Mississippi Valley Division,
Regional Planning and Environment Division South

St. Tammany Parish, Louisiana Feasibility Study

Appendix D – Engineering

March 2024

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SECTION 1

INTODUCTION

1.1 DATUM

All elevations are referenced to the North American Vertical Datum of 1988 (NAVD 88 (Geoid 12B)) unless otherwise noted.

1.2 PERIOD OF ANALYSIS

The period of analysis for the project is 50 years, from year 2032 to year 2082.

1.3 FINAL ARRAY OF ALTERNATIVES

The Final Array of Alternatives carried forward from hydraulic and hydrologic modeling, preliminary engineering and design, development of full cost estimates, and environmental and resource analysis. The Final Array of Alternatives, including the measures for each alternative, is summarized in the main report. Refer to Annex D1 of this appendix for maps of each Alternative in the Final Array. For levee design criteria, refer to Section 2 of this appendix.

1.3.1 Alternative 1: No Action

Under the No Action Alternative, no risk reduction would occur. The study area would continue to experience damages from riverine, rainfall, surge, and coastal storm related surge and flooding.

1.3.2 Alternative 2: Nonstructural

The Nonstructural (NS) Plan consists of parish-wide measures in areas of flood damage (flood risk management and coastal storm risk management) to structures. This alternative would include flood proofing (dry and wet), structure raising, buyouts, and relocations.

Note: There is no alternative 3 in the final array of alternatives. Alternative 3 was screened earlier in the planning process and was not carried forward to the final array.

1.3.3 Alternative 4: Lacombe

Alternative 4 includes three potential alignments of new levee to reduce flooding. Measures 4a, 4a.1, and 4b were evaluated in the Final Array, resulting in possible selection of one for the tentatively selected plan (TSP). Alternatives 4a, 4a.1, and 4b are mutually exclusive.

Only one levee alignment could be selected if justified; however, a justified levee could be combined with other alternatives. Refer to Figure D:1-1.

