silty sand. Nearshore gulf deposits underlie interdistributary deposits between Stations 485+00 and 570+00. Nearshore gulf deposits are found at approximately -50 ft NGVD and extend to an unknown depth. These deposits generally consist of silty sand and sand with shell fragments and occasional lenses of silt and clay.

Foundation Conditions. The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to medium, with occasional layers of silt (ML) and lean clays (CL). Layers of organic clays, which typically display high moisture content, exist in many areas from the original ground surface down approximately to elevation -20 ft NGVD.

Field Exploration. Twelve general type borings were made along parts of the proposed alignment between March 1991 and April 1993. Four undisturbed type borings were made along parts of the proposed alignment during April 93.

Underseepage. Not used.

Pile Curves. Pile capacity curves were completed for a 12-in.-square concrete pile and a Class B timber pile, respectively. A FS of 3.0 is recommended if no pile tests are performed and with a pile test, a FS of 2.0 is recommended. K_C = 1.00 and K_T = 0.70 were used to complete data for the curves.

Stability of Levees. Existing conditions along the proposed alignment were estimated and the slopes and berm distances for the proposed levee were designed for the Q construction case. A FS of 1.3 is required for the levee stability.

Cantilever I-Wall. I-wall stability and required penetration were determined by the Method of Planes. A FS was applied to the soil parameters. For the friction angle, the FS was applied as follows:

$$\Phi_d = \tan\frac{-1(\tan\Phi_A)}{FS}$$

where

Φ_d = developed friction angle

Φ_A = available friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces was equated to zero and the summation of overturning moments was determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

Two I-wall designs were analyzed for this project. The first was for an existing sheet-pile wall adjacent on either side of the Lake Cataouatche Pumping Stations 1 and 2 between approximate Stations 307+00 to 310+00. Since this reach is subject to wave loads, the sheet pile was analyzed for the following design cases.

Note: There is a significant wave load on the sheet-pile wall:

Q-Case

FS = 1.5 with static water at still-water level (swl)

FS = 1.25 with static water at swl plus wave load

FS = 1.0 with static water at swl plus 2 ft

S-Case

FS = 1.2 with static water at swl plus wave load

General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

The S-Case was the governing design case for the pumping station sheet-pile walls. For design results, see Plate F-6.

The other design reach runs from approximate Station 518+00 B/L to the Bayou Segnette floodwall. In this reach, the top of sheet pile will range from elevation 10.5 (near Station 518+00 B/L) to 9.5 (near Bayou Segnette pump station). The crown of the levee will range from elevation 5.5 ft NGVD (near Station 518+00 B/L) to 5.0 ft NGVD. (near Bayou Segnette pump station).

The following design cases were analyzed for determining required penetration for the levee/I-wall in this reach.

Note: There is no significant wave load on I-wall:

Q-Case

FS = 1.5 with static water at swl

FS = 1.0 with static water at swl plus 2 ft

General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

The 3-to-1 penetration to head ratio was the governing design case for the proposed levee/I-wall. However, to compensate for future flood conditions (general land subsidence and sea level changes), additional sheet-pile penetration has been incorporated into the design. See Plate F-7.

T-Type Walls. The T-type walls supported on bearing piles will provide protection adjacent to I-type gates and pumping plants.

Erosion. No protection is considered necessary along the levee other than seeding the levees. Any erosion caused by hurricane floods will be restored under normal maintenance.

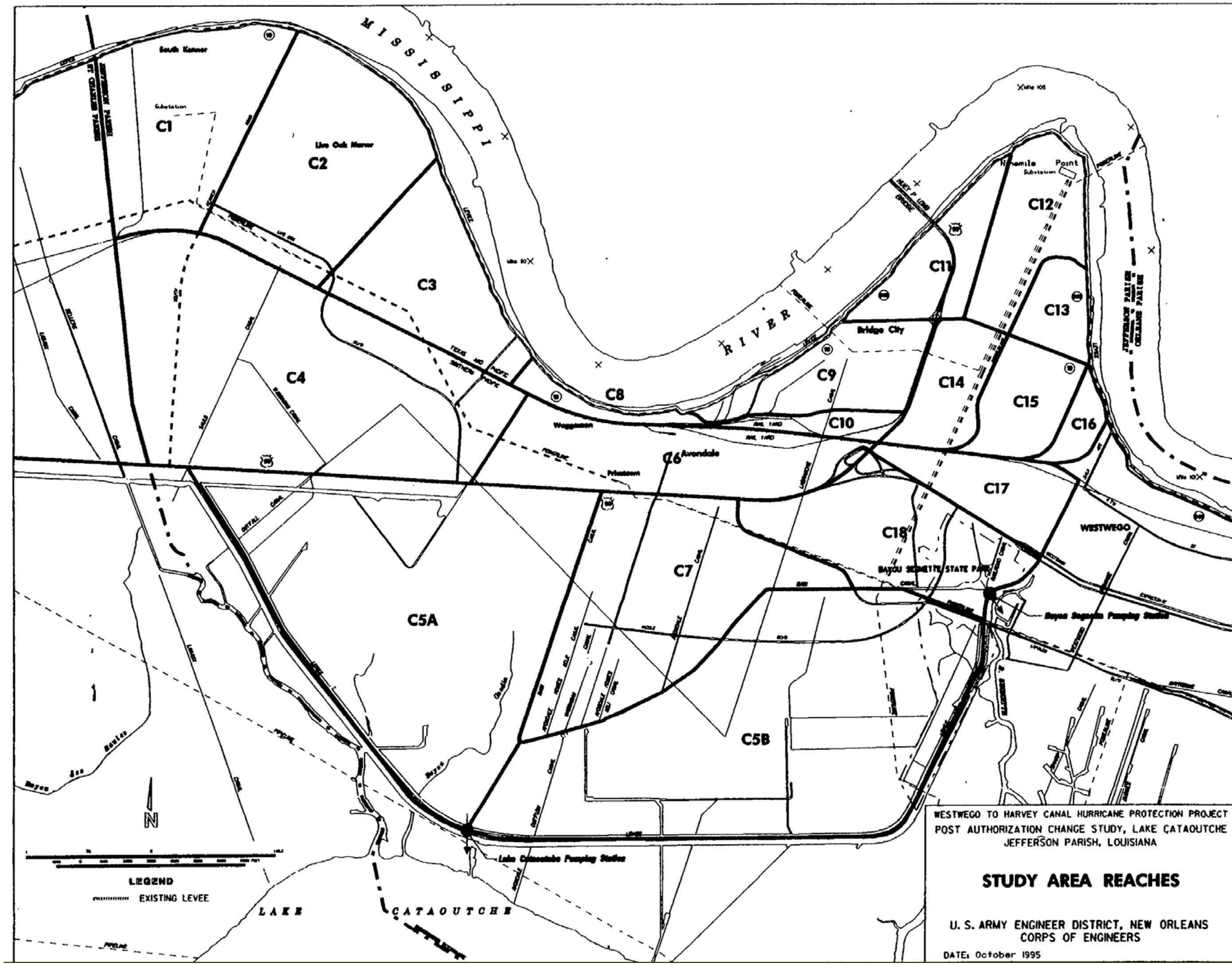
Structural – Lake Cataouatche, LA (Reference 39). Image of area shown on following page.

General. The structural features in the Lake Cataouatche area consist of approximately 10 miles of levees and 2 miles of floodwalls. The floodwall features include I-walls, I-wall/levee combinations; pile supported inverted T-walls, and swing gate closure structures at street crossings. The following is a summary of the pertinent structural criteria for these structures.

I-Type Floodwalls. In the design of the I-walls, the loading cases that were considered as follows:

- Case I. Q-Case, FS = 1.5 with water to swl
- Case II. Q-Case, FS = 1.0 with water to swl + 2 ft.

Minimum penetration to head ratio equal to 3:1


MISSISSIPPI
RIVER
South Kenner
C1
C2
Live Oak Manor
C3
C4
C5A
C5B
C6
C7
C8
C9
C10
C11
C12
C13
C14
C15
C16
C17
C18
Nine Mile Point
Bridge City
Waggaman
Avondale
Westwego
Bayou Segnette State Park
Bayou Segnette Pumping Station
Lake Cataouatche Pumping Station
LAKE CATAOUTCHE
LEGEND
EXISTING LEVEE
N
WESTWEGO TO HARVEY CANAL HURRICANE PROTECTION PROJECT
POST AUTHORIZATION CHANGE STUDY, LAKE CATAOUTCHE
JEFFERSON PARISH, LOUISIANA
STUDY AREA REACHES
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE: October 1995

T-Type Floodwalls. The T-wall consists of a reinforced concrete stem on a monolithic concrete base of varying width supported on prestressed concrete piles, except for the Fronting Protection T-wall at the Segnette Pumping Station which will be founded on steel H-piles. The T-walls were designed for the following loading conditions:

- Case I – Static water pressure with water to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force
- Case II – Static water pressure with water to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force
- Case III – Static water pressure with water 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
- Case IV – Static water pressure with water 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
- Case V – No water, no wind
- Case VI – No water, wind from protected side (75 percent forces used)
- Case VII – No water, wind from floodside (75 percent forces used).

Additionally, at the Segnette Pumping Station Front End Protection T-wall, the tension loads from the existing tie rods are included in the above load cases.

Swing Gates and Gate Monoliths. Gate monoliths with swing gates are to be constructed at street crossing in lieu of I-walls. The gate structures are to be designed for the following load conditions:

- Case I – Gate closed, static water pressure with water to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force
- Case II – Gate closed, static water pressure with water to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force
- Case III – Gate closed, static water pressure with water 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
- Case IV – Gate closed, static water pressure with water 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
- Case V – Gate open, no water, no wind, truck on protected side edge of base slab
- Case VI – Gate open, no water, no wind, truck on floodside edge of base slab
- Case VII – Gate open, no water, wind from protected side, truck on floodside edge of base slab (75 percent forces used)
- Case VIII – Gate open, no water, wind from floodside, truck on protected side edge of base slab (75 percent forces used).

Sources of Construction Materials.

Sheet Pile. Generally, the sheet-pile sections specified during advertisement were used for construction. However, sheet-pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below is a tabulation of sheet-pile sections.

Lake Cataouatche	
Bayou Segnette State Park Floodwall	**
** Information not found at the time of publication.	

Levee Material (Lake Cataouatche Area). Borrow for embankment construction will come from multiple sources: (1) Borrow will be trucked from David Pond Project, (2) from the excavation of the floodside canal, (3) a 13.6-acre borrow area adjacent to Bayou Segnette State Park, and (4) from widening of the interior drainage canal.

As-built Conditions

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation.) – West Bank and Vicinity – Modifications and Changes.

DACW29-00-C-0042. Westbank – Vicinity of New Orleans, Hurricane Protection Project, LA, Lake Cataouatche, Segnette State Park Floodwall, Jefferson Parish, LA.

Modification was issued to allow the contractor to dress down the levee slopes and crown width in certain reaches in order to give the contractor a borrow source at the job site for required embankment/structural backfill work.

Inspection During Original Construction, QA/QC, State What Records Are Available.

See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

DACW29-00-C-0042. Westbank – Vicinity of New Orleans, Hurricane Protection Project, LA, Lake Cataouatche, Segnette State Park Floodwall, Jefferson Parish, LA.

Attached are preparatory phase reports.

Inspection and Maintenance of Original Construction

The West Bank and Vicinity Hurricane Protection Project is one of the more recent hurricane protection systems in the New Orleans District. So far, no structures have been brought under the Periodic Inspection Program, but Annual Compliance Inspections have existed for some time for these local works turned over for operation and maintenance under the West Jefferson Levee District.

Annual Compliance Inspection (i.e., trees, etc.).

This district is responsible for maintaining 20.2 miles of levee on the west bank of the Mississippi River, and 40 miles of back levees, which are being upgraded under the West Bank Hurricane Protection Project, in Jefferson Parish. In 2004, it was stated, "Although the hurricane protection levees are not totally complete, we conducted an interim joint inspection of the system on 8 June 2004." The levees and floodwalls are in excellent condition. An "Acceptable" rating is assigned.

Periodic Inspections.

There are no structures under the Periodic Inspection Program in the Cataouatche area, of the West Bank and Vicinity Hurricane Protection Project.

Other Features – Jefferson West Bank, Lake Cataouatche.

Brief Description.

The primary components of the hurricane protection system for the Jefferson West Bank, Lake Cataouatche subarea are described above, namely the levees and floodwalls designed and constructed by the Corps. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

Pre-Katrina Conditions.

According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, and outfall pump stations were in good condition and prepared for high inflows from rainfall prior to 29 August 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

Interior Drainage System.

Overview. The Jefferson West Bank, Lake Cataouatche subarea contains about 30 square miles and generally slopes north to south from the Mississippi River. It is sparsely developed except for areas along the Mississippi River that are highly developed. Many features are typical of large urban cities in the United States, and some features are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations that pump the water into the Cataouatche Canal, Lake Cataouatche, and Bayou Segnette. No stormwater is pumped into the Mississippi River.

The entity responsible for local drainage in the Jefferson West Bank polder is Jefferson Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson West Bank. The land generally falls south from the Mississippi River.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 85 subbasins. Pump station information is presented below.

The canals are open and most are grass-lined. The canals and ditches not only collect storm-water from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10 percent storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 in./hr. Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, and canals. There are criteria for new developments to use stormwater detention in order to offset downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. There are no Southeast Louisiana (SELA) Urban Flood Control Projects in this subpolder.

Pumping stations.

Jefferson Parish Lake Cataouatche

Jefferson Parish is located west of the city of New Orleans and borders the west side of Orleans Parish. Figure 33 is a map of Jefferson Parish with the pump stations that were studied identified by red bullets. Jefferson Parish is separated by the Mississippi River into East and

West Banks. The East Bank pump stations are connected by a grid of canals. The canals running east and west serve to equalize flow between the major outfall canals, allowing rainwater to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into subbasins that, for smaller rainfall events, operate independently. However, over-bank flow does occur between adjacent subbasins for a 10-year event. This report examined six pump stations on the East Bank with a total of 36 pumps and 17 pump stations on the West Bank with a total of 65 pumps.

Figure 33 is a map showing the Jefferson Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

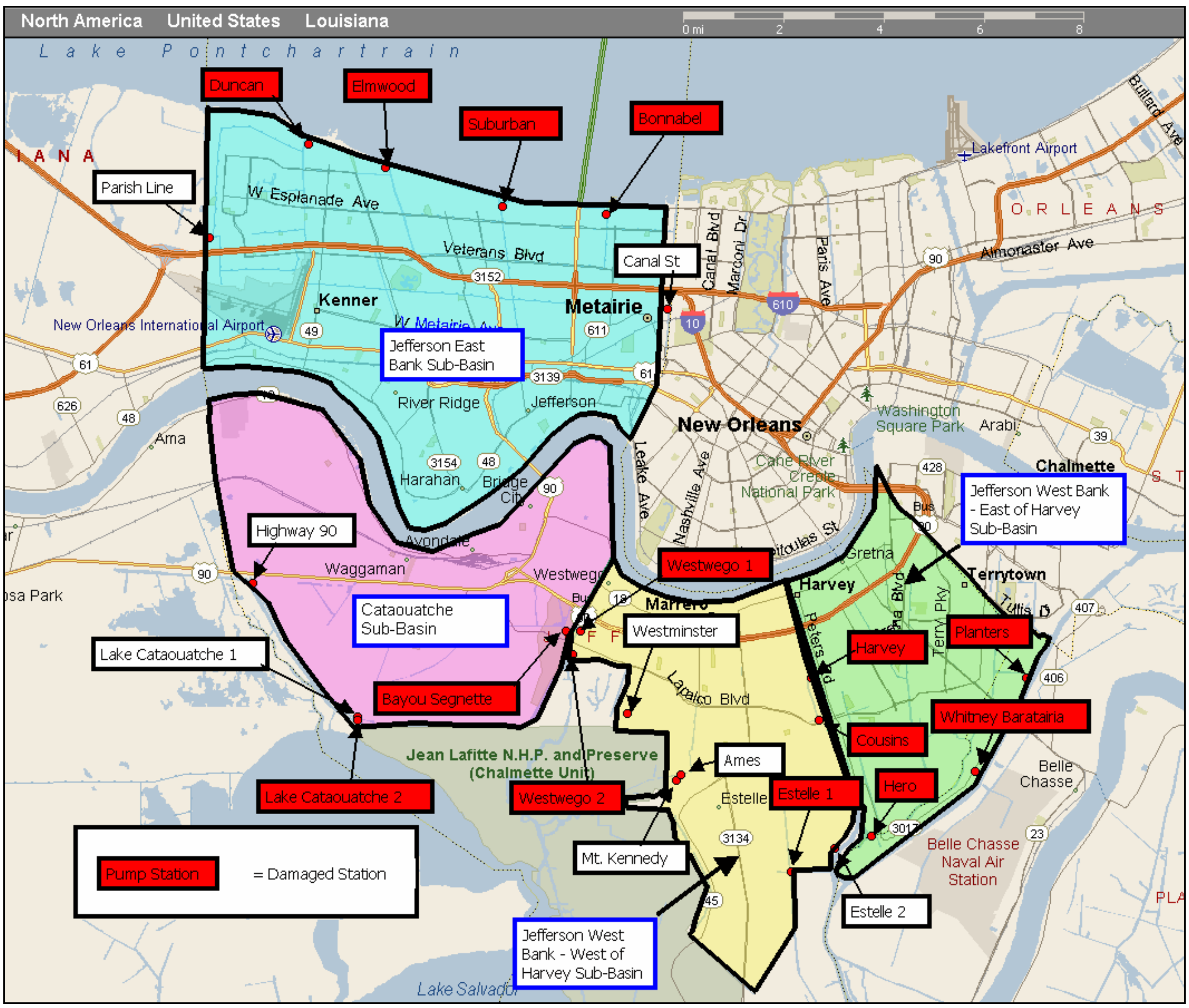


Figure 33. Jefferson Parish pump station locations.

Table 39 contains a summary of pump stations by drainage basin in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 39
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank – East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

West Bank – West of Harvey (Cataouatche)

The West Bank-West of Harvey Cataouatche drainage basin has four significant pump stations, which are briefly described below. Volume VI provides more detailed information. The basin is bordered by the Mississippi River on the north and east sides. Its drainage system includes the river, Lake Cataouatche, and the Main, Waggaman, and Bayou Segnette Canals.

Lake Cataouatche No. 1
Intake location: ....Main Canal
Discharge location: ....Lake Cataouatche
Nominal capacity: ....500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	250	n/a	Diesel	Vertical
2	250	n/a	Diesel	Vertical

Lake Cataouatche No. 2
Intake location: ....Main Canal
Discharge location: ....Lake Cataouatche
Nominal capacity: ....600 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	1982	Diesel	Vertical
2	300	1982	Diesel	Vertical

Highway 90
Intake location: .... Waggaman Canal
Discharge location: .... Outer Cataouatche Canal
Nominal capacity: .... >90 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	45	n/a	Electric	n/a
2	n/a	n/a	Electric	n/a
3	45	n/a	Electric	n/a

Bayou Segnette
Intake location: .... Main Canal
Discharge location: .... Bayou Segnette
Nominal capacity: .... 2,156 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
New 1	610	n/a	Diesel	n/a
New 2	610	n/a	Diesel	n/a
1	156	n/a	Diesel	Vertical
2	156	n/a	Diesel	Vertical
3	156	n/a	Diesel	Vertical
4	156	n/a	Diesel	Vertical
5	156	n/a	Diesel	Vertical
6	156	n/a	Diesel	Vertical

Levees and Floodwalls.

MRL **-** MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are no floodwalls that are part of the MRL Project in this reach.

Non-Corps - Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3.6. Westwego to Harvey

Introduction

As shown in Figure 34, this area consists of approximately 22 miles of levee and 2 miles of floodwalls in the Westwego area along the existing V-Levee alignment to the vicinity of the old Estelle Pumping Station and along the existing Harvey Canal-Bayou Barataria Levee tying into the floodwall at the Cousins Pumping Station, then from the pump station to the navigable sector floodgate complex which is to be constructed in Harvey Canal near the Cousins Pumping Station. This area was the first area of the project authorized, and as such, has the most construction completed.