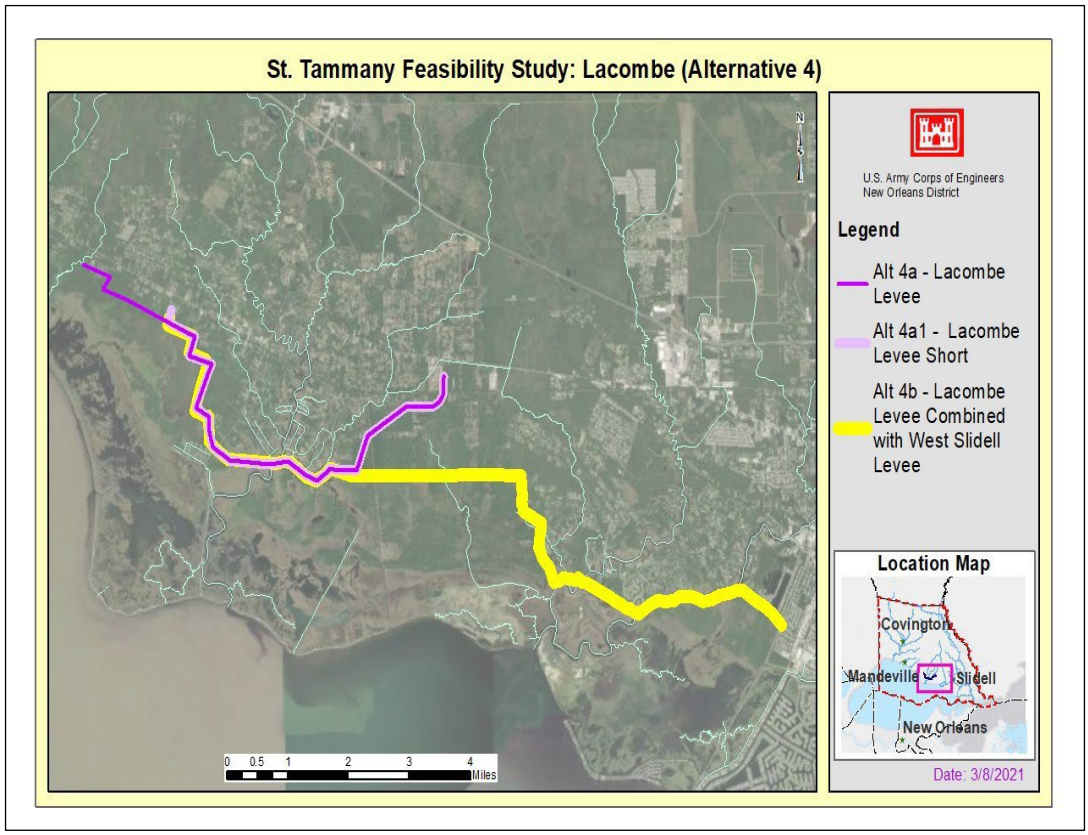


Figure D:1-1. Alternative 4 Lacombe

1.3.3.1 Alternative 4a: Lacombe

Alternative 4a consists of a new levee in the unincorporated Lacombe, Louisiana, to reduce flooding. This alternative also consists of floodwalls, pump stations, floodgates, vehicular floodgates, and ramps. Refer to Figure D:1-2.

1.3.3.1.1 Description of Alignment

The new levee alignment would be situated south of U.S. Highway 190 in Lacombe from east of Bayou Cane to east of Cypress Bayou. The levee would be constructed on the south side of U.S. Highway 190 in Lacombe from east of Bayou Cane to east of Cypress Bayou and consist of approximately 9 miles (47,700 feet) of continuous levee.

1.3.3.1.2 Levee Design Section and Borrow Quantities

The new levee would be designed using a preliminary design elevation of 12.5 feet NAVD 88. Quantity calculations are based on the use of existing ground elevations obtained from the terrain raster dataset. The preliminary project delivery team (PDT) assumptions are a 10- foot-wide levee crown and side slopes of 1V:3H. The construction of this levee alignment would impact approximately 110 acres. This levee alignment would require 595,000 cubic yards of fill (borrow material) (includes 30 percent contingency). Berm sections would be designed and quantified once data is available for analysis.