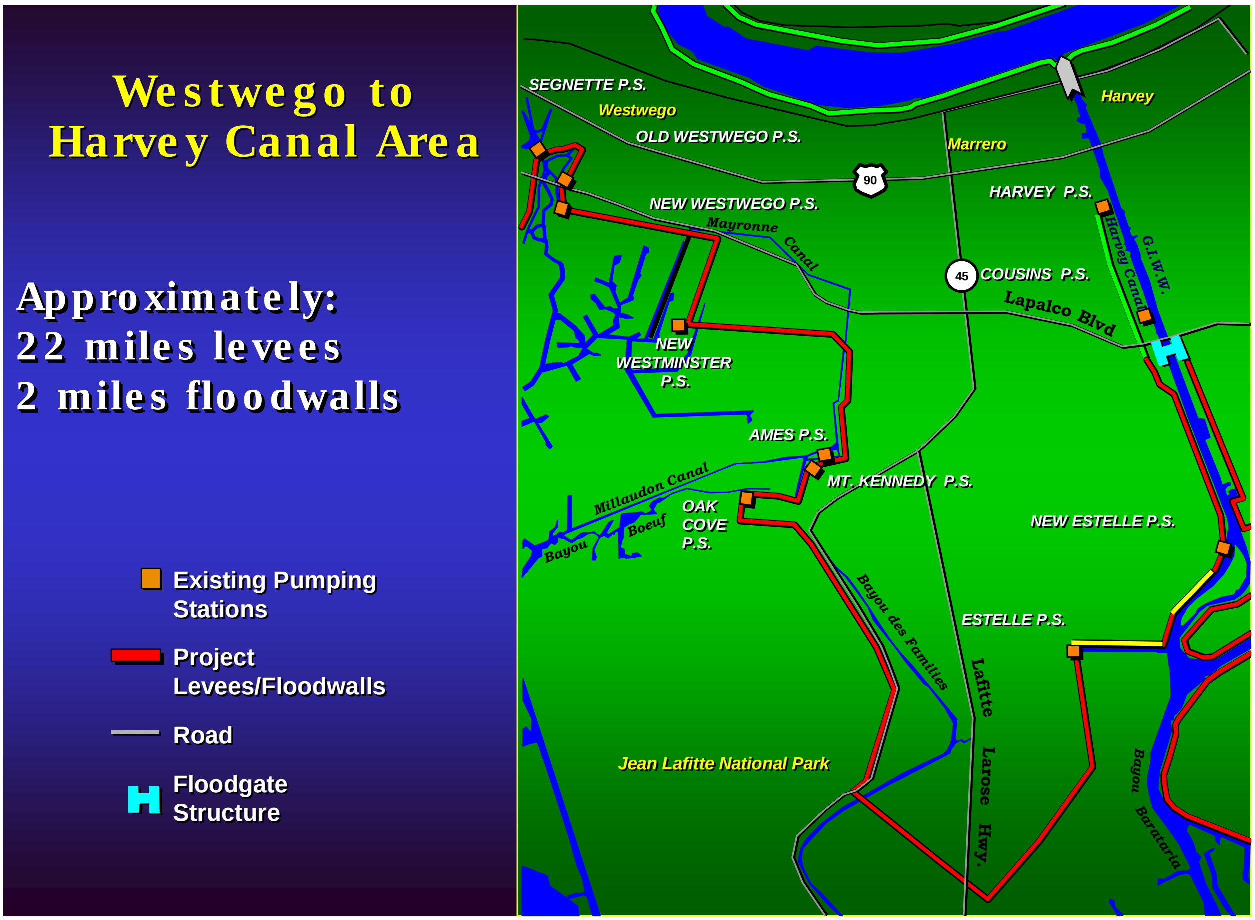


Figure 34. Westwego to Harvey Canal area project features.

Pre-Katrina

Construction in this area started in 1991. Before Hurricane Katrina, all of the first enlargement construction contracts had been completed, except for one, which was under construction. This contract is still ongoing, scheduled for completion this year. One second enlargement contract also had been completed. There were a total of 15 construction contracts that were completed before Hurricane Katrina. Remaining work in this area consists of the future levee enlargements, as most of these contracts have third enlargement levee contracts.

Design Criteria and Assumptions - Functional Design Criteria

Hydrology and Hydraulics.

For Westwego to Harvey, the design hurricane characteristics are shown in Table 40; the design tracks are shown on Figure 35. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 40
Design Hurricane Characteristics

Location	Track	CPI in.	Radius of Maximum Winds, nautical miles	Forward Speed knots	Maximum Wind Speed[1] mph	Direction of Approach
Westwego to Harvey	-	27.6	30	6	100	South-Southwest

[1] Wind speeds represent a 5-minute average 30 ft above ground level.

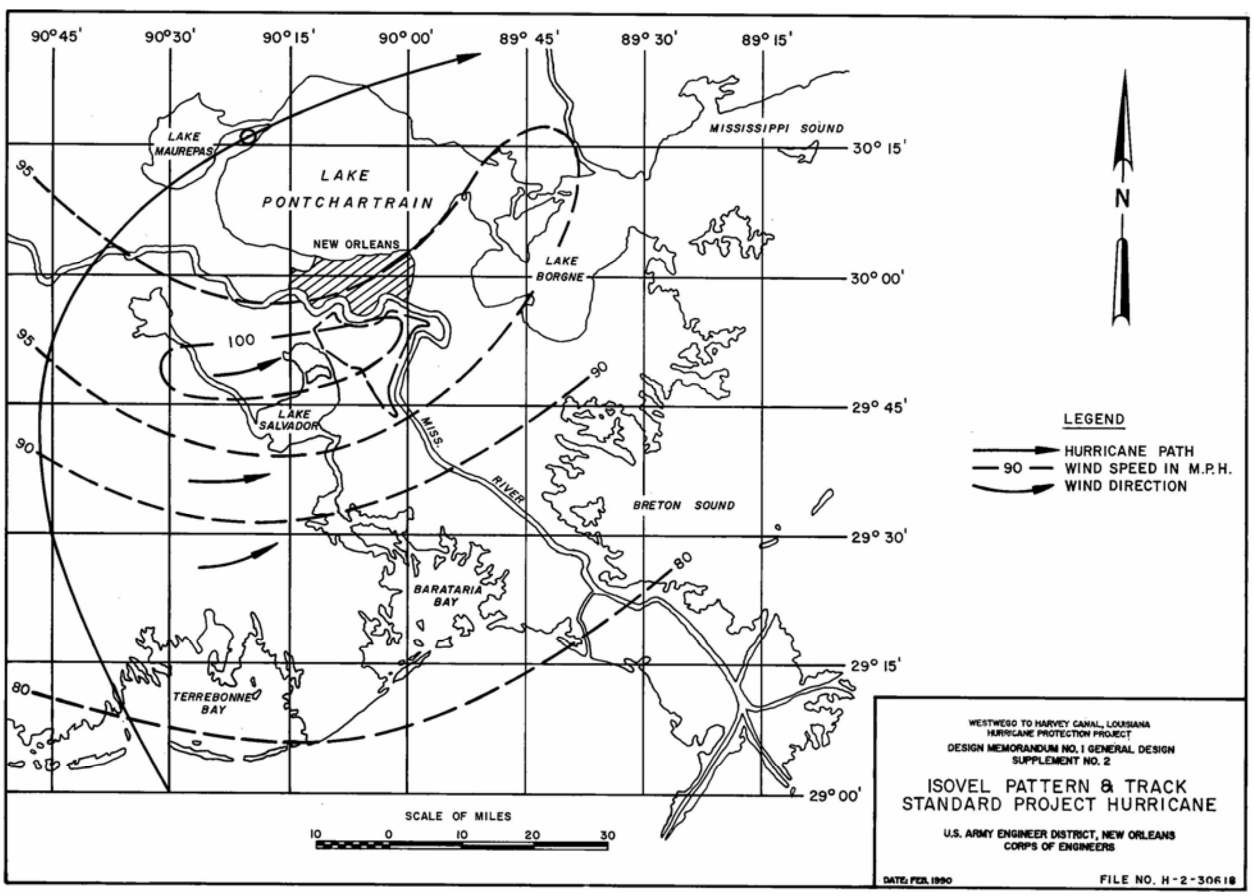


Figure 35. Isovel pattern and track standard project hurricane.

Surge. Wind tide levels for Westwego to Harvey area were computed using the same methodology as used for Lake Pontchartrain Lakefront for Lake Cataouatche, without the numerical modeling. In addition, a future condition analysis was not performed.

Waves. The levee reach from Bayou Segnette to Highway 3134 would be subject to waves generated in Lakes Salvador and Cataouatche. Wave runup was calculated using the methodology described in Orleans East Bank. The remaining reaches in the Westwego to Harvey reach were not considered to be subject to waves.

Summary. Table 41 contains maximum surge or wind tide level, wave, and design elevation information.

Table 41
Wave Runup and Design Elevations (Transition zones not tabulated – Governing DM is listed)

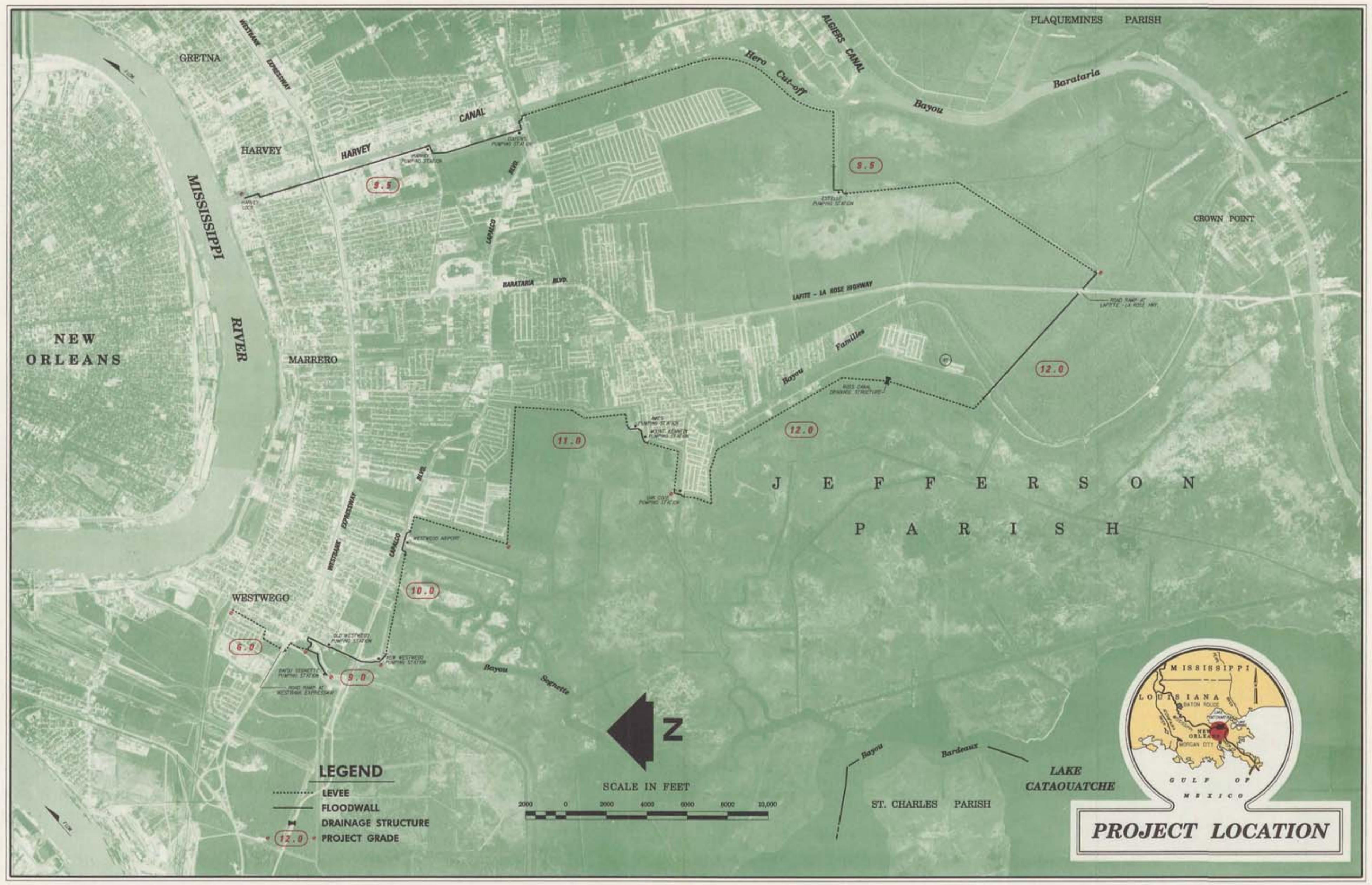
Location	DM	Average Depth of Fetch, ft	Significant Wave Height, Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, ft	Runup Height ft	Free-board ft	Design Elevation Protective Structure, ft
Bayou Segnette to Dugues Canal	DM01 Sup02 Feb 1990	5.5	3.0	4.0	7.0 NGVD	3.0	-	10.0 NGVD
Dugues Canal to Estelle Canal	DM01 Sup02 Feb 1990	6.5	3.3	4.0	8.0 NGVD	3.0	-	11.0 NGVD
Estelle Canal to Bayou Des Familles	DM01 Sup02 Feb 1990	7.5	3.7	4.1	9.0 NGVD	3.0	-	12.0 NGVD
Bayou Des Familles to Highway 3134	DM01 Sup02 Feb 1990	2.5	2.1	4.1	9.0 NGVD	3.0	-	12.0 NGVD
Highway 3134 to Apex of V-Levee	DM01 Sup02 Feb 1990	-	-	-	9.0 NGVD	-	3.0	12.0 NGVD
Apex of V-Levee to Harvey Canal	DM01 Sup02 Feb 1990	-	-	-	7.5 NGVD	-	2.0	9.5 NGVD

Geotechnical.

This report covers the soil investigation and design of approximately 82,000 ft of improved levees and 25,800 feet of floodwalls (Reference 33). Image of area shown on following page.

The project alignment has been divided into five design reaches due to a variation in soil strength, stratification and required protection elevations. Many of these design reaches were divided into subreaches because of varying ground elevations, levee crown transitions, and soil strength variation. Area and reaches are as follows:

Design Reaches		**Base Line Stations**		**Base Line Stations**
Westwego and Westminster Levee	5 subreaches	0+00	To	256+42
Oak Cove Levee	3 subreaches	256+45	To	420+96
Highway 45 Levee	1 subreach	420+97	To	572+16
V-line Levee	3 subreaches	572+17	To	804+32
Harvey Canal Levee	3 subreaches	804+33	To	1072+00



WESTWEGO TO HARVEY CANAL, LA.

HURRICANE PROTECTION PROJECT

JEFFERSON PARISH, LOUISIANA

C·A·D

WESTWEGO TO HARVEY CANAL, LOUISIANA
HURRICANE PROTECTION PROJECT

DESIGN MEMORANDUM NO. 1, GENERAL DESIGN
SUPPLEMENT NO. 2

PLAN OF PROTECTION

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

DATE: FEBRUARY 1990	DESIGN FILE: 30618SA45.DGN	PLOT SCALE: 1	FILE NO. H-2-30618

Geology. The project site is located on the deltaic plain portion of the Mississippi River Alluvial Plain. Specifically, the project is located on the northern edge of the Barataria basin on the western side of the Mississippi River. The Barataria basin is an interdistributary basin dominated by features which include natural levee ridges, crevasse-splay deposits, marsh, lakes and swamps. The eastern and northern edge of the basin is defined by the Bayou Lafourche natural levee ridge. The Gulf of Mexico constitutes the southern boundary. Elevations range from approximately +10 to +15 ft NGVD in the back swamp and lake areas.

The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to stiff. In many areas, organic clays (OH) and peat (PT) may be found in the top 20 ft and have a very soft consistency.

The V-line Levee, from Station 563+00 to Station 588+00 is underlain by large layers of silt (ML) and sand (SM) 5 to 10 ft below the surface. Thin strata of silt and sand are encountered at various other locations in the foundation.

Field Exploration. One hundred and fifteen borings were made along the proposed alignment. Of the 115 borings, 102 were obtained in 1976 by Eustis Engineering of Metairie, LA, at the request and authorization of the West Jefferson Levee District, the local project sponsor. The rest of the borings, 13, were obtained in 1988 and 1989 by the Corps; the majority of these were check borings. All Eustis borings were undisturbed (5-in. or 3-in. ID). Eight Corps borings were 5-in. undisturbed borings, and five were general type borings.

Underseepage. Not used.

Hydrostatic Pressure Relief. Not used.

Pile Foundation. Pile capacity curves were generated for various structures along the alignment (pumping stations, gates, T-walls, etc.).

Design single pile compression and tension capacities versus tip elevations for piles were determined for various locations.

Design data were determined for the Q and S shear strength. For piles in compression, a FS of 2.0 was applied to the shear strengths, and a conjugate stress ratio K_C of 1.0 was used in the S-Case for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths, and a K_t of 0.7 was used in the S-Case. In some instances, the Q-Case indicates the least pile capacities, and in other instances, the S-Case yields the least result. The minimum value was used for design.

Slope Stability. Using cross sections representative of existing conditions along the proposed alignment, the slopes and berm distances for the recommended levees were designed with borrow pies on the floodside except in the area of V-Line Levee from Station 660+00 to Station 800+00 where the borrow will be on the protected side.

The stability of the levees was determined by the Method of Planes using the design Q shear strengths as shown on the plots of soil data on Plates F107 through F111. A FS of 1.3 was required for the levee stability and a FS of 1.5 was the minimum required for failures into borrow pits and canals.

I-Type Floodwalls. I-wall stability and required penetration were determined by the Method of Planes. A FS was applied to the soil parameters. For the friction angle, the FS was applied as follows:

$$\Phi_d = \tan^{-1}\frac{(\tan\Phi_A)}{FS}$$

where

Φ_d = developed friction angle

Φ_A = available friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces was equated to zero and the summation of overturning moments was determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

Dynamic wave force was a design factor in the floodwalls design, from New Westwego Pumping Station (Station 69+94) to the V-vertex of the V-line Levee (Station 660+06). The results of hydraulic analysis indicate that these walls will be subjected to the pressure and forces implanted by a broken wave.

The following design cases were analyzed for determining required penetration:

Case I: No significant wave load on I-wall:

Q-Case

FS = 1.5 with static water at swl

FS = 1.0 with static water at swl + 2 ft

General: If the penetration of head ratio is less than 3:1, increase it to 3:1.

Case II: Significant wave load on I-wall:

Q-Case - Same as above plus

FS = 1.25 with static water to swl plus wave load

S-Case

FS = 1.2 with water to swl plus wave load

General: If the penetration to head ratio is less than 3:1, increase it to 3:1 or to that required by the S-Case, FS = 1. 5, whichever results in the least penetration.

T-Type Floodwalls. T-type floodwalls will be used in some areas. The T-type floodwall will support on a pile foundation of pre-stressed concrete or steel H–piles.

Structural.

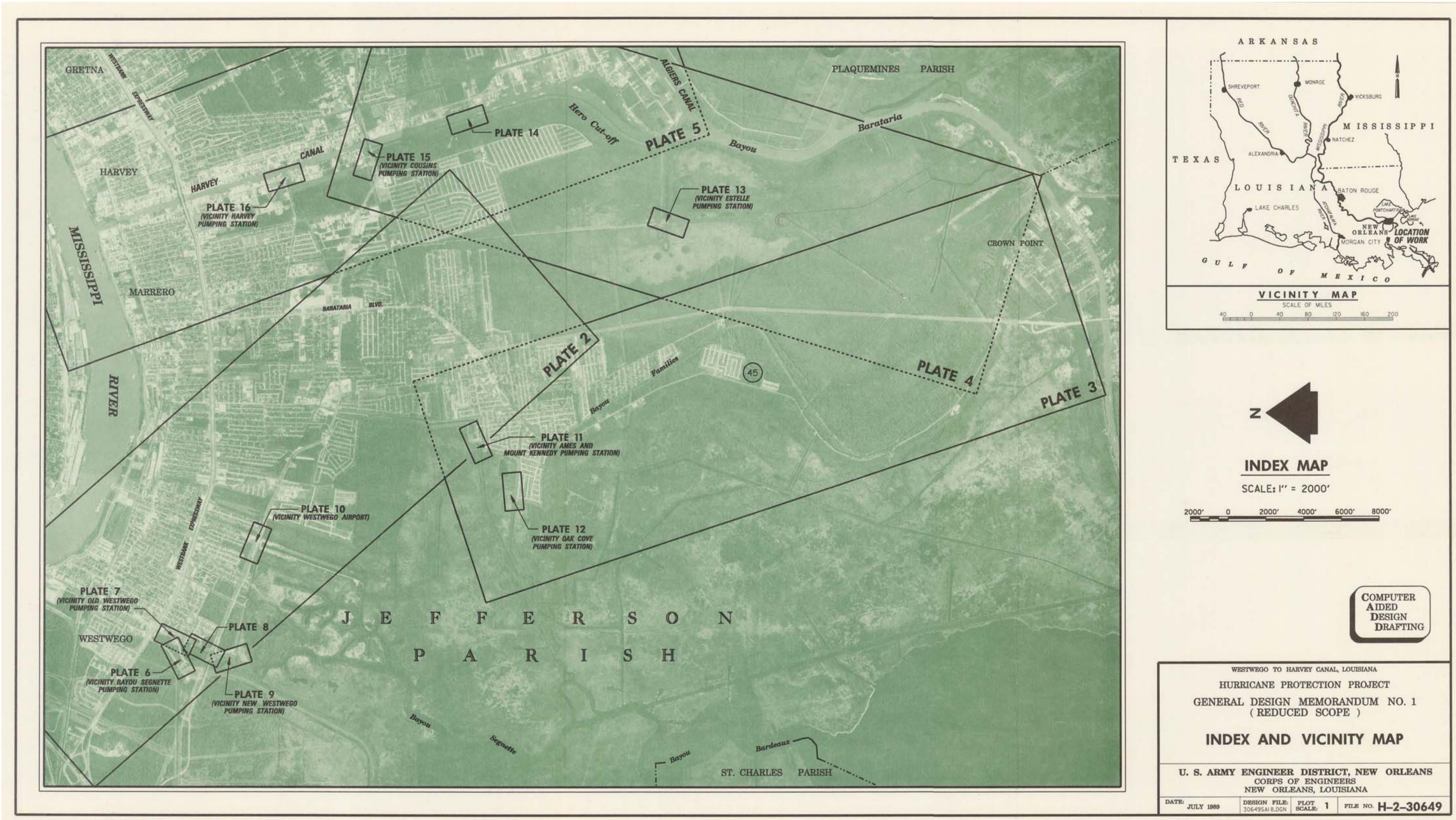
Westwego to Harvey Canal Area (References 32 and 33). Image of area shown on following page.

General. The fronting protection at each pumping station location will be raised to design elevation by either replacement of the existing protection with pile supported T-walls, or by increasing the height of the existing fronting T-walls or sheet-pile bulkheads. Gate monoliths will be constructed for street crossings in lieu of I-walls.

Structural Steel. The design of steel structures is in accordance with the requirements of the allowable working stresses recommended in “Working Stresses for Structural Design,” in EM 1110-1-2101, dated 1 November 1963, and amendment No. 2, dated 17 January 1972. The basic working stress for ASTM A-36 steel is 18,000 psi. Steel for steel sheet piling will meet the requirements of ASTM 328, “Standard Specifications for Steel Sheet Piling.”

Reinforced Concrete. The design of reinforced concrete structures is in accordance with the requirements of the strength design method of the current ACI building Code, as modified by the guidelines of “Strength Design Criteria for Reinforced Concrete Hydraulic Structures,” ETL 1110-2-312, dated 10 March 1988. The basic minimum 28-day compressive strength concrete will be 3,000 psi, except for prestressed concrete piling where the minimum will be 5,000 psi. For convenient reference, pertinent stresses are tabulated below:

f'_c	3,000 psi
fy (Grade 60 steel)	48,000 psi
Maximum flexural reinforcement	0.25 x balance ratio
Minimum flexural reinforcement	200/fy
f'_c (prestressed concrete piles)	5,000 psi
fu (prestressing strands, Grade 250)	250,000 psi
(prestressing strands, Grade 270)	270,000 psi



GRETNA
WESTBANK EXPRESSWAY
HARVEY
HARVEY CANAL
PLATE 16 (VICINITY HARVEY PUMPING STATION)
MISSISSIPPI RIVER
MARRERO
BARATARIA BLVD.
PLATE 15 (VICINITY COUSINS PUMPING STATION)
PLATE 14
Hero Cut-off
ALGIERS CANAL
PLATE 5
Bayou Barataria
PLAQUEMINES PARISH
PLATE 13 (VICINITY ESTELLE PUMPING STATION)
CROWN POINT
PLATE 2
PLATE 4
PLATE 3
Bayou Families
45
PLATE 11 (VICINITY AMES AND MOUNT KENNEDY PUMPING STATION)
PLATE 12 (VICINITY OAK COVE PUMPING STATION)
PLATE 10 (VICINITY WESTWEGO AIRPORT)
WESTBANK EXPRESSWAY
PLATE 7 (VICINITY OLD WESTWEGO PUMPING STATION)
PLATE 8
WESTWEGO
PLATE 6 (VICINITY BAYOU SEGNETTE PUMPING STATION)
PLATE 9 (VICINITY NEW WESTWEGO PUMPING STATION)
JEFFERSON PARISH
Bayou Segnette
Bayou Bardeaux
ST. CHARLES PARISH
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
NEW ORLEANS
MORGAN CITY
LOCATION OF WORK
GULF OF MEXICO
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
N
INDEX MAP
SCALE: 1" = 2000'
2000' 0 2000' 4000' 6000' 8000'
COMPUTER AIDED DESIGN DRAFTING
WESTWEGO TO HARVEY CANAL, LOUISIANA
HURRICANE PROTECTION PROJECT
GENERAL DESIGN MEMORANDUM NO. 1
(REDUCED SCOPE)
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DATE: JULY 1989
DESIGN FILE: 30649SAI8.DGN
PLOT SCALE: 1
FILE NO. H-2-30649
PLATE 1

I-Type Floodwall. The I-walls consist of steel sheet piling driven into the existing ground and, in some cases, into a new embankment. In the design of the I-walls, the loading case to be considered will be as follows:

- Q-Case, FS =1.5 with water to swl
- Q-Case, FS =1.25 with water to swl plus wave load
- Q-Case, FS = 1.0 with swl + 2 ft freeboard
- S-Case, FS = 1.2 with water to swl plus wave load

No water, lateral soil pressure (where applicable)

Tied-Back Sheet-Pile Walls. The tied-back sheet-pile walls will consist of steel sheet piling driven into existing ground and anchored with tie rods to a steel pipe, pile, or H-pile deadman. The upper portion of the sheet piling will be capped with concrete. The required sheet-pile penetration and maximum bending moment will be determined by applying a FS of 1.2 to the soil parameters. The required anchor force will be determined by applying a FS of 1.0 to the soil parameters.

T-Type Floodwall. The T-wall will consist of a reinforced concrete stem on a monolithic concrete base of varying width supported on precast, prestressed concrete piles or H-Piles. The base of the T-wall will be constructed on a 4-in. concrete stabilization slab. A continuous steel sheet-pile wall will be provided beneath the base for seepage cutoff purposes. These walls will be designed for the following load conditions:

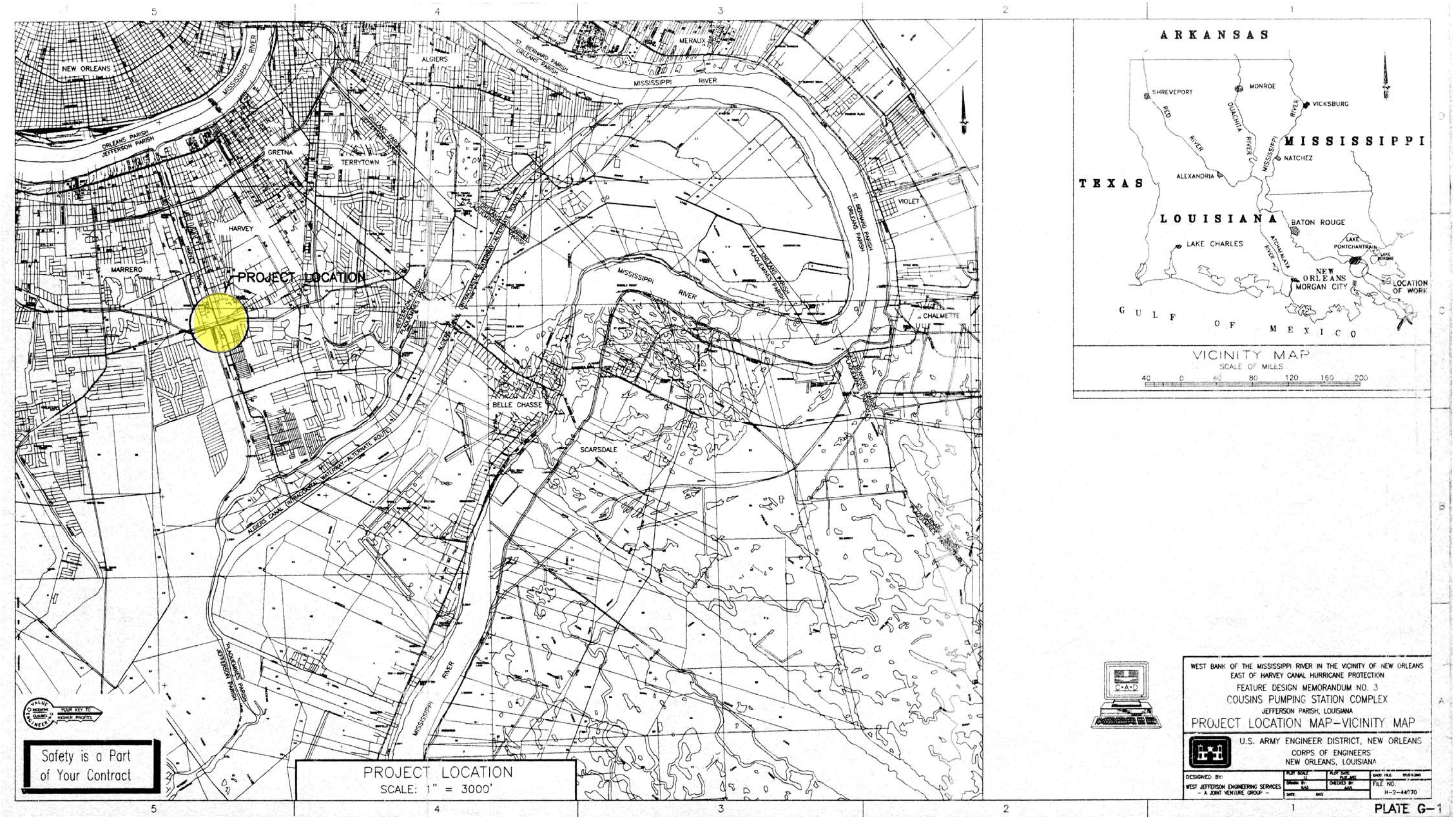
- Case I – Static water pressure with water to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force
- Case II – Static water pressure with water to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force
- Case III – Still-water pressure with water 2 ft above swl, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)
- Case IV – Still-water pressure with water 2 ft above swl, dynamic wave force, pervious sheet-pile cutoff (75 percent forces used)
- Case V – Static water pressure to swl, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)
- Case VI – Static water pressure to swl, dynamic wave force, pervious sheet-pile cutoff (75 percent forces used)
- Case VII – No water, no wind.

- Case VIII – No water, wind from protected side (75 percent forces used)
- Case IX – No water, wind from floodside (75 percent forces used).

Gates and Gate Monoliths. Gate monoliths will be constructed for street crossings in lieu of I-walls. The gate structures were designed for the following load conditions:

- Case I – Gate closed, static water pressure to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force
- Case II – Gate closed, static water pressure to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force
- Case III – Gate closed, static water pressure with water level 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
- Case IV – Gate closed, static water pressure with water level 2 ft above swl, no wind, pervious sheet-pile cutoff , no dynamic wave force (75 percent forces used)
- Case V – Gate closed, static water pressure to swl, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)
- Case VI – Gate closed, static water pressure to swl, dynamic wave force, pervious sheet-pile cutoff (75 percent forces used)
- Case VII – Gate open, no wind, truck or train on protected edge of base slab
- Case VIII – Gate open, no wind, truck or train on floodside edge of base slab
- Case IX – Gate open, wind from protected side, truck or train on floodside edge of base slab (75 percent forces used)
- Case X – Gate open, wind from floodside, truck or train on protected edge of base slab (75 percent forces used).

Cousins Pumping Station Complex - Reference 40. Image of area shown on following page.


NEW ORLEANS
MISSISSIPPI RIVER
ORLEANS PARISH
JEFFERSON PARISH
GRETNA
TERRYTOWN
ALGIERS
HARVEY
MARRERO
PROJECT LOCATION
ST. BERNARD PARISH
ORLEANS PARISH
MERAUX
VIOLET
CHALMETTE
BELLE CHASSE
SCARSDALE
ALGIERS CANAL (INTRACOASTAL WATERWAY-ALTERNATE ROUTE)
PLAQUEMINES PARISH
JEFFERSON PARISH
Safety is a Part of Your Contract
VALUE ENGINEERS
YOUR KEY TO HIGHER PROFITS
PROJECT LOCATION
SCALE: 1" = 3000'
ARKANSAS
SHREVEPORT
MONROE
VICKSBURG
RED RIVER
OUACHITA RIVER
MISSISSIPPI RIVER
MISSISSIPPI
NATCHEZ
TEXAS
ALEXANDRIA
LOUISIANA
BATON ROUGE
LAKE CHARLES
ATCHAFALAYA RIVER
LAKE PONTCHARTRAIN
LAKE BORGNE
NEW ORLEANS
MORGAN CITY
LOCATION OF WORK
GULF OF MEXICO
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
C•A•D
WEST BANK OF THE MISSISSIPPI RIVER IN THE VICINITY OF NEW ORLEANS
EAST OF HARVEY CANAL HURRICANE PROTECTION
FEATURE DESIGN MEMORANDUM NO. 3
COUSINS PUMPING STATION COMPLEX
JEFFERSON PARISH, LOUISIANA
PROJECT LOCATION MAP-VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY:
WEST JEFFERSON ENGINEERING SERVICES
- A JOINT VENTURE GROUP -
FILE NO.
H-2-44070
PLATE G-1

General. The Cousins Pumping Station will hold two 1,050-cfs horizontal pumps. The station consists of a concrete suction tube, pump, and discharge tube that will ultimately transfer water from Cousins Canal to Harvey Canal. The structure will be supported by a timber pile foundation. Other features of the project include 1,123 ft of I-type floodwall protection along the banks of the pumping station discharge channel and 184 ft of cantilever sheet-pile protection along the station discharge channel. The Destrehan Avenue Bridge will be lengthened by adding a 60-ft span to accommodate the widening of the Cousins Pumping Station discharge channel. A bottom roller floodgate will be provided at each end of the bridge. The following is a summary of the pertinent structural design criteria for these structures.

Water Elevations	**Elevations (ft NGVD)**
Still-water level (Harvey Canal)	7.5
Still-water Level + 2 ft freeboard (Harvey Canal)	9.5
Low water level (Harvey Canal)	0.0
Cousins Pumping Station	-8.5
Levee and Floodwall Net Grades	
I-walls, cantilever sheet-pile wall, bottom roller floodgates	11.5
Pumping station frontal protection T-wall	11.5
Culvert	11.5
Discharge channel closure wall	11.5

Culvert. The proposed culvert consists of a pile-supported, float-in type gravity structure. The structure was designed for computed hull-stress foundation loads at water elevations -1.0, +9.50, and +11.50 ft NGVD. The structure was also checked for floatation with a required FS of 1.30.

Destrehan Avenue Bridge. The lengthening of the Destrehan Avenue Bridge was designed in accordance with AASHTO requirements for HS-20 loading. Group 1 and 3 load cases were considered in the bridge design.

Bottom Roller Gates and Gate Monoliths. The pile designs for the bottom roller gate monoliths, based on pile load tests, are designed with a FS = 2.0. The following load cases were used for preliminary design:

- Case I – Gate closed, static water pressure to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)

- Case II – Gate closed, static water pressure to swl, no wind, pervious sheet-pile cutoff , no dynamic wave force (100 percent forces used)

- Case III – Gate closed, static water pressure to swl +2 ft, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)

- Case IV – Gate closed, static water pressure to swl +2 ft, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)

- Case V – Gate closed, wind from protected side (75 percent forces used)

- Case VI – Gate closed, winds from floodside (75 percent forces used)

- Case VII – gate open, no wind, and truck on protected side edge of base slab (100 percent forces used)

- Case VIII – Gate open, no wind, and truck on floodside edge of base slab (100 percent forces used)

- Case IX – Gate open, wind from protected side, and truck on floodside edge of base slab (75 percent forces used)

- Case X – Gate open, wind from floodside, and truck on protected side edge of base slab (75 percent forces used).

I-Type Floodwall. In the design of the I-walls, the following loading cases were considered:

- Case I – Water swl, Q-Case, FS = 1.5

- Case II – Water to swl + 2 ft, Q-Case, FS = 1.0

- Case III – Water to swl, S-Case, FS =1.2

- Case IV – Water to LPL with lateral earth pressure.

Pumping Station Frontal Protection. The pumping station frontal protection will be provided by a T-wall, supported on prestressed concrete piles. A continuous steel sheet-pile wall will be provided beneath the base for seepage cutoff purposes. The T-walls were designed for the following load conditions:

- Case I – Static water pressure to swl, no wind, impervious sheet-pile cutoff (100 percent of forces used).

- Case II – Static water pressure to swl, no wind, pervious sheet-pile cutoff (100 percent of forces used)

- Case III – Static water pressure to swl +2, no wind impervious sheet-pile cutoff (75 percent of forces used)

- Case IV – Static water pressure to swl +2 feet, no wind, pervious sheet-pile cutoff (75 percent of forces used)
- Case V – Water at low water level, no wind (100 percent of forces used)
- Case VI – Water at low water level, wind from protected side (75 percent of forces used).

Closure Wall. The closure wall will consist of two rows of 84-in.-diameter piles filled with sand. The floodside of the wall will be lined with steel sheet pile. A 18-ft by 4-ft cast-in-place concrete cap will connect the cylinder piles to the sheet piles. In the design of the closure wall, the following loading cases were considered:

- Case I – Water to swl, Q-Case, FS = 1.5
- Case II – Water to swl +2 ft, Q-Case, FS = 1.0.