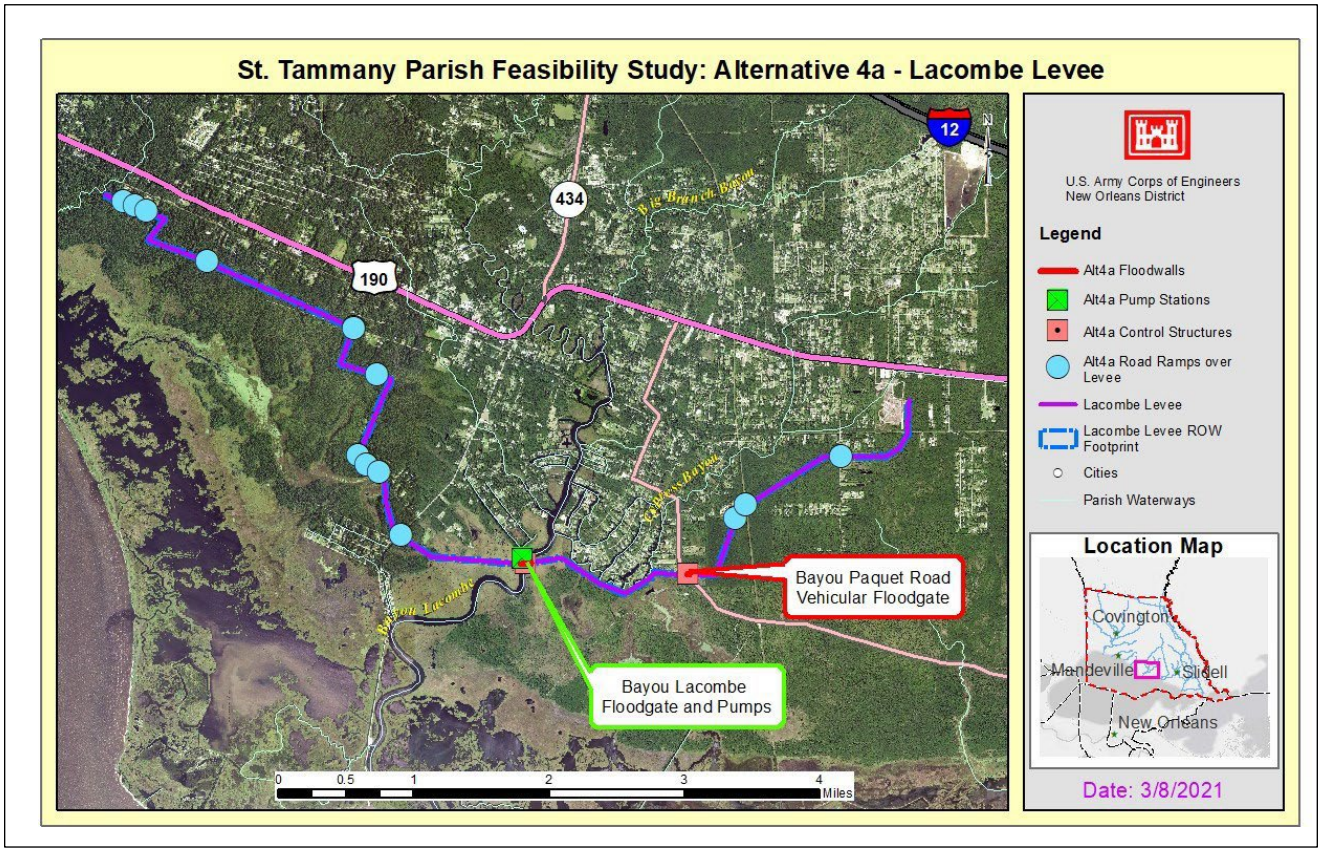


Figure D:1-2. Alternative 4a Lacombe Levee

1.3.3.1.3 Pump Stations and Floodgates

Alternative 4a includes a 300-foot-long pump station complex across Bayou Lacombe with a capacity of 3,200 cubic feet per second (cfs). This complex includes a 20-foot navigable floodgate. Pump station and floodgate construction would impact 12.6 acres. The preliminary design elevation in the vicinity of Bayou Lacombe would be 14.5 feet NAVD 88.

1.3.3.1.4 Vehicular Floodgates and Ramps

Alternative 4a includes 14 vehicular road ramps over the levee and one vehicular floodgate to provide vehicular access through the levee.

The ramps, listed from west to east, would be located at the following road crossings: Ferrier Estates Street, Monique Street, Dalmas Street, Pontchartrain Drive #1, 24th Street, Pontchartrain Drive #2, Barringer Road #1, Barringer Road #2, Barringer Road #3, Barringer Road #4, and Lake Road/LA Highway 434. On the east of Bayou Lacombe Pump Station and Gate Complex, a vehicular floodgate would be located at Bayou Paquet Road, with ramps at Chene Drive #1, Chene Drive #2, and Transmitter Road.

1.3.3.2 Alternative 4a.1: Lacombe Levee Short

Alternative 4a.1 consists of a shorter levee alignment on the west side of the Lacombe community, as compared to Alternative 4a. The levee alignment consists of approximately 7.5 miles (39,000 feet) in Lacombe to reduce flooding. This alternative also consists of floodwalls, floodgates, vehicular floodgates, and ramps. Refer to Figure D:1-3.

1.3.3.2.1 Description of Alignment

The new levee extends on the south side of U.S. Highway 190 from Shelby Drive to east of Cypress Bayou and consists of approximately 7.5 miles (39,000 feet) of continuous levee.

1.3.3.2.2 Levee Design Section and Borrow Quantities

The preliminary Alternative 4a.1 design elevation is 12.5 feet NAVD 88. Quantity calculations are based on use of existing ground elevations obtained from the most current terrain raster dataset. The preliminary PDT assumptions are a 10-foot-wide levee crown with 1V:3H side slopes resulting in an estimated quantity fill material of 574,000 cubic yards, which includes a 30 percent contingency. Acreage impact of Alternative 4a.1 is estimated at approximately 90 acres. Berm sections shall be designed and quantified once data is available for analysis.