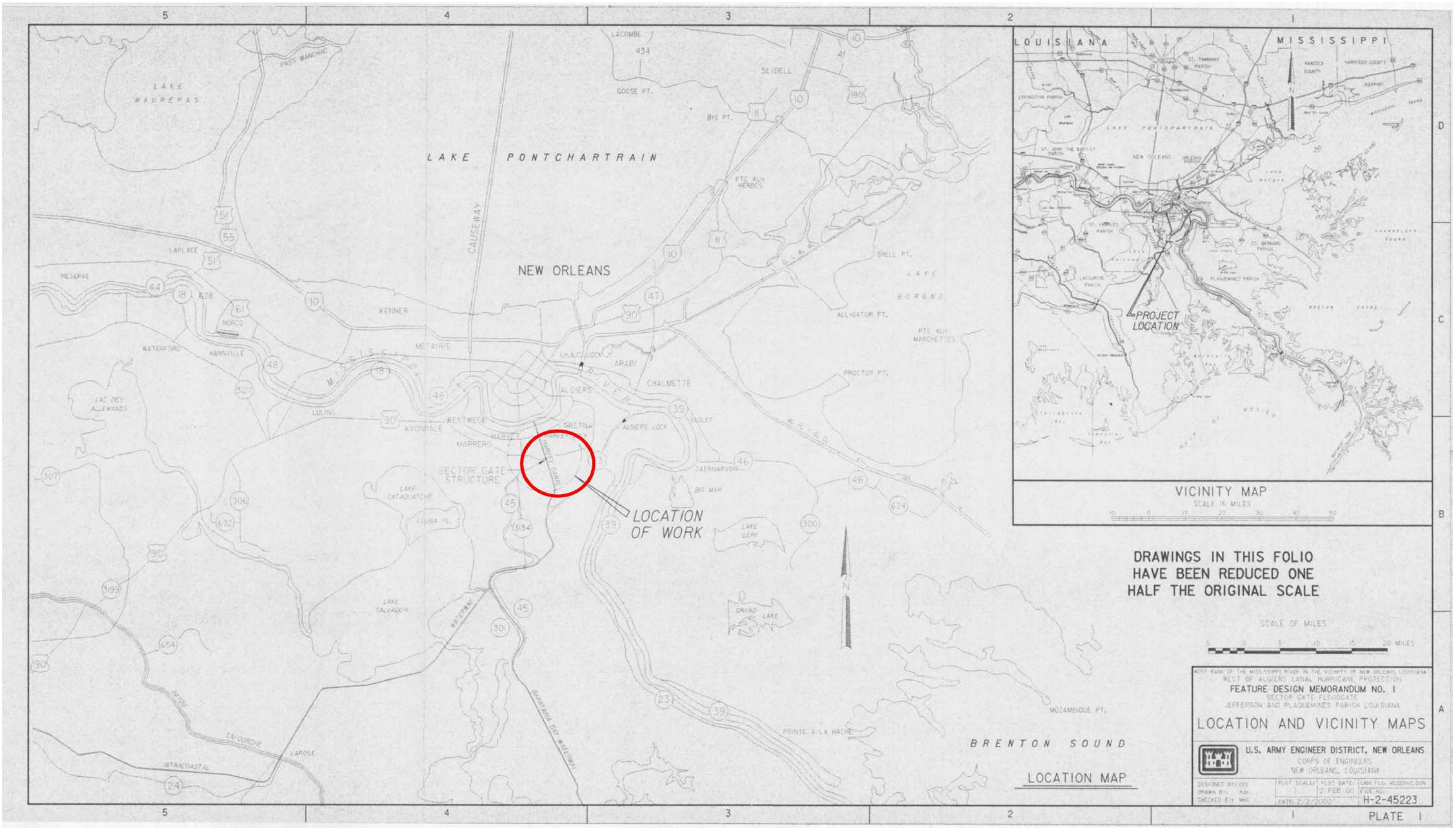
West of Algiers Canal Hurricane Protection - Sector Gate Complex – Reference 38. Map of area shown on following page.

General. This section presents the structural design criteria used to construct the sector gate complex portion of the East Harvey Canal hurricane protection features of the West Bank and Vicinity, New Orleans, LA, HPP. The sector gate complex is located in the Harvey Canal 250 ft downstream of the Lapalco Bridge. It consists of sector gate structure with a 125 ft opening and a sill elevation of -16.0 ft NGVD. The east side of the structure will be tied in by a floodwall to a floodwall running along the east side of Harvey Canal. On the west side the structure will be tied by a T-wall to a concrete flume located under Lapalco Bridge. An I-wall will be constructed along the west side of Harvey Canal and will tie into the west side of the concrete flume under Lapalco Bridge. The sector gate structure and tie-in floodwalls will be built to elevation 11.5 ft NGVD.

Basic Data. Basic data relevant to the elevations of the water surface, structure elevations and dimensions are as follows:

Design Water Elevations (ft NGVD)

Load Case	Gulf Side	Protected Side
Construction (graving site)	-	-
Transport loading	1.3	1.3
Setting condition (no backfill)	1.3	1.3
Normal operation	1.3	1.3
Max. direct gate operation	3.0	-1.0
Max. reverse gate operation	-1.0	4.0
Max. direct head - no hurricane	5.0	0.0
Max. reverse head - hurricane	-1.0	4.0
Direct head – hurricane (includes 2’ for subsidence)	9.5	-1.0
Direct head – hurricane (includes 2’ for subsidence)	9.5	-1.0
Direct head - hurricane – plus freeboard	11.5	-1.0
Maintenance dewatering	5.0	4.0


LOUISIANA
MISSISSIPPI
PROJECT LOCATION
VICINITY MAP
SCALE IN MILES
DRAWINGS IN THIS FOLIO HAVE BEEN REDUCED ONE HALF THE ORIGINAL SCALE
SCALE OF MILES
LAKE PONTCHARTRAIN
NEW ORLEANS
SECTOR GATE STRUCTURE
LOCATION OF WORK
BRENTON SOUND
LOCATION MAP
WEST BANK OF THE MISSISSIPPI RIVER IN THE VICINITY OF NEW ORLEANS, LOUISIANA
WEST OF ALGIERS CANAL HURRICANE PROTECTION
FEATURE DESIGN MEMORANDUM NO. 1
SECTOR GATE FLOODGATE
JEFFERSON AND PLAQUEMINES PARISH LOUISIANA
LOCATION AND VICINITY MAPS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
H-2-45223
PLATE 1

Structure Elevations (ft NGVD)

Top of floodgate	11.5
Top of fender and guidewalls	10.5 (9 ft above normal stage)
Sill	-16.0

Lateral Pressures (At-Rest K_O)

Sand $K_O = 0.50$
Semi-compacted cohesive soil $K_O = 0.80$
Stone and bedding material $K_O = 0.50$

Structure and Foundation Loadings

a. Loadings. The loads are described in Section 3, of ETL 1110-2-355 and modified as follows:

b. Dead Loads. For draft and buoyancy analysis added 3 percent to the concrete unit weight to account for swelling and construction tolerances.

Uplift. In lieu of accurate flow nets to determine seepage rates, a limit approach was used for this structure. Relief drains were not considered. Cutoff sheet piling walls are on both sides. The structure was designed for the three uplift conditions:

- Uplift Condition A assumes uniformly varying pressure between the gulf side and protected side sheet-piling cutoffs
- Uplift Condition B assumes the gulf side sheet-pile cutoff is impervious; the uplift pressure equals the protected side pressure head
- Uplift Condition C assumes the protected side sheet-pile cutoff is impervious; the uplift pressure equals the gulf side pressure head.

Thermal. The Nonlinear, Incremental Structural Analysis (NISA) was used to determine stress concentrations created during construction.

Wave Loads. The wave load refers to loads induced from a design wave when the module is buoyant. Two wave sizes are typically considered: a significant wave and a storm wave. The significant wave is anticipated within the 1-year construction period. The storm wave is a 50 year event and not considered. The structure shell is designed for inland waterway conditions. The total wave height observed along the GIWW is 3 ft (trough to crest).

Soil Drag. In lieu of more accurate analysis, drag shall be calculated as:

(Psoil-at-rest) x 0.5 x (Tangent of Internal Angle of Friction)

Impact. Boat impact on the gate is a 125 kip point load. This impact is applied to the gates and impact zone of the concrete walls.

Load Case Description

- **Normal Operation**. The gates are open; water stage is at elevation 1.3 ft NGVD. This is a usual load case with the ultimate strength design (USD) hydraulic load factor equal to 1.3.
- **Maximum Differential Head With Gate Operational**. The gates are designed to operate with a 4-ft head. The maximum stage is at elevation 3.0 ft NGVD and the minimum stage is elevation -1.0 ft NGVD. This is a usual load case with the USD hydraulic load factor equal to 1.3.
- **Maximum Reverse Head**. The gates are designed to operate with a 5-ft reverse head. The protected side stage is at elevation 4.0 ft NGVD and the minimum gulf side stage is elevation -1.0 ft NGVD. This is a usual load case with the USD hydraulic load factor equal to 1.3. This is also the maximum reverse operating head.
- **Maximum Direct Head - No Hurricane**. Gates are designed to be closed with gulf side at elevation5.0 ft NGVD and the protected side at elevation 0.0 ft NGVD. This is a usual load case with the USD hydraulic load factor equal to 1.3.
- **Maximum Direct Head - Hurricane Condition**. This condition is for the design hurricane: the gates are closed. The gulf side water stage is at elevation 9.3 ft NGVD and the protected side is at elevation 1.0 ft NGVD. The elevation of 9.3 includes an allowance for future ground subsidence and sea level rise. This is a usual load case with the USD hydraulic load factor equal to 1.3.
- **Maximum Direct Head Plus Freeboard - Hurricane Condition**. This condition is also for the design hurricane; the gates are closed. The gulf side water stage is at elevation 11.3 ft NGVD and the protected side is at elevation -1.0. Two feet of freeboard are included. This is an unusual load case with the USD hydraulic load factor reduced to 1.0.
- **Maintenance Dewatering**. This is for the maintenance dewatering condition with the gulf side needle dam experiencing a water stage at elevation 5.0 ft NGVD and the protected side water stage at elevation 4.0 ft NGVD. This is a short term loading; the USD hydraulic load factor is reduced to 1.0.

Sources of Construction Materials.

Sheet Pile. Generally, the sheet-pile sections specified during advertisement were used for construction. However, sheet-pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below is a table of sheet-pile sections listed by DM.

Westwego to Harvey	
Company Canal Floodwall	PZ-22*, Casteel CZ-128
Old Westwego to New Westwego PS Floodwall	IN-RU3, Casteel CZ-114
Cousins PS Complex	
Discharge Channel Floodwalls	Arbed AZ-46, AZ-13, PZ-27
* As advertised – not confirmed as-built.	

Levee Materials (Westwego to Harvey Canal). The majority of proposed levees will be built of the materials obtained from adjacent borrow pits except for the Harvey Canal portion, which will be obtained from Highway 45 levee borrow area.

Highway 45 levee from BL Station 425+46 to 575+89 will be constructed in one lift with semi-compacted clay fill. All other levees will be constructed in three lifts with an approximate 3-year interval between successive lifts. Lift 1 consists of constructing the levee and berms to full net grade and section, with construction of the berms preceding levee construction. The levees will be restored to net grade in Lift 2. In Lift 3, the levee crown will be built 1 ft above net grade. The berms will not be rebuilt after initial placement as part of Lift 1.

As-built Conditions

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation).

DACW29-95-C-0103. Westwego to Harvey Canal, LA, Westbank Hurricane Protection Levee, Estelle Pump Station to LP&L Powerlines, 1st Lift, Jefferson Parish, LA.

Reviewed modification log report and modification documents; no applicable modifications or changes were found.

DACW29-96-C-0032. Westwego to Harvey Canal Hurricane Protection Plan, New Westwego Pumping Station to Orleans Village, Jefferson Parish, LA.

Modification No. P00011 allowed CZ-101 sheet piles conforming to ASTMA328 with a minimum material thickness of 0.335 in. and maximum overall width of 27 in. to be substituted for SZ-20 or PZ-22 sheet piles.

Modification No. A00001 revised requirement for Atterberg limit tests from one for each density test or every 2,000 cu yd of semi-compacted fill, to one for each ten density tests, or every 20,000 cu yd.

Modification No. A00009 extended the sandfill levee base by 218 ft to facilitate the transition between the levee and sheetpile wall.

DACW29-99-C-0014. Westwego to Harvey, LA, Hurricane Protection Levee, V-Line Levee, East of Vertex, Second Lift, Louisiana Highway 3134 to Estelle Pumping Station, Jefferson Parish, LA.

Modification No. A00001 provided for levee repairs of failed sections of completed work between Station 220+00 B/L and 232.00 B/L and between Station 200+00 B/L and 190+00 B/L, by partially degrading the originally designed levee section and using this material to construct a stability berm on the protected side. This, in essence, reduced the levee cross section and lowered the crown height from elevation 10.0 ft NGVD to elevation 9.0, ft NGVD and reduced the crown width from 10 to 7 ft.

DACW29-00-C-0047. Westwego to Harvey Canal, LA, Hurricane Protection Levee, Westwego Seaplane Airport Canal Closure, First Lift, Jefferson Parish, LA.

Modification No. A00001 raised the sand base to elevation 2.0 ft NGVD to facilitate placing embankment in the dry.

Inspection During Original Construction, QA/QC, State What Records Are Available.

DACW29-96-C-0032. Westwego to Harvey Canal Hurricane Protection Plan, New Westwego Pumping Station to Orleans Village, Jefferson Parish, LA.

Attached are percent complete lists and grain size analysis records.

DACW29-98-C-0043. Southeast Louisiana Urban Flood Control Project, Keyhole Canal, 4th Street to LaPalco Boulevard, Jefferson Parish, LA.

Attached from time to time are the preparatory inspection reports and the status summary of the job.

DACW29-99-C-0014. Westwego to Harvey, LA, Hurricane Protection Levee, V-Line Levee, East of Vertex, Second Lift, Louisiana Highway 3134 to Estelle Pumping Station, Jefferson Parish, LA.

No QA/QC reports were found.

DACW29-00-C-0047. Westwego to Harvey Canal, LA, Hurricane Protection Levee, Westwego Seaplane Airport Canal Closure, First Lift, Jefferson Parish, LA.

Attached are preparatory phase reports.

DACW29-01-C-0029. Westbank and Vicinity, New Orleans, LA, Hurricane Protection Project, Algiers Canal Levee Enlargement and Floodwall, East Side Hero Levee to Belle Chase Highway, Plaquemines Parish, LA.

No QA/QC reports were found.

Inspection and Maintenance of Original Construction

Annual Compliance Inspection (i.e., trees, etc.).

As stated in the Lake Cataouatche Section, this area received a rating of “Acceptable” for the levee system under the Annual Compliance Program for the West Jefferson Levee District.

Periodic inspections.

There are no structures under the Periodic Inspection Program in the Westwego to Harvey area of the West Bank and Vicinity Hurricane Protection Project.

Other Features – Jefferson West Bank, Westwego to Harvey

Brief Description.

The primary components of the hurricane protection system for the Jefferson West Bank, Westwego to Harvey subarea are described above, namely the levees and floodwalls designed and constructed by the Corps. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

Pre-Katrina Conditions.

According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to 29 August 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

Interior Drainage System.

Overview. The Jefferson West Bank, Westwego to Harvey subarea contains about 22 square miles and generally slopes north to south from the Mississippi River. It is mostly developed except for some low areas in the southern tip. Many features are typical of large urban cities in the United States, and some features are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations that pump the water into the Harvey Canal, Bayou Segnette, or canals and bayous on the south side of the polder. No stormwater is pumped into the Mississippi River.

The entity responsible for local drainage in the Jefferson West Bank basin is Jefferson Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson West Bank. The land generally falls south from the Mississippi River. A land feature visible on the topographic layout that affects the local drainage is a ridge that runs north-south between the Harvey Canal and Bayou Segnette. The locations of the interior ditches, canals, and pump stations were influenced by this ridge.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 125 subbasins. Pump station information is presented on following page of this volume.

The canals are open and most are grass-lined. The canals and ditches not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10 percent storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 in./hr. It will increase to 0.5 in./hr after the SELA projects are complete (see status below). Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, ditches, and canals. There are criteria for new developments to use stormwater detention to offset downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. In some cases, Jefferson Parish and the Corps are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. Jefferson Parish is the local, cost-sharing sponsor for the Jefferson Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the Parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

The SELA projects in the Jefferson West Bank, Westwego to Harvey subpolder are shown in Figure 36. The work consists of adding capacity to 15 canals, increasing pumping capacity at the Cousins Pump Station, and improving two existing pump stations. Prior to Hurricane Katrina, the improvements to the two pump stations were under design, the Cousins Pump Station was under construction, 10 canals were complete, 2 canals were partially complete, and 3 canals were under design.

Pumping Stations - Jefferson Parish Westwego to Harvey.

Jefferson Parish is located west of the city of New Orleans and borders the west side of Orleans Parish. Figure 37 is a map of Jefferson Parish with the pump stations that were studied identified by red bullets. Jefferson Parish is separated by the Mississippi River into East and West Banks. The East Bank pump stations are connected by a grid of canals. The canals running east and west serve to equalize flow between the major outfall canals, allowing rainwater to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into subbasins that, for smaller rainfall events, operate independently. However, over-bank flow does occur between adjacent subbasins for a 10-year event. This report examined six pump stations on the East Bank with a total of 36 pumps and 17 pump stations on the West Bank with a total of 65 pumps.

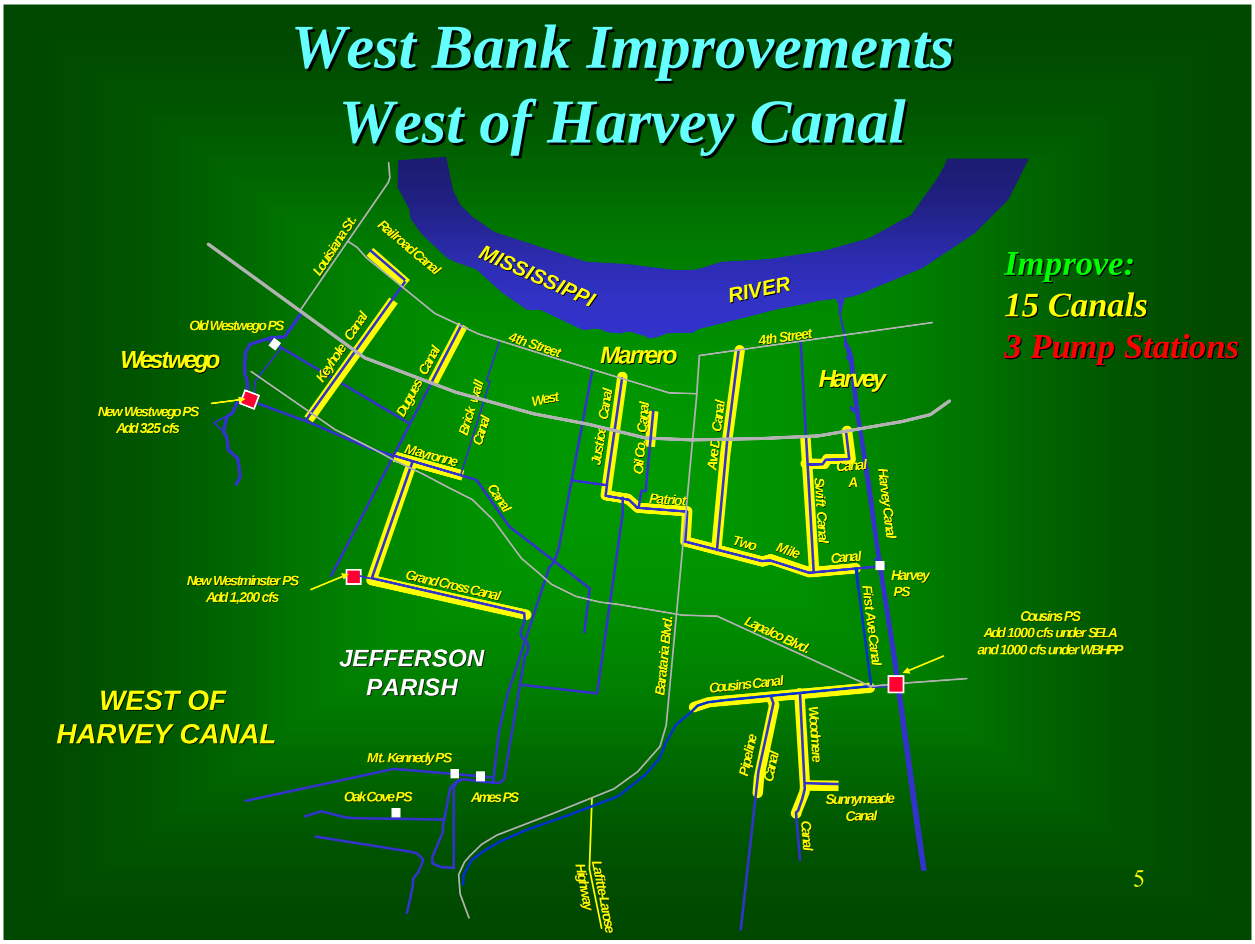


Figure 36. SELA Urban Flood Control Projects in Jefferson West Bank, Westwego to Harvey.

Figure 37 is a map showing the Jefferson Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

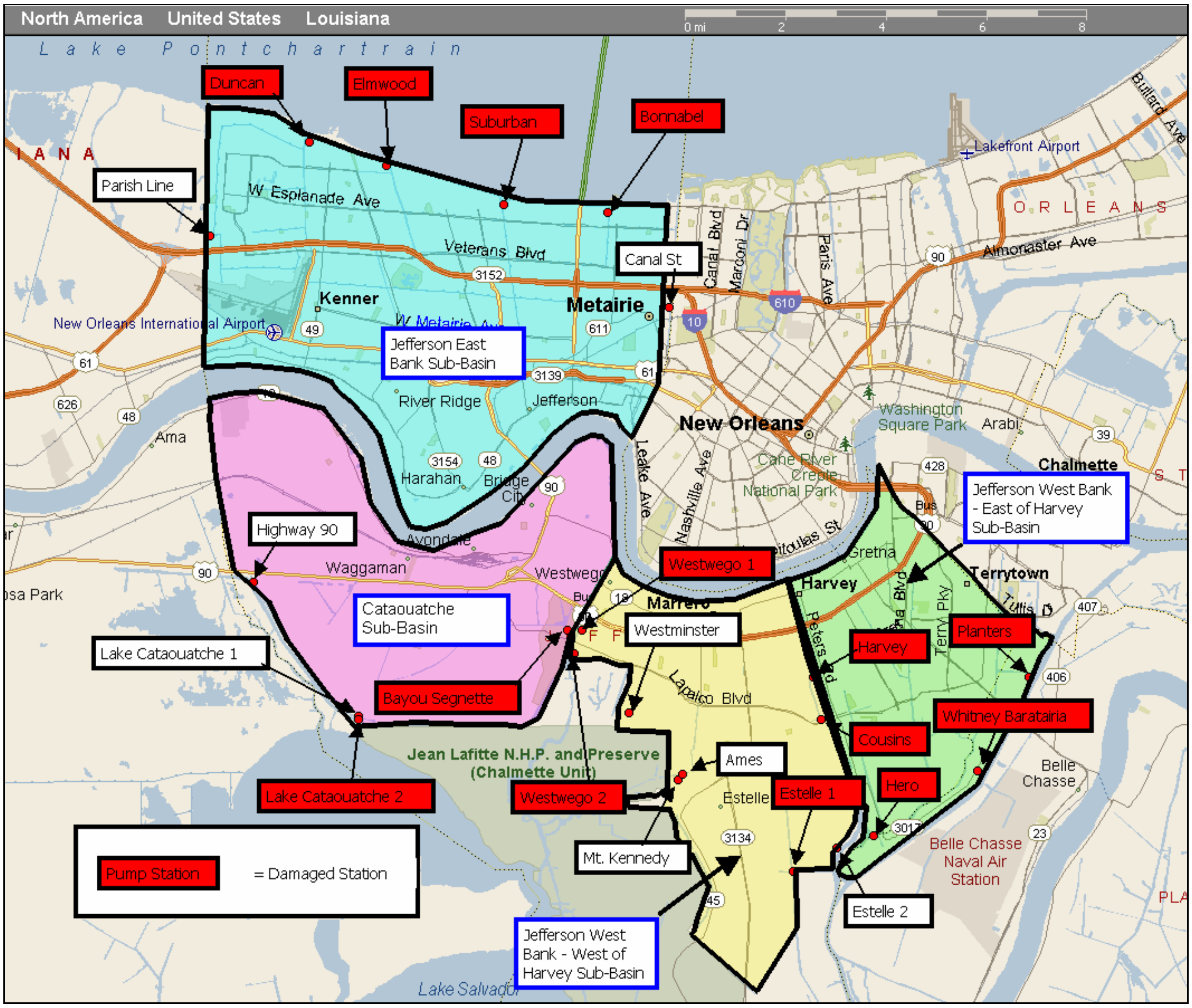


Figure 37. Jefferson Parish pump station locations.

Table 42 contains information about each individual pump at each of the examined pump stations in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump, and missing information was left blank or highlighted.

Table 42
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank – East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

Drainage Basin

West Bank – West of Harvey

The West Bank – West of Harvey drainage basin has eight significant pump stations, which are briefly described below. Volume VI provides more details. The basin is bordered by the Mississippi River on the north. The drainage system includes the Mississippi River, as well as wetlands and the First Avenue, Two Mile, Cousins, Harvey, Pipeline, Kenta/Seivers, Grand Cross, Inner Milladoun, Bayou Segnette, WPA, G, and H Canals.

Harvey
Intake location: ....First Avenue and Two Mile Canal
Discharge location: ....First Avenue and Two Mile Canal
Nominal capacity: ....960 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	320	n/a	Electric	n/a
2	320	n/a	Electric	n/a
3	320	n/a	Electric	n/a

Cousins No. 1
Intake location: .... Cousins Canal and First Avenue Canal
Discharge location: .... Harvey Canal
Nominal capacity: ....800 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	50	n/a	Electric	Vertical
2	250	n/a	Diesel	Vertical
3	250	n/a	Diesel	Vertical
4	250	n/a	Diesel	Vertical

Cousins No. 2

Intake location: .... Cousins Canal and First Avenue Canal
Discharge location: .... Harvey Canal
Nominal capacity: ....2,200 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1,100	n/a	Diesel	n/a
2	1,100	n/a	Diesel	n/a

Estelle

Intake location: ....Pipeline Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....682 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	150	n/a	Electric	n/a
2	150	n/a	Electric	n/a
3	150	n/a	Electric	n/a
4	232	n/a	Electric	n/a

New Estelle

Intake location: .... Pipeline and Canal G
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....1,140 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	570	n/a	Diesel	n/a
2	570	n/a	Diesel	n/a

Mount Kennedy

Intake location: ....Kenta/Seivers Canal
Discharge location: .... Bayou Segnette
Nominal capacity: ....500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	167	n/a	Electric	n/a
	167	n/a	Electric	n/a
3	167	n/a	Electric	n/a

Westminster 1 and 2

Intake location: .... Grand Cross
Discharge location: .... Wetlands
Nominal capacity: ....1,248 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	312	n/a	Electric	n/a
2	312	n/a	Electric	n/a
3	312	n/a	Electric	n/a
4	312	n/a	Electric	n/a

Ames

Intake location: .... Inner Milladoun
Discharge location: .... Bayou Segnette
Nominal capacity: ....1,930 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	390	1982	Electric	Vertical
2	390	1982	Electric	Vertical
3	1,150	n/a	Diesel	Horizontal

Westwego No. 1
Intake location: .... WPA Canal
Discharge location: .... Bayou Segnette
Nominal capacity: ....300 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	n/a	Diesel	Vertical

Westwego No. 2
Intake location: ....Avenue H Canal
Discharge location: .... Bayou Segnette
Nominal capacity: ....935 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	312	n/a	Diesel	n/a
2	312	n/a	Diesel	n/a
3	311	n/a	n/a	n/a

Levees and Floodwalls.

MRL **-** MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are no floodwalls that are part of the MRL in this reach.

Non-Corps - Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3.7. East of Harvey Canal Area

Introduction

This area consists of approximately 25 miles of levee, 5 miles of floodwalls, and a sector floodgate in the Harvey Canal as shown in Figure 38 below. This area consists of work both East and West of Algiers Canal.

Pre-Katrina

Construction in this area started in 2000. Before Hurricane Katrina, six construction contracts were completed in this area. Another three were under construction and are still ongoing, scheduled for completion in 2007. Remaining work in this area consists of first enlargement levee or floodwall contracts, pump station modifications and fronting protection contracts, and future second enlargement levee contracts.

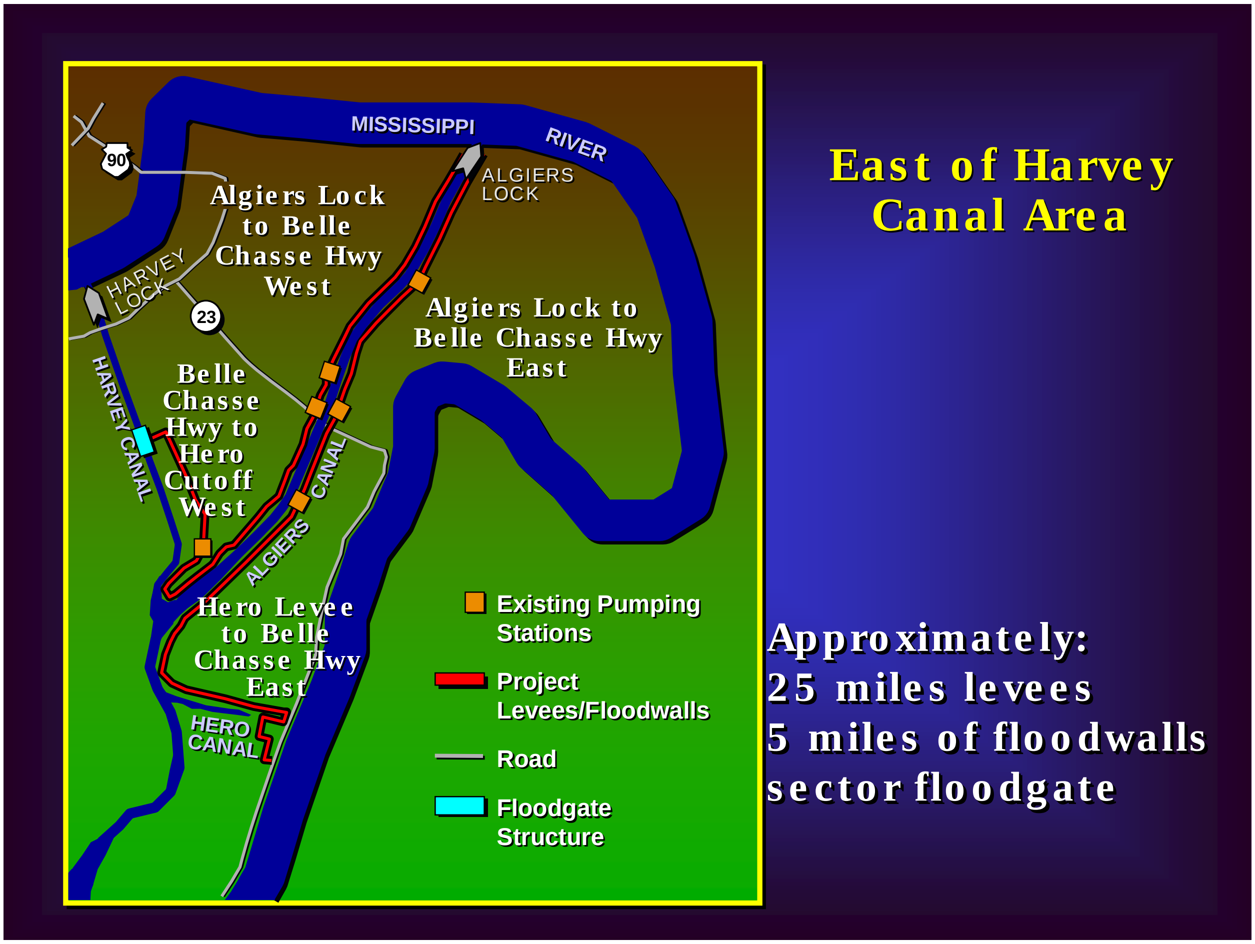


Figure 38. East of Harvey Canal area project features.

Design Criteria and Assumptions - Functional Design Criteria

Hydrology and Hydraulics.

For the East of Harvey Canal area, the design hurricane characteristics are shown in Table 43; the design tracks are shown on Figure 39. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 43
Design Hurricane Characteristics

Location	Track	CPI in.	Radius of Maximum Winds, nautical miles	Forward Speed knots	Maximum Wind Speed,[1] mph	Direction of Approach
East of Harvey	C	27.4	30	11	100	South

[1] Wind speeds represent a 5-minute average 30 ft above ground level.

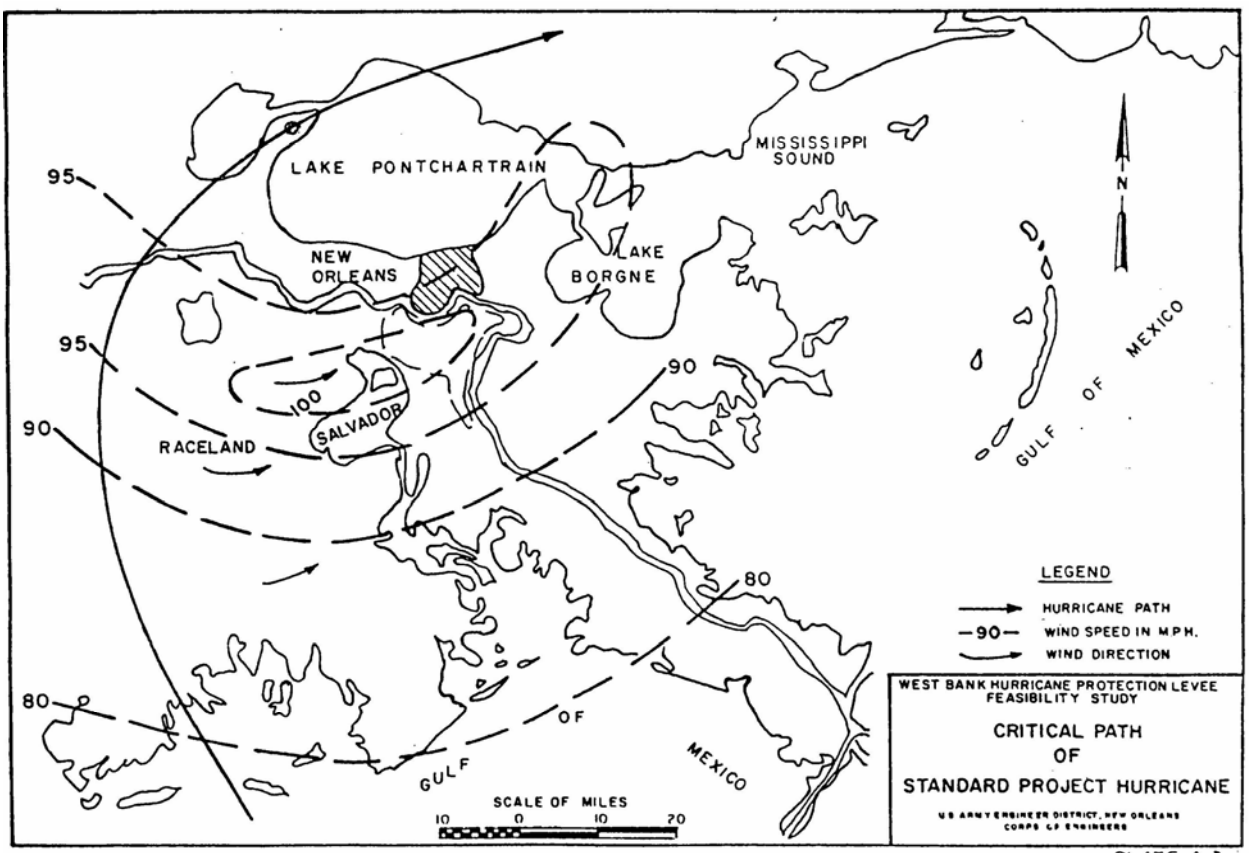


Figure 39. Critical path of standard project hurricane.

Surge. Wind tide elevations for East of Harvey Canal area were computed using the same methodology as used for Westwego to Harvey area.

A WIFM model was used to evaluate future land loss due to subsidence and an estimated sea level rise of 0.2 feet per 50 years. Model results indicated an increase in wind tide level of 1.0 ft by the year 2040.

Waves. For the East of Harvey Canal area, some levees and floodwalls would be sheltered from storm generated runup; small locally generated waves could occur. These small waves would be likely to occur along Oakville Levee, Harvey Canal, and Algiers Canal. Wave runup for all levees and floodwalls was calculated using methodology described in 1984 *Shore Protection Manual*.

Summary. Table 44 contains maximum surge or wind tide level, wave, and design elevation information.

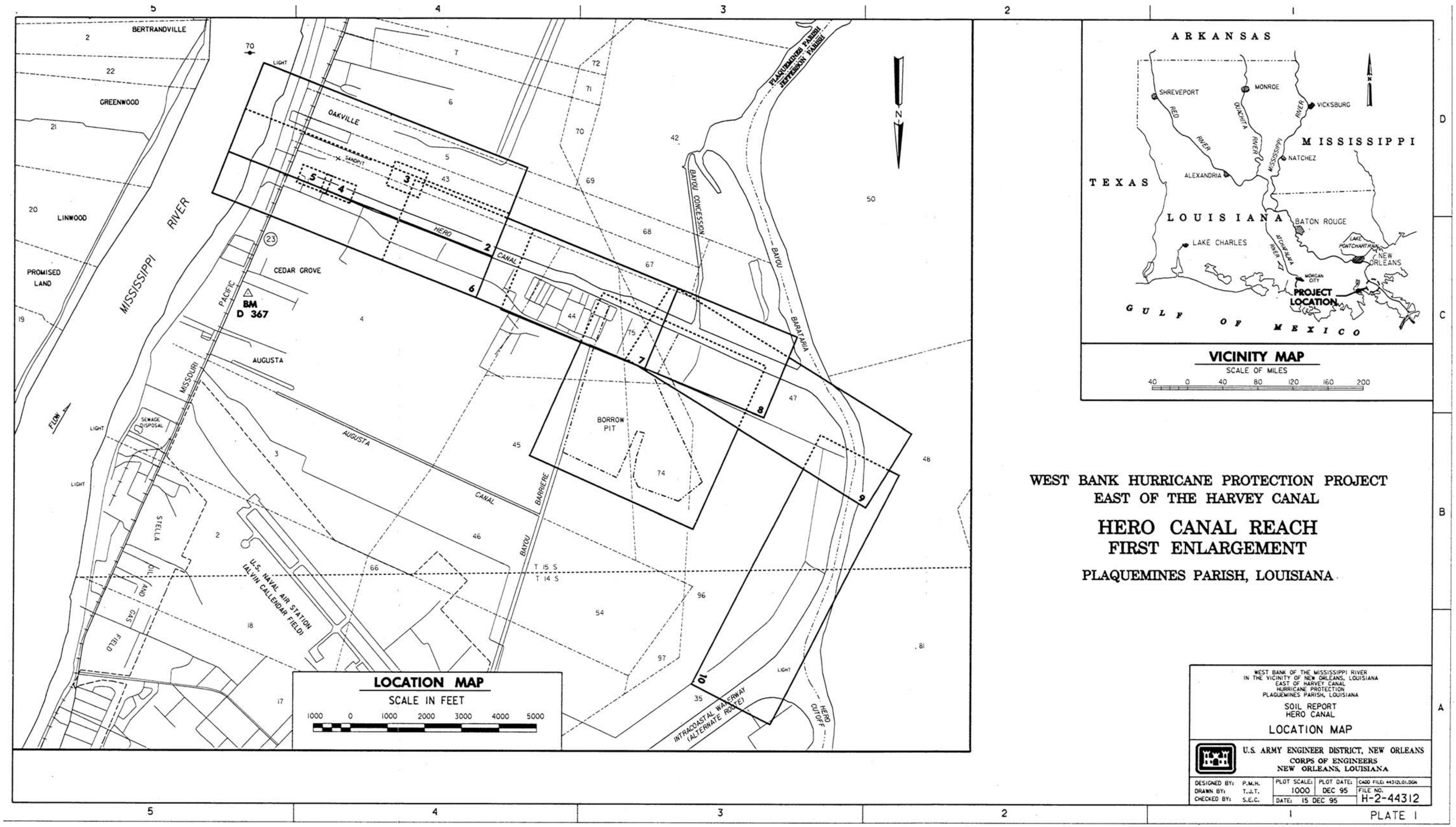
Table 44
Wave Runup and Design Elevations (Transition zones not tabulated – Governing Report is listed)

Location	Report	Average Depth of Fetch, ft	Significant Wave Height, Hs ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level ft	Runup Height ft	Freeboard ft	Design Elevation Protective Structure, ft
Oakville Levee	Feasibility Study	NA	1.0	2.7	7.0 NGVD	2.0	-	9.0 NGVD
Harvey Canal	Feasibility Study	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
Algiers Canal	Feasibility Study	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
Hero Canal Reach	Feasibility Study	NA	NA	NA	7.5 NGVD	3.0	-	10.5 NGVD

Interior Drainage. The design includes increasing the capacity of the Cousins Pumping Station. When the floodgate structure on the Harvey Canal is closed, the existing Harvey Pumping Station would be shut down, and interior drainage would be diverted to the Cousins Pumping Station. The capacity of this pumping station would be increased by 1,000 cfs, the outfall canal for the pump station enlarged, and the 1st Avenue Canal, which connects the Harvey and Cousins Pumping Stations, would be enlarged to handle the additional drainage. For the feasibility study, Manning's equation was used to size the 1st Avenue Canal. The continuity equation was used to design the outfall canal, with a velocity of 3.5 fps chosen based on erodibility of the channel bottom. The improvements to 1st Avenue Canal were modeled using UNET as part of the SELA project. The increase in capacity to Cousins Pumping Station was also evaluated as part of the SELA project. Improvements to 1st Avenue Canal and Cousins Pumping Station were constructed concurrent with the SELA drainage improvements.