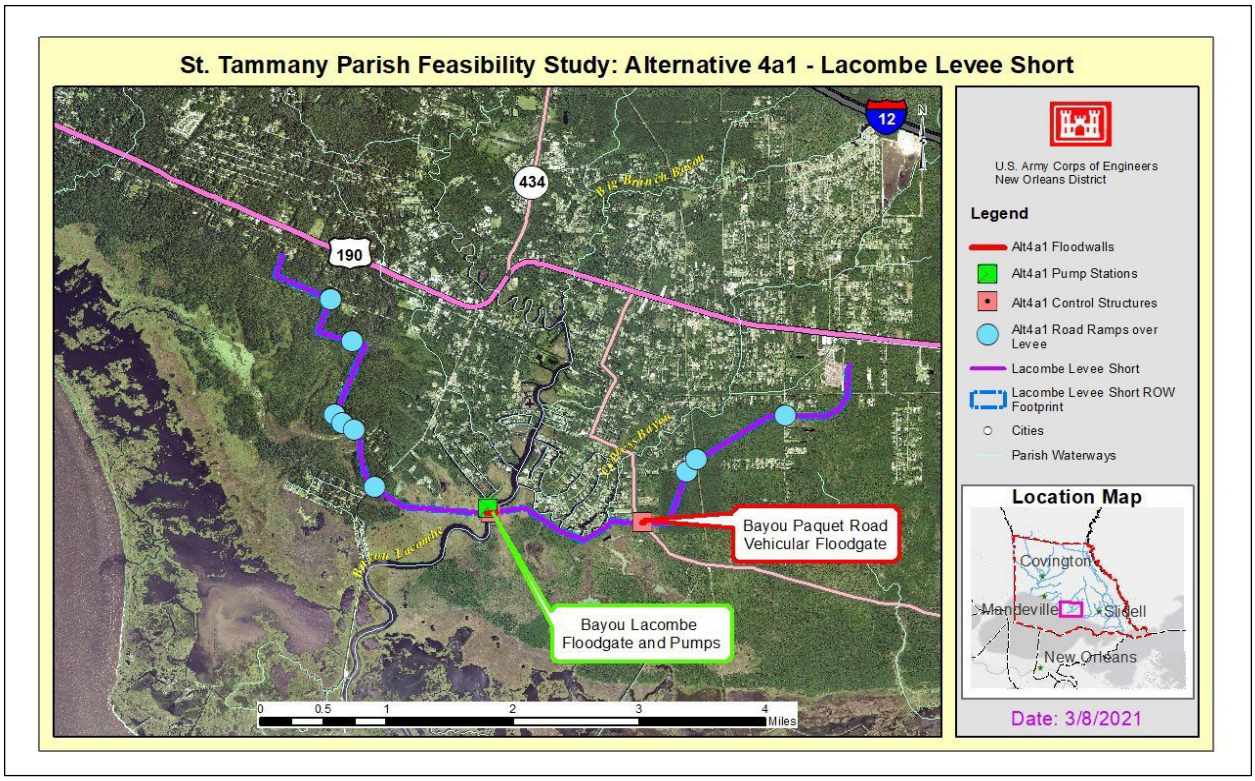


Figure D:1-3. Alternative 4a.1 Lacombe Levee Short

1.3.3.2.3 Pump Stations and Floodgates

Alternative 4a.1 includes a 300-foot-long pump station complex across Bayou Lacombe with a capacity of 3,200 cfs. This complex includes a 20-foot navigable floodgate. The construction of the pump station and floodgate would impact 12.6 acres. The preliminary design elevation in the vicinity of Bayou Lacombe would be 14.5 feet NAVD 88.

1.3.3.2.4 Vehicular Floodgates and Ramps

Alternative 4a.1 includes 10 vehicular road ramps over the levee and one vehicular floodgate provide vehicular access through the levee. The vehicular ramps, listed from west to east, would be located at the following road crossings: 24th Street, Pontchartrain Drive, Barringer Road #1, Barringer Road #2, Barringer Road #3, Barringer Road #4, and Lake Road/LA Highway 434. On the east side of Bayou Lacombe Pump Station and Gate Complex, there would be a vehicular floodgate at Bayou Paquet Road, and with additional ramps at Chene Drive #1, Chene Drive #2, and Transmitter Road.

1.3.3.2.5 Structural Assumptions for Alternative 4a and 4a.1

The new Bayou Lacombe Pump Station is assumed to have similar components and configuration as the U.S. Army Corps of Engineers (USACE) Westshore Lake Pontchartrain Reserve Relief Canal Pumping Station (WSLP Pump Station).

1.3.3.3 Alternative 4b: Lacombe Levee Combined with West Slidell Levee

Alternative 4b consists of approximately 13.7 miles (72,000 feet) of levee, which would combine the Lacombe levee from Alternative 4a.1 and the West Slidell levee from Alternative 5, to reduce flooding in the Lacombe, Slidell, and the area between the two cities. This alternative also consists of floodwalls, floodgates, vehicular floodgates, and ramps.

Refer to Figure D:1-4.