Geotechnical.

This report addresses design assumptions and parameters for new levees, enlargement of existing levees and floodwalls. The project consists of three design reaches for approximately 12,000 ft of floodwall and 125,000 ft of levee. Additional information included in Reference 31. Image of area shown on following page.


WEST BANK HURRICANE PROTECTION PROJECT
EAST OF THE HARVEY CANAL
HERO CANAL REACH
FIRST ENLARGEMENT
PLAQUEMINES PARISH, LOUISIANA
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
GULF OF MEXICO
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
LAKE PONTCHARTRAIN
NEW ORLEANS
MORGAN CITY
PROJECT LOCATION
RED RIVER
OUACHITA RIVER
MISSISSIPPI RIVER
ATCHAFALAYA RIVER
LOCATION MAP
SCALE IN FEET
1000 0 1000 2000 3000 4000 5000
BERTRANDVILLE
GREENWOOD
LINWOOD
PROMISED LAND
MISSISSIPPI RIVER
FLOW
OAKVILLE
SANDPIT
CEDAR GROVE
AUGUSTA
BM D 367
PACIFIC
MISSOURI
SEWAGE DISPOSAL
STELLA
OIL AND GAS FIELD
U.S. NAVAL AIR STATION (ALVIN CALLENDAR FIELD)
HERO CANAL
AUGUSTA CANAL
BAYOU BARRIERE
BAYOU CONCESSION
BAYOU BARATARIA
BORROW PIT
PLAQUEMINES PARISH
JEFFERSON PARISH
INTRACOASTAL WATERWAY (ALTERNATE ROUTE)
HERO CUTOFF
LIGHT
T 15 S
T 14 S
N
WEST BANK OF THE MISSISSIPPI RIVER
IN THE VICINITY OF NEW ORLEANS, LOUISIANA
EAST OF HARVEY CANAL
HURRICANE PROTECTION
PLAQUEMINES PARISH, LOUISIANA
SOIL REPORT
HERO CANAL
LOCATION MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: P.M.H.
DRAWN BY: T.J.T.
CHECKED BY: S.E.C.
PLOT SCALE: 1000
PLOT DATE: DEC 95
DATE: 15 DEC 95
FILE NO. H-2-44312
PLATE I

Geology. The project site is located on the deltaic plain portion of the Mississippi River Alluvial Plain. Specifically, the area is located on the northern edge of the Barataria basin on the western side of the Mississippi River between miles 73 to 98 above Head of Passes. The Barataria Basin is an interdistributary basin dominated by features which include natural levee ridges, crevasse-splay deposits, marsh, lake, and swamps. The eastern and northern edge of the basin is defined by the natural levee ridge of the Mississippi River and the western edge of the basin is defined by the Bayou Lafourche natural levee ridge. The Gulf of Mexico constitutes the southern boundary. Elevations range from approximately +10 to +15 ft NGVD in the back swamp and lake areas to below 0 ft NGVD in areas under pump.

The area is protected from Mississippi River overflows by the main stem levee system. Flooding originating in the Gulf of Mexico can travel across the marsh and through Bayou Barataria to threaten the area from the south. To protect the area from this tidal and storm surge flooding, local interests have constructed a network of levees that provide a limited degree of protection.

Foundation Condition. The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to medium. There are occasional layers of silt (ML), silty sand (SM), and lean clays (CL). Layers of organic clays, which typically display high moisture content, exist in the area near the Intracoastal Waterway from the original ground surface down to approximately elevation -20 ft NGVD.

Field Exploration. Five general-type borings were taken along parts of the proposed alignment in October 1992, and two borrow borings were taken in the borrow pit in March 1994. Other undisturbed and general type borings used for design can be found in the following reports stored in the foundation branch of the New Orleans District.

a. Algiers Lock and Canal – Soils Investigation, June 1948.

b. Algiers Lock and Canal – Definite Project Report, June 1948.

Raising of the existing Algiers Canal levee will provide part of the flood protection for this project.

Four undisturbed borings and four general type borings were taken along parts of the proposed alignment in January 1995.

Underseepage. Not addressed.

Hydrostatic Uplift. Not addressed.

Pile Foundation. Not addressed.

Slope Stability. The project was divided into three design reaches based on boring data. Reach III was split into subreaches “a” and “b” based on differing surface conditions. The reaches are as follows:

Reach I .... Floodwall west of Algiers Canal

Reach II.. Algiers Canal, East and West Bank

Reach IIIa...North of Hero Canal

Reach IIIb ....South of Hero Canal

The still-water level (swl) used for reaches I and II was elevation 7.5 ft NGVD. Low water level was used as elevation 0.0 ft NGVD. The swl for Reach IIIa was elevation 8.5 ft NGVD and for Reach IIIb elevation 7.0 ft NGVD, both with a low water level of elevation 0.0.

Stability of Levees**.** Existing conditions along the proposed alignment were estimated and the slopes and berm distances for the proposed levee were designed for the Q construction case. A FS of 1.3 is required for the levee stability. For Reaches IIIa and IIIb, surveys were taken in January 1995.

Based on historical data from the Larose to Golden Meadow area, shrinkage and settlement of levee fill should be in the range of 20 to 30 percent over the 3 or 4 years between the first and second lift. The final lift will compensate for the expected lifetime settlement of the levee. The Algiers Canal Levee should experience minimal settlement since the centerline of the levee will remain unchanged.

I-Type Floodwall. I-wall stability and required sheet-pile penetration was estimated using a penetration to head ratio of 3:1 to estimate sheet-pile penetration. There is no significant wave load on the I-wall. For Detail Design of the floodwall, the following criteria will be followed:

Q-Case

FS = 1.5 with water to flow line or swl

FS = 1.25 with water to flow line plus approved freeboard for river levees or with swl and wave load for hurricane protection levees

FS = 1.0 with swl plus 2 ft freeboard for hurricane protection levees.

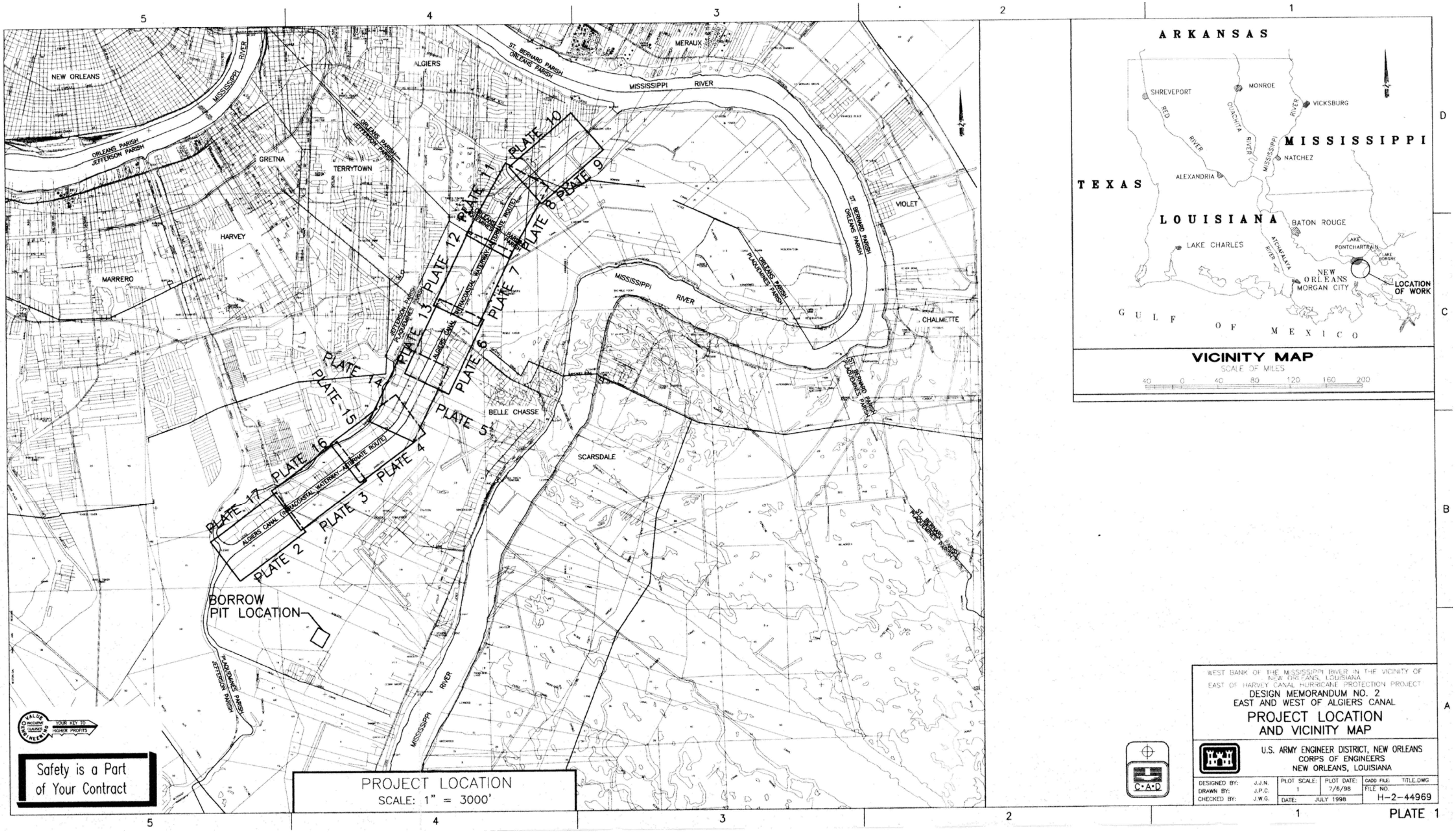
S-Case

FS = 1.2 with water to flow line or swl and wave load. If a hurricane protection floodwall has no significant wave load, determine the penetration using Q-Case criteria only

FS = 1.0 with water to flow line plus approved freeboard for river levees.

T-Type Floodwall. Not used.

Structural – (East and West of Algiers Canal - Reference 59). Image of area shown on following page.


NEW ORLEANS
ALGIERS
MERAUX
MISSISSIPPI RIVER
ORLEANS PARISH
JEFFERSON PARISH
ST. BERNARD PARISH
ORLEANS PARISH
GRETNA
TERRYTOWN
HARVEY
MARRERO
VIOLET
CHALMETTE
BELLE CHASSE
SCARSDALE
PLAQUEMINES PARISH
PLATE 2
PLATE 3
PLATE 4
PLATE 5
PLATE 6
PLATE 7
PLATE 8
PLATE 9
PLATE 10
PLATE 11
PLATE 12
PLATE 13
PLATE 14
PLATE 15
PLATE 16
PLATE 17
ALGIERS CANAL (INTRACOASTAL WATERWAY-ALTERNATE ROUTE)
BORROW PIT LOCATION
Safety is a Part of Your Contract
YOUR KEY TO HIGHER PROFITS
PROJECT LOCATION
SCALE: 1" = 3000'
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
LAKE PONTCHARTRAIN
LAKE BORGNE
NEW ORLEANS
MORGAN CITY
LOCATION OF WORK
RED RIVER
OUACHITA RIVER
MISSISSIPPI RIVER
ATCHAFALAYA RIVER
GULF OF MEXICO
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
WEST BANK OF THE MISSISSIPPI RIVER IN THE VICINITY OF NEW ORLEANS, LOUISIANA
EAST OF HARVEY CANAL HURRICANE PROTECTION PROJECT
DESIGN MEMORANDUM NO. 2
EAST AND WEST OF ALGIERS CANAL
PROJECT LOCATION AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: J.J.N.
DRAWN BY: J.P.C.
CHECKED BY: J.W.G.
PLOT SCALE: 1
PLOT DATE: 7/6/98
DATE: JULY 1998
CADD FILE: TITLE.DWG
FILE NO. H-2-44969
PLATE 1

General. The structural portions of project that have been completed include three segments of I-wall across the Belle Chasse Tunnel on both the east and west side and adjacent to the railroad on the west side. None of the gate structures or the pump station frontal protection structures has been constructed.

I-Walls. Analyses for the cantilevered I-walls were performed using the Corps CWALSHT program. The analyses were performed by applying a FS of 1.5 to the Q soil parameters when considering the design hurricane storm level at elevation 9.5 ft NGVD. A FS of 1.0 was applied to the Q soil parameters for a separate analysis evaluating a water level at elevation 11.5 ft NGVD.

Minimum Penetration. The minimum sheet-pile penetration for cantilever sheet-pile walls was determined by providing a minimum sheet-pile penetration below the ground surface to water head ratio of 3:1

Loading Cases. In the design of the I-walls, the following loading cases were considered:

- Case I – Water to swl, Q-Case, FS = 1.5
- Case II – Water to swl + 2 ft, Q-Case, FS = 1.0.

Sources of Construction Materials.

Sheet Pile. Generally, the sheet-pile sections specified during advertisement were used for construction. However, sheet-pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below is a table of sheet-pile sections for East of Harvey, listed by DM.

East of Harvey	**DM**
East and West of Algiers Canal	
Belle Chasse PS No. 1 Tie-In	**
Belle Chasse PS No. 2 Tie-In	PZ-27
Planters PS Tie-In	PZ-27
S&WB PS No. 11 Tie-In	**
S&WB PS No. 13 Tie-In	**
* As advertised – Not confirmed as-built. ** Information not available at the time of publication.	

Levee Material. Borrow material will be hauled from a nearby pit where the limits have been preliminarily established.

As-built Conditions

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation).

DACW29-01-C-0029. Westbank and Vicinity, New Orleans, LA, Hurricane Protection Project, Algiers Canal Levee Enlargement and Floodwall, East Side Hero Levee to Belle Chase Highway, Plaquemines Parish, LA.

Modification No. P00001 provided for the government to furnish 347 15-ft SPZ-22 sheet piles which were substituted for the contract required PZ-22 sheet piles.

Inspection During Original Construction, QA/QC, State What Records Are Available.

See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

Inspection and Maintenance of Original Construction

Annual Compliance Inspection (i.e., trees, etc.).

As stated in the Lake Cataouatche section, this area received a rating of "Acceptable" for the levee system under the West Jefferson Levee District.

Periodic Inspections.

There are no structures under the Periodic Inspection Program in the East of Harvey Canal area, of the West Bank and Vicinity Hurricane Protection Project.

Other Features – Jefferson and Orleans West Bank, East of Harvey

Brief Description.

The primary components of the hurricane protection system for the Jefferson and Orleans West Bank, East of Harvey subarea are described above, namely the levees and floodwalls designed and constructed by the Corps. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

Pre-Katrina Conditions.

According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to 29 August 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

Interior Drainage System.

Overview. The Jefferson and Orleans West Bank, East of Harvey subarea contains about 30 square miles and generally slopes north to south from the Mississippi River. It is mostly developed except for a few tracts near the Harvey Canal in Jefferson Parish and Denver Canal in Orleans Parish. Many features are typical of large urban cities in the United States, and some features are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open and enclosed canals collect the stormwater and carry it to stormwater pump stations that pump the water into the Intracoastal Waterway. No stormwater is pumped into the Mississippi River.

The entities responsible for local drainage are Jefferson Parish and Orleans Parish in their respective jurisdictions. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson and Orleans West Bank. The land generally slopes from the Mississippi River to the Intracoastal Waterway.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 118 subbasins. Pump station information is presented on page III-510 of this volume.

The canals are open and most are grass-lined. The interior canals and ditches not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10 percent storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 in./hr. It will increase to 0.5 in./hr after the SELA projects are complete (see status below). Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, ditches, and canals. There are criteria for new developments to use stormwater detention in order to offset downstream impacts.

Where local drainage is considered to need improvement, the parishes are working to improve the drainage. In some cases, Jefferson Parish and the Corps are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. Jefferson Parish is the local, cost-sharing sponsor for the Jefferson Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the Parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

The SELA projects in the Jefferson West Bank, East of Harvey subarea are shown in Figure 40. The work consists of adding capacity to five canals, increasing pumping, and adding a new pump station - Whitney/Barataria Pump Station. Prior to Hurricane Katrina, the pump station was partially completed, four canals were complete, and one canal was partially complete, but functional when Katrina made landfall.

Pumping Stations - Jefferson Parish West Bank and Orleans Parish West Bank.

Figure 41 is a map of Jefferson Parish with the pump stations that were studied identified by red bullets. Jefferson Parish is separated by the Mississippi River into East and West Banks. The West Bank is subdivided into subbasins that, for smaller rainfall events, operate independently. However, overbank flow does occur between adjacent subbasins for a 10-year event. This report examined 17 pump stations on the West Bank with a total of 65 pumps. Figure 42 is a map showing the Orleans Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

Table 45 contains information about each individual pump at each of the examined pump stations in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump, and missing information was left blank or highlighted.

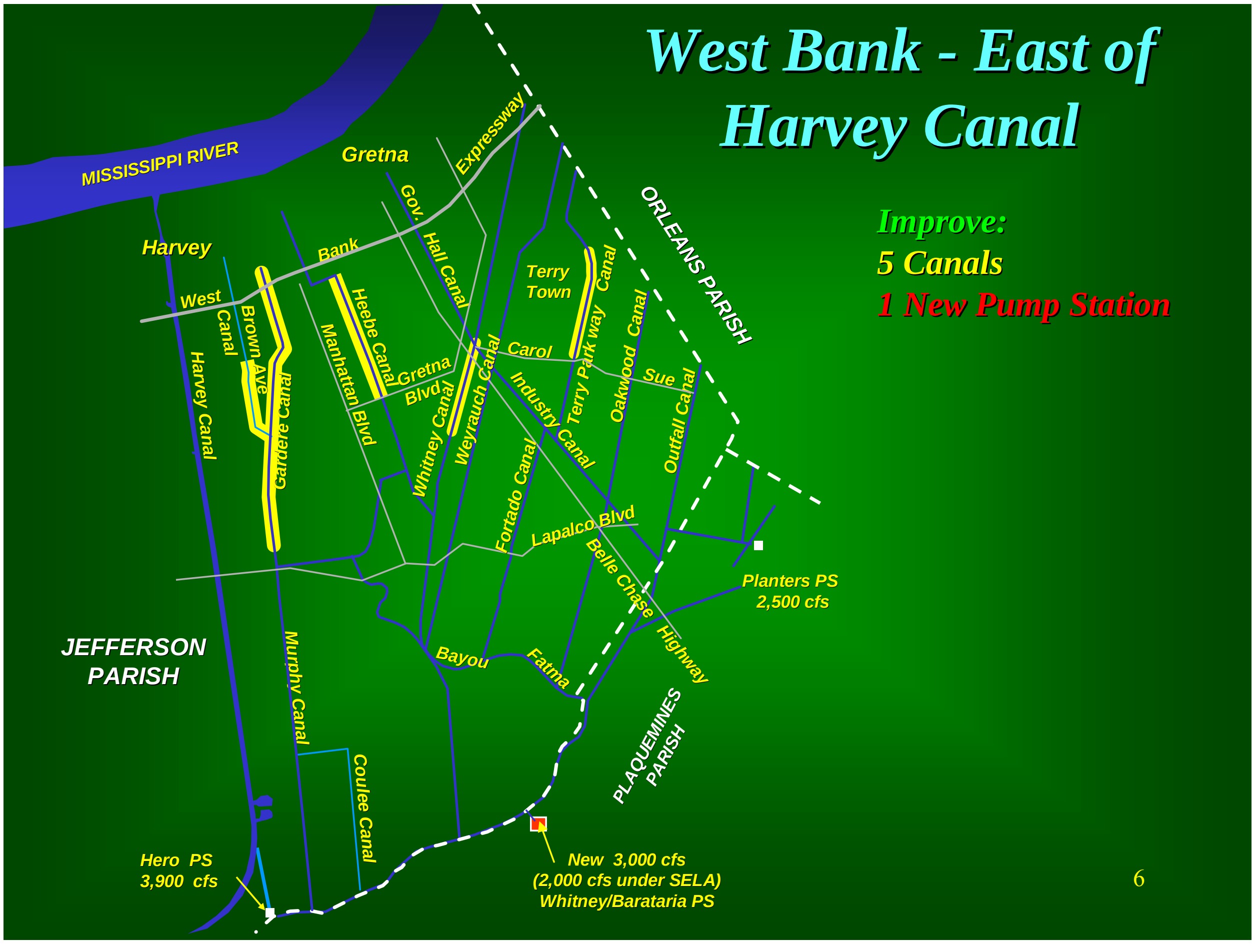


Figure 40. SELA Urban Flood Control Projects in Jefferson West Bank, East of Harvey.

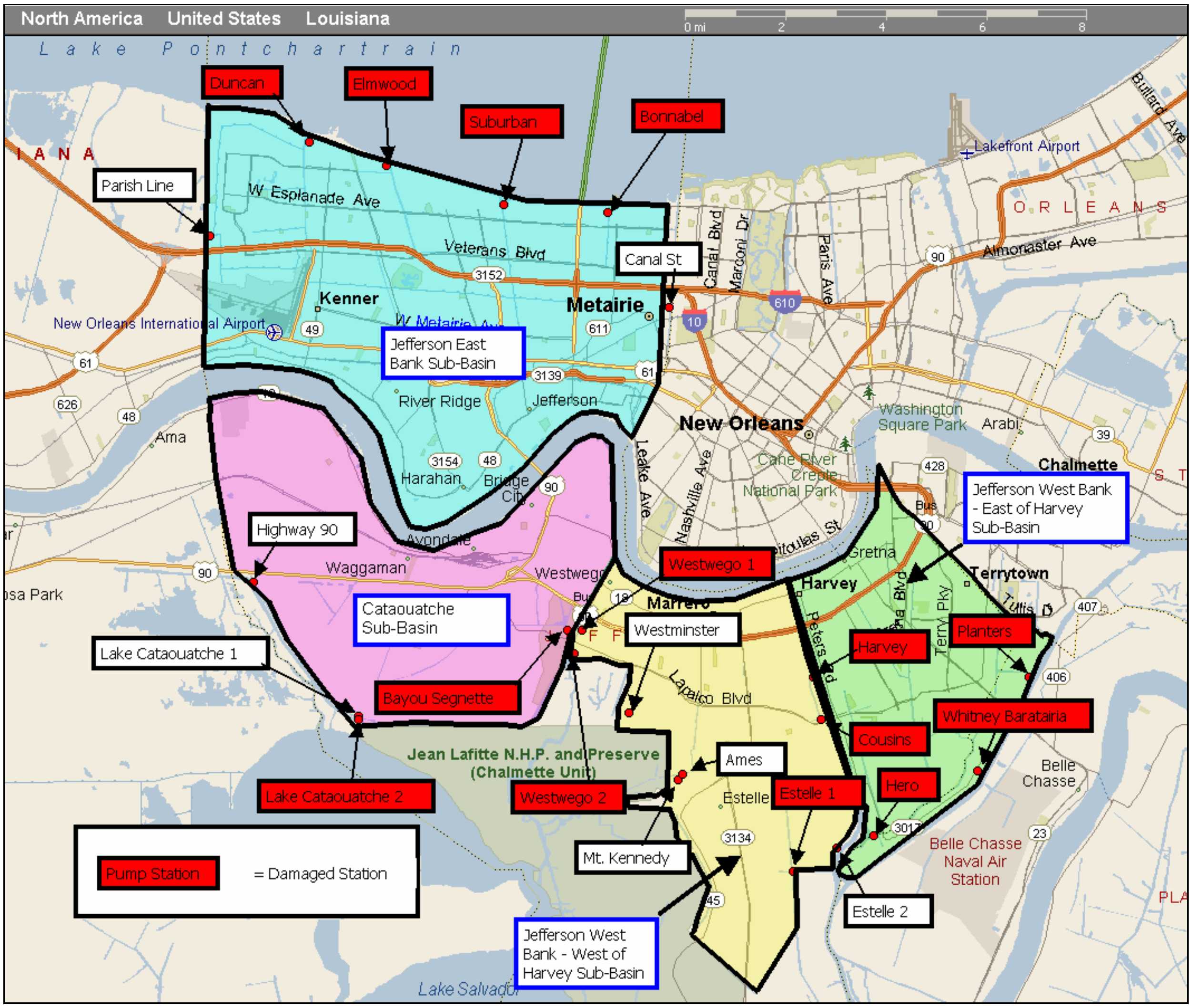


Figure 41. Jefferson Parish pump station Locations.

Table 46 contains information about each individual pump at each pump station in Orleans Parish. The list is composed of information that was collected in the field. Not all information was available for each pump, and missing information was left blank or highlighted.

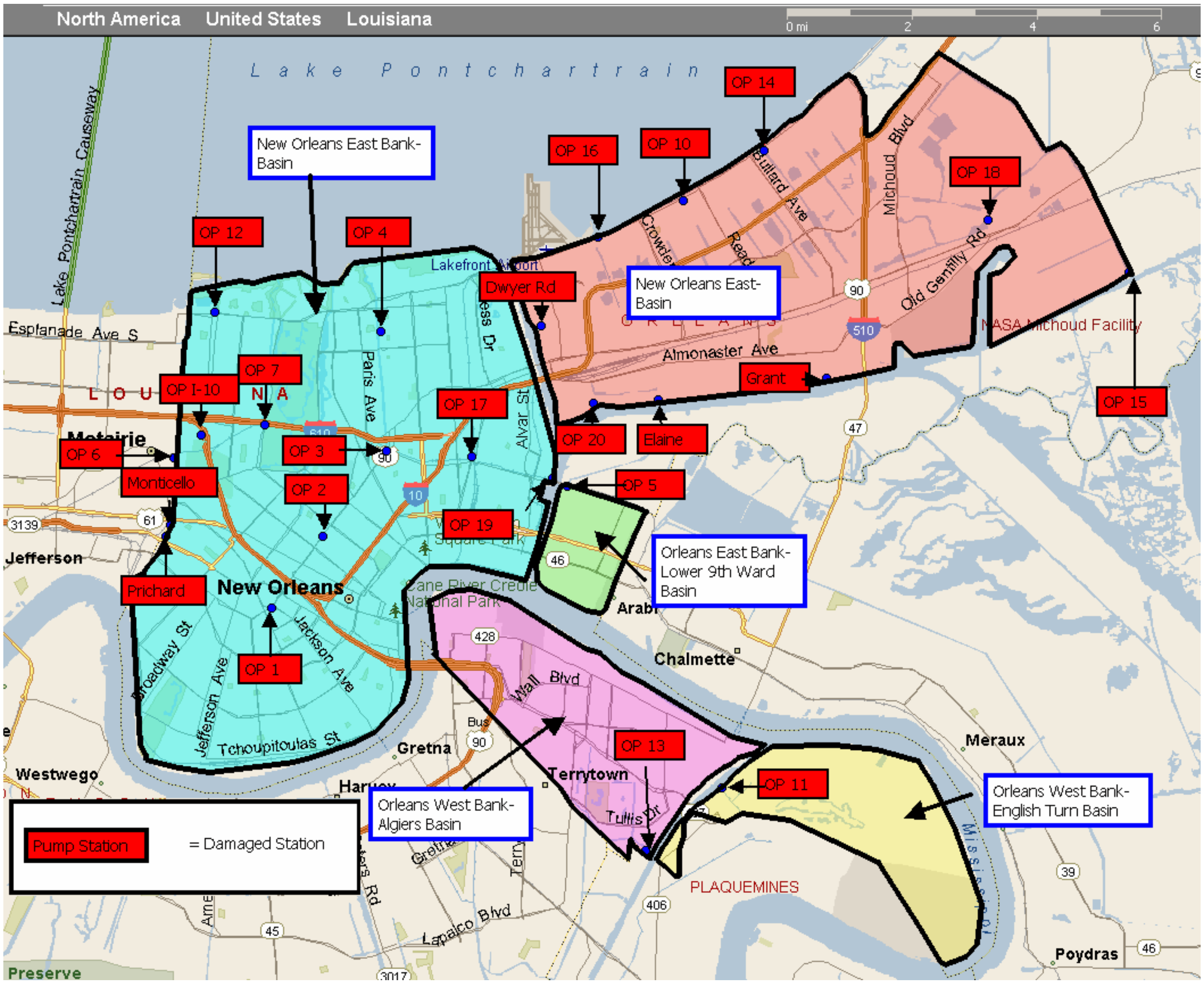


Figure 42. Orleans Parish pump station locations.

Table 45
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank – East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

Table 46
Summary of Orleans Parish Pump Stations by Drainage Basin

Basin	East Bank	East	East Bank-Lower 9th Ward	West Bank-Algiers	West Bank-English Turn	Total
Number of pump stations	12	9	1	1	1	24
Number of pumps	68	24	7	7	5	111
Total rated capacity (cfs)	36,615	4,852	1,850	4,700	1,690	49,707
Estimated cost of damages	n/a	n/a	n/a	n/a	n/a	n/a

Drainage Basins

West Bank – East of Harvey (Jefferson Parish)

The East of Harvey drainage basin on the West Bank has three significant pump stations. The basin is bordered by the Mississippi River on the north, and the Intracoastal Waterway on the southwest. The drainage system consists of the surrounding bodies of water, as well as the Planters Bypass and Hero Outfall Canals. The three pump stations are briefly described below. Volume VI provides more detailed descriptions.

Planters
Intake location: ....Planters Bypass Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....2,356 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	288	n/a	Diesel	n/a
2	288	n/a	Diesel	n/a
3	288	n/a	Diesel	n/a
4	288	n/a	Diesel	n/a
5	52	n/a	Electric	n/a
6	288	n/a	Electric	n/a
7	288	n/a	Electric	n/a
8	288	n/a	Electric	n/a
9	288	n/a	Electric	n/a

Hero
Intake location: ....Hero Outfall Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....3,852 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	100	n/a	Electric	n/a
2	300	n/a	Electric	n/a
3	300	n/a	Electric	n/a
4	1,020	n/a	Diesel	n/a
5	1,020	n/a	Diesel	n/a
6	300	n/a	Electric	n/a
7	203	n/a	Diesel	n/a
8	203	n/a	Diesel	n/a
9	203	n/a	Diesel	n/a
10	203	n/a	Diesel	n/a

Whitney/Barataria
Intake location: ....n/a
Discharge location: .... Intercoastal Canal
Nominal capacity: ....3,750 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1,250	n/a	Electric	n/a
2	1,250	n/a	Electric	n/a
3	1,250	n/a	Electric	n/a

West Bank – English Turn (Orleans Parish)

The West Bank – English Turn drainage basin is bordered by the Intracoastal Waterway on its northwest side. The Mississippi River wraps around its north and east sides. It only has one significant pump station, which is described below. Volume VI provides more detailed information.

OP 11
Intake location: .... Donner Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....1,690 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	250	1953	Electric 25 Hz	Horizontal
B	250	1953	Electric 25 Hz	Horizontal
D	570	1990	Electric 60 Hz	Horizontal
E	570	1990	Electric 60 Hz	Horizontal
CD – 3C	50	1953	Electric 25 Hz	Centrifugal

West Bank – Algiers (Orleans Parish)

The West Bank – Algiers drainage basin is bordered by the Intracoastal Waterway on the southeast. The Mississippi River wraps around the west, north, and east sides. It only has one significant pump station, which is described below. Volume VI provides more detailed information.

OP 13

Intake location: .... Nolan and East Donner Canals
Discharge location: .... Intracoastal Waterway
Nominal capacity: .... 4,700 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
V1	250	1981	Electric 60 Hz	Vertical
V2	250	1981	Electric 60 Hz	Vertical
CD 3	50	1981	Electric 60 Hz	Vertical
D4	1,000	1981	Diesel	Horizontal
D5	1,000	1981	Diesel	Horizontal
6	1,075	1981	Electric 60 Hz	Horizontal
7	1,075	1981	Electric 60 Hz	Horizontal

Levees and Floodwalls.

MRL - MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are no floodwalls that are part of the MRL Project in this reach.

Non-Corps **-** Several Local Interest and/or private levees are located in the project area. No design criteria for these levees have been available to the Corps.

3.3 Post-Katrina Changes to the Protection System

3.3.1. Documents Available

Most documents which support and document the changes to the hurricane protection system can be found in the archives to the Task Force Guardian program. These may include decision documents, assessment documents, and solicitation documents for the construction activities.

- Damage Survey Reports (DSRs) were conducted early after the storm event to quickly describe the conditions to inform federal decision makers.

- Project Information Reports (PIRs) were the authority reports developed to begin repair and construction activities.

- Construction Plans and Specifications were developed by A-E contractors after a solicitation was conducted by Contracting Division of Task Force Guardian or the New Orleans District Contracting Office.

This document solely captures the work being performed by Task Force Guardian. The New Orleans District has also performed work on federal and non-federal components of the hurricane protection system in Jefferson, St. Charles, St. Bernard, and Plaquemines Parishes.

3.3.2. What Exists as of 1 June 2006

3.3.2.1. Orleans East Bank

The Orleans East Bank portion of the program includes the east bank of the Mississippi River between the 17th Street Canal and Inner Harbor Navigational Canal (IHNC). Within these boundaries, project will address damages in the following areas:

- Orleans East Bank Lakefront – 5.2 miles of earthen levee segment located in New Orleans and roughly parallels the shoreline of Lake Pontchartrain between the IHNC on the east and 17th Street Canal on the west. The levee contains seven ramps that traverse the levee profile.

- 17th Street Outfall Canal – The 17th Street Outfall Canal lies in Jefferson Parish immediately west of the Orleans Parish boundary line. The canal extends approximately 2.4 miles from Pump Station No. 6 near Interstate Highway 10 to its confluence with Lake Pontchartrain.

- London Avenue Outfall Canal is located on the south side of Lake Pontchartrain in Orleans Parish, east of the 17th Street and Orleans Avenue Canals. The London Avenue Outfall Canal extends approximately 3.0 miles from Pump Station No. 3 to its confluence with the Lake Pontchartrain.

- Orleans Avenue Outfall Canal is located between 17th Street Outfall Canal and London Avenue Outfall Canal and extends approximately 1.8 miles from Pumping Station No. 7 in the vicinity of I-610 to its mouth at Lake Pontchartrain.

- Damage along IHNC is addressed in a separate section of this document.

Primary damages to the flood protection in the Orleans East Bank basin consist of a 455-ft breach in the east side I-wall along 17th Street Outfall Canal, breaches on both the east side (425 ft) and west side (720 ft) I-wall along London Avenue Outfall Canal, breaches along the west side of IHNC floodwall, and damages to all 15 pumping stations.

In the Orleans East basin, 12 separate construction projects have been identified to repair the damaged areas, not including pump stations, and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $182 million in construction costs.

Project OEB01 includes construction of a sheet-pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet-pile wall will be offset 50 ft into the canal and tied into the existing wall to provide interim protection. The length of breach is 455 ft. Stone will be placed in the canal face of the sheet pile for channel stabilization. This will allow the New Orleans Sewerage and Water Board to operate Pump Station No. 6 at full capacity during normal rainfall events.

Project OEB02 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 455 feet of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0 ft NGVD. This wall is supported by steel H-piles and a steel sheet-pile cutoff wall embedded in the concrete wall.

Project OEB03 includes construction of a sheet-pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet-pile wall will be offset 40 ft into the canal and tied into the existing wall to provide interim protection. The length of breach is 425 ft. Stone will be placed in the canal face of the sheet pile for channel stabilization. This protection will allow the New Orleans Sewerage and Water Board to operate Pump Station No. 3 at full capacity during normal rainfall events.

Project OEB04 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 200 ft of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0 ft NGVD. This wall is supported by steel H-piles and a steel sheet-pile cutoff wall embedded in the concrete wall.

Project OEB05 includes construction of a sheet-pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet-pile wall will be offset 20 ft into the canal and tied into the existing wall to provide interim protection. The length of breach is 720 ft. Stone will be placed in the canal face of the sheet pile for channel stabilization. OEB 5 also contains an interim repair for damaged sheet-pile wall along the east side of London Avenue Outfall Canal opposite the breach on the west side. This interim repair consists of driving a sheet-pile cell to enclose the damaged floodwall.

Project OEB06 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 720 ft of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0 ft NGVD. This wall is supported by steel H-piles and a steel sheet-pile cutoff wall embedded in the concrete wall. OEB6 will include the replacement and repair of damaged floodwall along the east side of London Avenue Outfall Canal near Robert E. Lee Boulevard. The existing I-wall is rotated several inches at the top for length of approximately 500 ft. Under this project, the wall will be replaced with reinforced concrete L-Wall section. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0 ft NGVD. This wall is supported by steel H-piles and a steel sheet-pile cutoff wall embedded in the concrete wall.

Project OEB07 includes intermittent scour repair of 5.2 miles of earthen levee along the Lake Pontchartrain Lakefront. The bulk of the damage is lakeside erosion, scour at the base of the floodwalls, and damaged slope paving. The scour repairs will require a small amount of borrow material.

Project OEB09 includes the construction of an interim gated flood control structure at the confluence of 17th Street Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 3,000 cfs.

Project OEB010 includes the construction of an interim gated flood control structure at the confluence of London Avenue Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 3,000 cfs.

Project OEB011 includes the construction of an interim gated flood control structure at the confluence of Orleans Avenue Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 2,500 cfs.

Project OEB012 includes the construction of a levee tie-in connecting the interim closure structure to the Lake Pontchartrain Hurricane Protection Levee along Lakeshore Drive. This levee tie-in is a combination of steel sheet pile and earthen levee with back side slope paving for scour protection.

Project OEB013 includes the construction of a levee tie in connecting the interim closure structure to the Lake Pontchartrain Hurricane Protection Levee along Lakeshore Dr. This levee tie-in is a combination of steel sheet pile and earthen levee with back side slope paving for scour protection.

3.3.2.2. IHNC

The Inner Harbor Navigation Canal (IHNC) portion of the program includes the flood protection paralleling the IHNC from the Mississippi River to Lake Pontchartrain.

The IHNC work area contains approximately 10 miles of levee and floodwalls along the Inner Harbor Navigation Canal in a heavily industrialized area.

Overtopping of the hurricane protection by Hurricane Katrina was evident along nearly all portions of the canal. There were four breaches in the protection system, two on the east side and two on the west side. The east side breaches are both located in the lower 9th ward neighborhood and the west side breaches are both in the vicinity of France Road and Benefit Street. Temporary repairs and closures were made in these areas until permanent restoration work is completed. This project under Task Force Guardian will restore the protection back to pre-hurricane Katrina conditions. In the areas of the breaches, this project will replace/repair those walls back to pre-storm project authorized elevations. In the areas of scour, those walls and scour will be repaired accordingly.

In the IHNC area, eight separate construction projects were identified to repair the damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $62.7 million in construction costs.

Project IHNC-01 – There is approximately 4,000 lineal feet of concrete I-wall flood barrier along the east side of the IHNC between North Claiborne Avenue and Florida Avenue. The damages in this reach consisted of a breach of the floodwall immediately south of Florida Avenue (250 ft) and one approximately 100 yards north of Claiborne Avenue (850 ft) with the remaining portions of the floodwall having areas of severe scour and tilting of the I-wall. The work includes replacement of the concrete I-wall with a concrete T-wall, supported on H-piles and sheet piling.