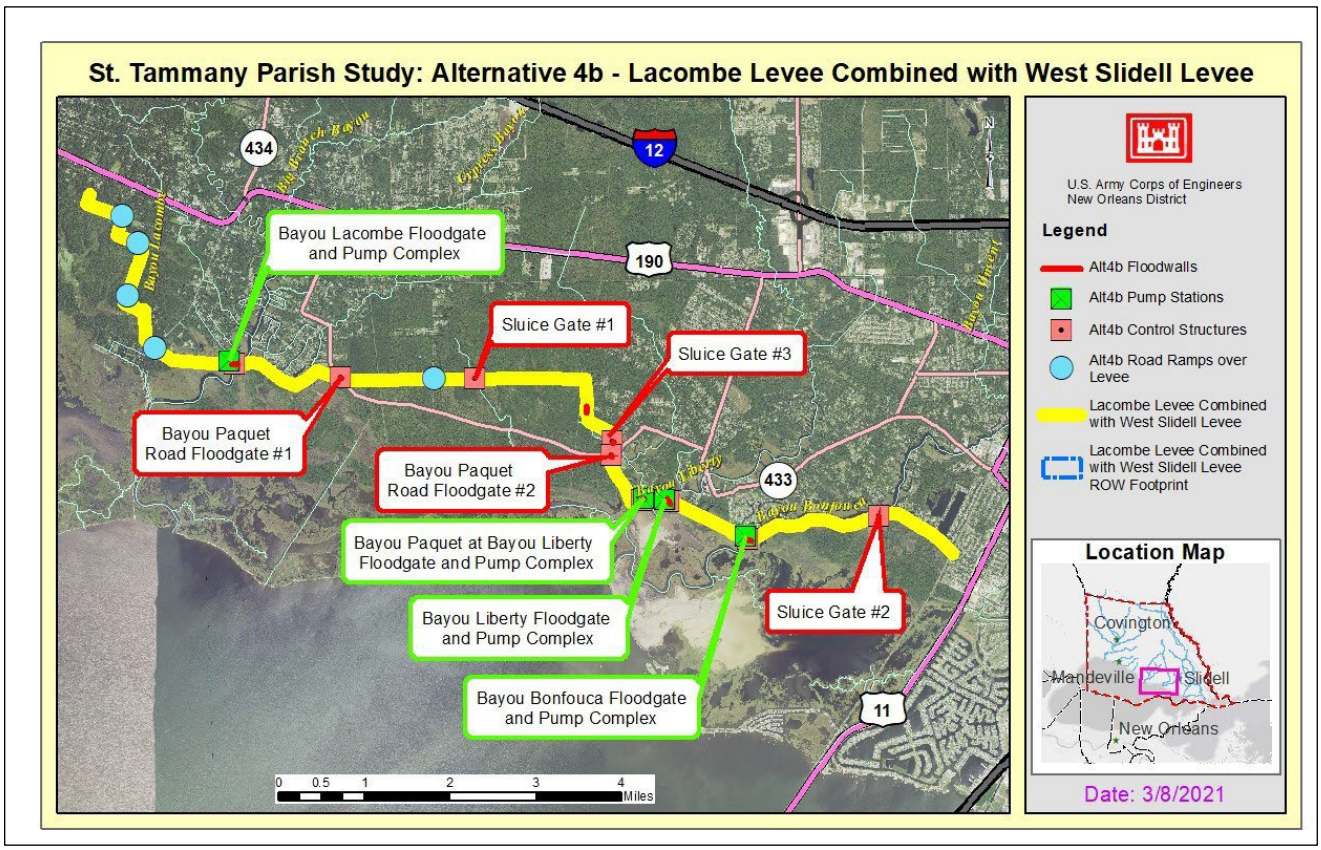


Figure D:1-4. Alternative 4b Lacombe Levee Combined with West Slidell Levee

1.3.3.3.1 Description of Alignment

The new levee would be continuous and would start on the unincorporated community of Lacombe, Louisiana on the south side of U.S. Highway 190 on Shelby Drive. Then it would continue east and cross Bayou Paquet at two locations. The levee would

continue east and cross Bayou Liberty, and Bayou Bonfouca, along the northern perimeter of the Big Branch Marsh National Wildlife Refuge (BBMNWR). The levee would terminate on the west side of the Norfolk Southern Railway Corp. railroad tracks (west of U.S. Highway 11 in the vicinity of Dellwood Pump Station) in Slidell.

1.3.3.3.2 Levee Design Section and Borrow Quantities

Alternative 4b levee elevations would vary depending on the levee location. In the Lacombe area, the elevation is 12.5 feet, except in the vicinity of Bayou Lacombe, which is 14.5 feet. The levee in the area between Lacombe and West Slidell is 13 feet. The elevation of the West Slidell portion varies between 13 feet and 17 feet depending on the location. The existing ground elevations and corresponding quantity calculations are based on the most current terrain raster dataset. The preliminary PDT assumptions are that the new levee would have a 10-foot-wide levee crown and side slopes of 1V:3H. The construction of the levee alignment would impact 165 acres. This levee alignment would require 1,205,000 cubic yards of fill/borrow (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

1.3.3.3.3 Floodwall Elevation and Location

There is a 350 feet floodwall segment with top of wall elevation of 17 feet located approximately 3,500 feet northwest of Bayou Paquet Road floodgate. This floodwall is designed to fit the alignment between some properties at the western end of West Doucette Road and a utility corridor that is located west of those properties. The construction of this floodwall segment would impact approximately 0.4 acres.

1.3.3.3.4 Typical Floodwall Section

The typical T-wall section would consist of 3-foot-thick, 8.5-foot-wide slab with a 1.5-foot- thick stem. The height of the stem would vary. Preliminary assumptions are two rows of 1H:3V battered steel HP12 x 74 piles, 60-foot deep, spaced at 5-foot centers, and 30-foot- deep steel PZ sheet pile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The design of the T-wall, including the foundation, is subject to change once detailed geotechnical investigations are conducted. Refer to Figure D:1-5 for a typical T-Wall configuration.

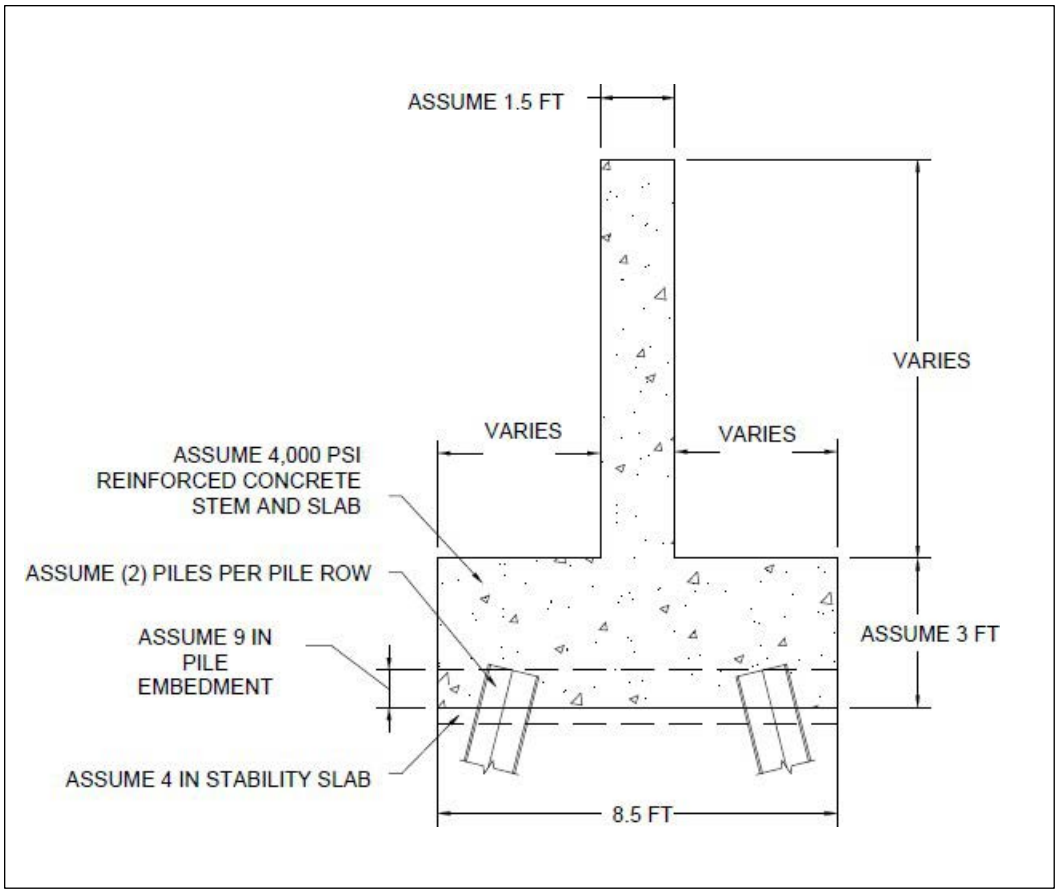


Figure D:1-5. Typical Floodwall Cross Section for all Alternatives with Floodwalls