Project IHNC-02 – This section of the project consists of concrete I-wall. The damage in this area consisted of a breach of the floodwall at the container terminal along France Road. There was also heavy scour of the floodwall in this area. The repairs consist of removing approximately 1,300 lineal feet of the damaged concrete I-wall and replacing the damaged section of wall with new concrete L-wall. The new wall will be supported by steel H-piles and longer steel sheet piles.

Project IHNC-03 – There are approximately 2.75 miles of floodwall and levee along the east side of the IHNC between the Gulf Intracoastal Waterway and Lake Pontchartrain and another 1,000 lineal feet of floodwall on the west side of the IHNC between Almonaster Avenue and Highway 90. The damages in this reach consisted of intermittent scour of the levee and scour and damage at the wall/gate closures and at the wall/levee interfaces. The repairs consist of filling in the scour areas, repairing the gate concrete sills and seals, installing new sheet piling, placing rock and ballast, and placing stone erosion protection.

Project IHNC-04 – On the west side from Highway 90 to Lake Pontchartrain and on the east side between Dwyer Street and Hayne Boulevard, flood protection consists of concrete I-wall that experienced relatively minor scour damage along its base. The repairs consist of filling in the scour areas, and cleaning existing and installing new relief wells.

Project IHNC-05 – This portion of the project consists of approximately 1,600 ft of existing levee and concrete floodwall that extends from the vicinity of France Road ramp towards the IHNC. This area was breached and experienced severe scour. The repair consists of replacement with a new concrete T-wall.

Project IHNC-07 – There is approximately 1,400 lineal feet of concrete I-wall flood barrier along the east side of the IHNC between the IHNC Lock and North Claiborne Avenue. The damages along this reach consisted of intermittent scour along the base of the floodwall. The work includes filling in the scour repairs and providing erosion protection.

Project IHNC-08 – On the west side of the IHNC from 700 ft north of Benefit Street to Highway 90, flood protection consists of concrete I-wall embedded in compacted earthen levee embankment. The damages in this area consisted of scour along the base of the floodwall. The repairs consist of scour repair and erosion protection.

Project IHNC-09 – On the west side of the IHNC from the lock to Florida Avenue, flood protection consists of concrete I-wall. The damages in this area consisted of scour along the base of the floodwall. The repairs consist of scour repair and erosion protection.

3.3.2.3. New Orleans East

The New Orleans East portion of the flood protection system is bounded by the east bank of the Inner Harbor Navigational Canal (IHNC), Lake Pontchartrain shoreline between the IHNC and South Point, the eastern boundary of the Bayou Sauvage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) between the IHNC and eastern edge of the

Bayou Sauvage National Wildlife Preserve. Within these boundaries, project will address damages in the following areas:

- New Orleans East Lakefront includes the Citrus Lakefront Levee and New Orleans East Lakefront Levee consisting of 12.4 miles of earthen levee paralleling the Lakefront from the IHNC to South Point. It also includes floodwalls at the Lakefront Airport and Lincoln Beach.

- GIWW – The New Orleans East basin includes the north bank of the GIWW Flood Protection System from the IHNC to the eastern edge of the Fish and Wildlife Preserve. The system contains the Citrus Back Levee and New Orleans East Back Levee, which consists of approximately17.5 miles of earthen levees and concrete floodwalls.

- The New Orleans East Levee consists of 8.4 miles of earthen levee from South Point to the GIWW along the eastern boundary of the Bayou Sauvage National Wildlife Preserve.

Primary damages to the flood protection in the New Orleans East basin consists of 12,750 ft of levee breach in the New Orleans East Back Levee between Michoud Canal and the CSX Railroad along the GIWW; a couple of floodwall breaches in this reach at Pump Station 15 (800 ft) near the Maxent Levee, and at the Air Products Hydrogen Plant near the Michoud Canal (300 ft); floodgate floodwall and adjacent levee damage at the CSX railroad; and 2,000 feet of floodwall damage in the Citrus Back Levee along the GIWW between the IHNC and Paris Road. The other damages consist mostly of levee and floodwall scour at various locations throughout the New Orleans East basin and damages to all eight pump stations.

In the New Orleans East basin, ten separate construction projects have been identified to repair the damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $83 million (not including pump stations) in construction costs.

NOE01 – Project NOE01 consists of rebuilding approximately 5 miles of the existing levee up to elevation 19.5 ft NGVD with 680,000 cu yd of earthen material, and seeding and fertilizing. The entire reach of levee was brought up to an interim level of protection of elevation +10 ft NGVD by 15 November 2005.

NOE02 – Project NOE02 includes removing the damaged steel sheet-pile wall, installing a new concrete T-wall, filling in scour holes, and bringing the damaged levee back up to pre-hurricane Katrina elevation.

NOE03 – Project NOE03 includes removing the damaged concrete I-wall and steel sheet-pile wall, filling in scour holes, installing new sheet pile, raising the damaged levee to pre-hurricane Katrina elevation, and then seeding and fertilizing. The damaged reach was first brought up to an interim level of protection of elevation +10 ft NGVD by 15 November 2005 before final repairs are made.

NOE04 – Project NOE04 includes removing the damaged concrete I-wall sections, filling in the scour holes, regrading the damaged levee, constructing new concrete L-wall, and putting in slope paving and an earthen stability berm on the landside of the wall. The repaired levee section and stability berm will be seeded and fertilized. The damaged reach was first brought up to an interim level of protection of elevation +10 ft NGVD by 1 December 2005 before final repairs are made.

NOE05 – Project NOE05 includes the removal of the existing concrete wall and railroad closure gate, filling the scoured areas, constructing a new closure gate and new concrete T-walls and I-walls, placement of riprap, concrete slope paving, and concrete roadway.

NOE06 – Project NOE06 consists of filling in the scour holes and placing a concrete pavement section next to the concrete wall. It also includes filling in the scour hole and paving the damaged road section with concrete at the interface of the Floodgate L-15 concrete wall and levee.

NOE07 – Project NOE07 includes intermittent scour repair along approximately 19 miles of earthen levee along the Lake Pontchartrain Lakefront and the eastern boundary of the Bayou Sauvage National Wildlife Preserve. The work consists of filling in the scour areas with semi-compacted fill, reshaping where needed, and seeding and fertilizing.

NOE08 – Project NOE08 includes filling in the scour holes and capping with gabion structures around several gated drainage control structures to prevent future erosion. The gabion structures are wire baskets filled with stone interlocked to form a surface erosion barrier.

NOE09 – Project NOE09 includes filling in the scour holes next to the existing concrete I-wall floodwall with embankment material, installing bedding material, grouted riprap, and concrete slope paving above the scour to prevent future erosion. It also includes adding an earthen stability berm on both flood and protected sides of the wall. The project also consists of intermittent repairs to damaged concrete and various joints and gates in the walls, and the installation of relief wells and sheet pile in selected areas.

NOE10 – Project NOE10 includes filling in the scour holes next to the floodwalls with embankment material, installing bedding material, and concrete slope paving above the scour to prevent future erosion. These walls are around pump stations and utility lines along Lakefront and New Orleans East Levee systems.

3.3.2.4. St. Bernard Parish

The St. Bernard basin Hurricane Protection System includes the levee/floodwall extending from the Inner Harbor Navigation Channel (IHNC) easterly, along the Gulf Intracoastal Waterway (GIWW), to the Bayou Bienvenue Control Structure, continuing along the Mississippi River Gulf Outlet (MRGO) southeasterly, then turns generally to the west, where it ties into the Mississippi River Levee at Caernarvon. A portion of the hurricane protection system in this area also provides hurricane protection to the Lower 9th Ward area in Orleans Parish. Within this area of protection, the Task Force Guardian authorities will address damages to the following project features:

- About 8 miles of the 30 total miles of hurricane protection levee were damaged:
  - Most severely damaged levees are along the reach adjacent to the MRGO extending from the Bayou Bienvenue Control Structure to the southeast for 11.8 miles
  - Minor levee scour along GIWW in Orleans Parish
  - Miscellaneous scour on the levee from MRGO to Caernarvon
- Repair of Bayou Dupre Control Structure
- Repair of Bayou Bienvenue Control Structure
- Repair of five floodgates, floodwall, and minor levee damages from Bienvenue Control Structure to GIWW lock
- Repair Creedmore Structure.

The New Orleans District has performed work on the non-federal St. Bernard Back Levee.

In the St. Bernard Parish, nine separate construction projects were identified to repair damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $50.2 million in construction costs.

STB 01 – The work for this project included site preparation work in the areas of levee damage between the Bayou Bienvenue and Bayou Dupre Control Structures. The contracted work (rental agreement contract) is complete.

STB 02 – The work for this project included site preparation work in the borrow areas between the Bayou Bienvenue and Bayou Dupre Control Structures. The borrow area is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO Canal. This rental agreement contract is complete.

STB 03 – The 5.6-mile reach of levee along the MRGO extending east from the Bayou Dupre Control Structure was severely damaged from overtopping. The entire levee reach is being restored to the design grade elevation, requiring the placement of an estimated 800,000 cu yd of fill material. The borrow area for this fill material is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO Canal. Protection is restored when the levee reaches elevation 17.5 ft NGVD.

STB 04 – The 6.2-mile reach of levee along the MRGO between the Bayou Bienvenue and the Bayou Dupre Control Structures was also severely damaged from overtopping. The entire levee reach is being restored to the design grade elevation, requiring the placement of an estimated 1,350,000 cu yd of fill material. The borrow area for this fill material is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO Canal. Clay material is also being barged into the site to supplement onsite borrow. Protection is restored when the levee reaches elevation 17.5 ft NGVD.

STB 05 – Minor scour repairs are needed on the backside of the levee and structural and structural backfill scour adjacent to floodwalls and four closure structures, which are located between the Bayou Bienvenue Control Structure and the Florida Avenue Bridge, with most of the levee reach adjacent to the GIWW. An estimated 26,000 cu yd of fill material are required for this work, which is being furnished by the contactor.

STB 06 – The work on this project includes repair of structural damage and loss of structural backfill at the Bayou Dupre Control Structure. A significant scour hole is to be filled with 17,500 cu yd of granular backfill and protected with grouted riprap. An estimated 22,500 tons of riprap and 13,400 cu yd of embankment fill are required for the repairs.

STB 07 – The work on this project includes repair of structural damage and loss of structural backfill at the Bayou Bienvenue Control Structure. A significant scour hole is being filled with 28,600 cu yd of granular backfill and protected with grouted riprap. An estimated 32,100 tons of riprap and 3,400 cu yd of embankment fill is required for the repairs.

STB 08 – The work includes repair of minor scour on the backside of the levee from the MRGO to Caernarvon, which is about 10.8 miles in length. An estimated 36,000 cu yd of fill material are required for this work.

STB 09 – The work includes constructing a cofferdam and removing debris from the structure to permit closure of the gates and inspection of the structure to determine if further repairs are necessary. Contract work is complete.

3.3.2.5. Plaquemines Parish

The Plaquemines Parish basin includes long, narrow strips of protected land on both sides of the Mississippi River between New Orleans and the Gulf of Mexico. The Mississippi River Levees (MRL) protect the Parish from river flooding. Protection from hurricane-induced tidal surge is achieved by the New Orleans to Venice (NOV) hurricane protection system. The NOV is a system of levees on the gulf side of the protected lands. Additional berms and floodwalls are constructed on top of the MRL in the lower part of the parish, where hurricane-induced water levels are higher than river flood stages. The distance between the gulf-side levees (back levees), and the MRL is less than a mile in most places.

Altogether the Plaquemines Parish MRL and NOV systems include 162 miles of levee and 7 miles of floodwall. There are fifteen non-federal pump stations for interior drainage. The levees are crossed by numerous oil pipelines, constructed in various manners. Some crossings bridge the levee without touching the embankment; some are constructed on top of the line of protection; and some pass through the line of protection with measures to prevent seepage. There is a wicket gate closure on the back levee at Empire, where a shipping canal connects the Mississippi River to the Gulf of Mexico.

- The Plaquemines Parish East Bank MRL system extends from the Parish line at Braithwaite 35 miles downstream to Bohemia. The floodside slopes have concrete slope pavement from the bottom of the embankment to the design high water level. The crown is surfaced with 9 in. of crushed limestone. The freeboard and protected-side slopes are grassed.

- The East Bank NOV back levee runs between Phoenix and Bohemia, a distance of 16 miles. It is a grass-covered earthen levee.

- The West Bank Plaquemines MRL system extends from the parish line at Belle Chasse, 70 miles downstream to Venice. Its composition is similar to the East Bank MRL with concrete slope pavement, crushed limestone surface course, and the remaining slopes grassed. Below Port Sulphur (29 miles above Venice) the MRL design grade is lower than the NOV hurricane design grade, so the NOV is constructed as berms or floodwalls on top of the MRL.

- The West Bank NOV extends from St. Jude to Venice, a distance of 36 miles. The NOV protection along the river includes 6 miles of floodwalls in 13 distinct reaches, projecting above the MRL from 2 to 8 ft. The back levee is a grass covered earthen embankment.

All of the levees in Plaquemines Parish sustained damage from Hurricanes Katrina and Rita. There was considerable crown and slope scour along the total length. The MRL slope pavement sustained damage from the hundreds of ships and barges that crashed upon it. There were also several severe breaches, coinciding with pipeline crossings and with some floodwalls. Five of the 6 miles of NOV floodwall along the Mississippi River were damaged beyond repair. There were major breaches at sheet-pile wing walls at two pump stations in the back levee. A major breach occurred at the Shell pipeline crossing near Nairn. And the West Pointe a la Hache pipeline crossing was severely damaged. Wind and water damage from Katrina and Rita severely impacted nearly every structure within the East Bank area of protection and on the West Bank below Myrtle Grove (50 miles above Venice).

New Orleans District has performed repair work on the non-federal levees in Plaquemines Parish.

Task Force Guardian has divided the Plaquemines Parish flood protection recovery process into 22 projects, labeled P01 through P26, with no projects for P05, P09, P10, or P23 (the work of the missing projects was combined with the other 22):

P01 – This project consisted of preparing the borrow area, at the southern end of the project, for use by levee-construction contractors. The work involved clearing and burning vegetation from 40 acres.

P02 – Walker Road borrow is used for most of the MRL projects because of its high quality material and the speed which it could be brought into production. This work was performed by the Memphis District Corps of Engineers. Altogether, more than 390,000 cu yd of fill was excavated and processed by the time the MRL projects were completed in March 2006.

P03 – This project was for repair of the Gravolet Canal breach near Bohemia, LA. The 15-ft-deep by 550-ft-wide breach was temporarily closed under Task Force unwatering, and some of that temporary repair has been incorporated into this permanent project. The work consisted of preparing the borrow area, excavation, and reconstructing the levee by placing fill and armor stone, and restoring the surface by fertilizing and seeding.

P04 – This project, for cleaning and repairing levee scour sites, was executed by the Memphis District Corps of Engineers Revetment Unit.

P06 – This project was executed under a rental contract for equipment and personnel to repair scour sites along a 20- mile reach of river levee. The work included hauling, placing, and compacting fill and crushed limestone surfacing, as well as fertilizing and seeding disturbed areas. The project used government-furnished borrow from Walker Road borrow pit (Project P02).

P07 – This project was executed under a rental contract for furnishing equipment and personnel to repair scours along a 5-mile reach of river levee. The work included hauling, placing, and compacting fill and crushed limestone surfacing, as well as fertilizing and seeding disturbed areas. The project used government-furnished borrow from Walker Road borrow pit (Project P02).

P08 – This project was for repair of slope and crown scour along the 20-mile reach from Port Sulphur to Fort Jackson. Approximately 350,000 cu yd of government-furnished material from the Walker Road and Buras borrow pits was required. The contract included demolition of the Buras floodwall, which was required before the embankment could be repaired.

P11 – Project P11, for shaping the MRL along the 10-mile reach from Fort Jackson to Venice, was completed by New Orleans District and Memphis District hired labor. They used bulldozers to patch and compact scour sites with material recovered from the toe of the levee and borrowed from the damaged NOV section on top of the MRL. The NOV will be repaired under Project P12.

P12 – This project will reconstruct the NOV hurricane protection levee on the same 10-mile reach that P11 addresses on the MRL. The NOV design grade in this reach is between 2.6 and 3.5 ft higher than the MRL. The project requires 210,000 cu yd of fill material from new borrow pits in Buras and Triumph.

P13 – This project will replace damaged NOV hurricane protection floodwalls with levees constructed above and behind the MRL levee. The NOV design grade in this reach is between 1.7 and 2.1 ft higher than the MRL. The project requires approximately 145,000 cu yd of fill from a new borrow pit in Port Sulphur.

P14 – This project will replace damaged NOV hurricane protection floodwalls with levees constructed above and behind the MRL levee between Empire and Buras. The NOV design grade in this reach is between 2.9 and 3.1 ft higher than the MRL. The enlargement requires approximately 600,000 cu yd of fill.

P15 – This project is for repairs to the Empire Canal floodgate. The gate is currently inoperable and in need of structural, mechanical, and electrical repairs. The structure must be dewatered to accomplish the construction.

P16 – This project is for repair of crown and slope scour that the 10-mile reach, B2, incurred from Hurricane Katrina. This lowest reach of the back levees, from Fort Jackson to Venice, was covered with marsh grass, debris, and boats when the floodwaters receded, but the levee was not severely damaged and there were no breaches.

P17 – This project will replace damaged NOV hurricane protection floodwalls with levees near Buras. The NOV design grade in this reach is 3.1 ft higher than the MRL. The project will utilize approximately 310,000 cu yd of fill from a new borrow pit in Triumph.

P18 – This project was for repair of levee crown and slope scour along the 11-mile-long B-1 Reach, from Empire to Fort Jackson. The levee was not severely damaged in this reach, so overall quantities were small; only 14,000 cu yd of fill was required for the entire project.

P19 – This project is for repair of crown and slope scour along the 18-mile-long Reach A, from City Price to Empire. This levee sustained significant damage at several places, including severe crown scour, breaches, and wall tie-in failures. All together more than 300,000 cu yd of fill material is required.

P20 – This project will repair floodwalls at Sunrise and Hayes Pump Stations. Emergency sheet-pile walls were constructed at Sunrise during Task Force Unwatering, to achieve closure of a deep, wide breach. This project will make those repairs permanent and will construct additional sheet-pile wing walls at the Hayes Pump Station.

P21 – This project will make repairs to floodwalls at Freeport, Home Place Marina, Gainard Woods Pump Station, and Diamond Pump Station. Floodwalls are sheet-pile I-walls; some are capped with reinforced concrete. Improvements include replacing I-walls with T-walls or embankment, or adding fill to reduce the height of wall stick-up.

P22 – This project was for repair of a levee breach at Woodland, on the west bank of the Mississippi River. The rental contractor was moved from a completed project in St. Bernard Parish in order to complete the repair before 1 December.

P24 – This project will replace damaged NOV hurricane protection floodwalls with levees. The NOV design grade in this reach is between 1.7 and 2.1 ft higher than the MRL. The project will require approximately 582,000 cu yd of fill from a new borrow pit at Myrtle Grove. This borrow pit will eventually be incorporated into the planned Mississippi River Diversion Channel at Myrtle Grove.

P25 – This project was to reconstruct the levee section, extend sheet-pile cutoff walls 30 ft upstream and downstream of the siphon, and replace slope pavement on both sides and on the crown of the levee.

P26 – This project is for removing damaged sections of the reinforced concrete slope pavement and filling the holes with riprap to protect the levee. The riprap repairs are an interim remedy, designed to last for several years until concrete sections can be re-cast.

3.3.2.6. St. Charles Parish

No post-Katrina hurricane restoration work has been performed by Task Force Guardian. Construction work has been performed by the New Orleans District.

3.3.2.7. Jefferson East Bank

No post-Katrina hurricane restoration work has been performed by Task Force Guardian. Work has been performed by the New Orleans District.

3.3.2.8. West Bank and Vicinity

No post-Katrina hurricane restoration work has been performed by Task Force Guardian. Construction activities continue under the direction of the New Orleans District.

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