1.3.3.3.5 Pump Stations and Floodgates

There are a series of pump station complexes (four with navigable floodgates) and sluicegates that are part of this alternative. The locations for the pump stations complexes and floodgates are listed in order starting on the west side of alternative 4b:

- Bayou Lacombe Pump Station (3,200 cfs and 300 foot long) complex. This complex includes a 20-foot navigable floodgate (12.6 acres of construction area)
- Sluicegate #1 would be located at Bayou Bonfouca West Tributary Number 1 (0.25 acres of construction area)
- Sluicegate #3 would be located at an upstream tributary of Bayou Paquet (0.25 acres of construction area). There is no pump station at this location.
- Bayou Paquet at Bayou Liberty Pump Station complex (500 cfs and 400 foot long) (12.6 acres of construction area). The complex includes a 20-foot navigable floodgate.

- Bayou Liberty Pump Station complex (3,200 cfs and 400 foot long). The complex includes a 20-foot navigable floodgate (12.6 acres of construction area)
- Bayou Bonfouca Pump Station complex (3,700 cfs and 300 foot long). The complex includes a 20-foot navigable floodgate (12.6 acres of construction area)
- and Sluicegate #2 would be located at a crossing of Bayou Bonfouca with an unamend waterway where the bayou turns north (0.25 acres of construction area).

1.3.3.3.6 Vehicular Floodgates and Ramps

Alternative 4b includes a series of vehicular ramps where the roads cross the levee alignment to provide vehicular access. There are five road ramps and two vehicular floodgates.

Features are listed from west to east:

Pontchartrain Drive, Barringer Road #1, Barringer Road #2, and Lake Road/LA Highway 434. East of Bayou Lacombe Pump Station and floodgate, there is a vehicular floodgate at Bayou Paquet Road (west), and a ramp at Transmitter Road. After Sluicegate # 1, and Bayou Paquet Pump Station and floodgate, there is a 30-foot vehicular floodgate at Bayou Paquet Road (east).

1.3.3.3.7 Structural Assumptions for Alternative 4b

1.) Pump Stations:

a. Bayou Lacombe Pump Station
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
b. Bayou Paquet at Bayou Liberty Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
c. Bayou Liberty Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
d. Bayou Bonfouca Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump

Station.

ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.

iii. Assumed integrated navigable floodgate for recreational vessels.

2.) Floodwalls:

a. 350 linear foot floodwall reach

i. Assumed T-wall for this alignment reach as the alignment is adjacent to private property.

ii. Assumed T-wall dimensions and configurations similar to USACE New Orleans to Venice Non-Federal West NOV-NF-W-06B.5 Magnolia Pump Station T-walls.

iii. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.

iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.

v. Assumed permanent access to be included in construction area.

3.) Access Floodgates:

a. Paquet Road West floodgate

i. Assumed roller floodgate.

ii. Assumed rough ground elevation via Google Earth.

iii. Assumed floodgate width based on road width.

b. Paquet Road East floodgate

i. Assume roller floodgate.

ii. Assumed rough ground elevation via Google Earth.

iii. Assumed floodgate width based on road width.

4.) Control Floodgates:

a. Sluicegates #1, #2 and #3

i. Assumed sluicegate.

ii. Assumed width based on stream width.

iii. Assumed sill is 5 feet below ground elevation.

1.3.4 Alternative 5: Bayou Liberty/ Bayou Vincent/ Bayou Bonfouca

1.3.4.1 West Slidell Levee

The West Slidell levee consists of a combination of 6.5 miles of levee and floodwall alignment to reduce flooding. This alignment is a combination of approximately 6.5 miles (34,000 feet) of levees and 0.08 miles (450 feet) of floodwall. This alternative also consists of floodwalls, floodgates, sluicegates, vehicular floodgates, ramps, detention pond with weir, and channel improvements. See Figures D:1-6; D:1-7; D:1-8; D:1-9; and D:1-10.

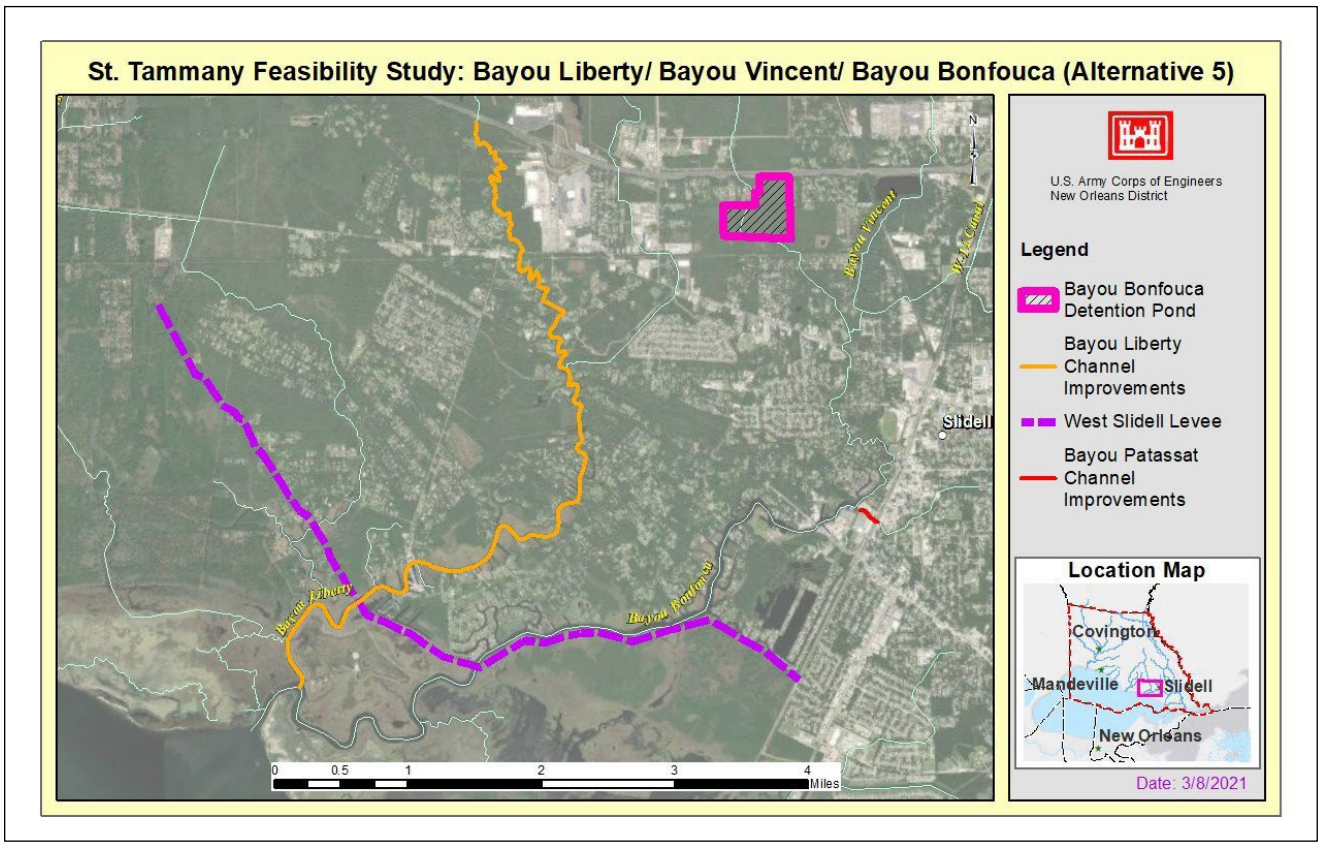


Figure D:1-6. Alternative 5 Bayou Liberty/ Bayou Vincent/ Bayou Bonfouca

1.3.4.1.1 Description of Alignment

This alignment is located on the west side of the City of Slidell, Louisiana. The levee extends from south of Highway 190 on the southwest of South Tranquility Road, crosses Bayou Paquet, continuing past Bayou Liberty, and extending to Bayou Bonfouca. The levee would continue east along the northern perimeter on the Big Branch Marsh NWR and would terminate west of the Norfolk Southern Railway Corporation railroad tracks (west of U.S. Highway 11 in the vicinity of Dellwood Pump Station) in Slidell.

1.3.4.1.2 Levee Design Section and Borrow Quantities

The elevation of Alternative 5 varies between 13 feet and 17 feet depending on the location. Existing ground elevation was obtained from the most current terrain raster dataset. The preliminary assumptions are that the levee would have a 10-foot-wide levee crown and side slopes of 1V:3H. The construction of this levee alignment would impact 78 acres. This levee alignment would require 611,000 cubic yards of fill/borrow (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

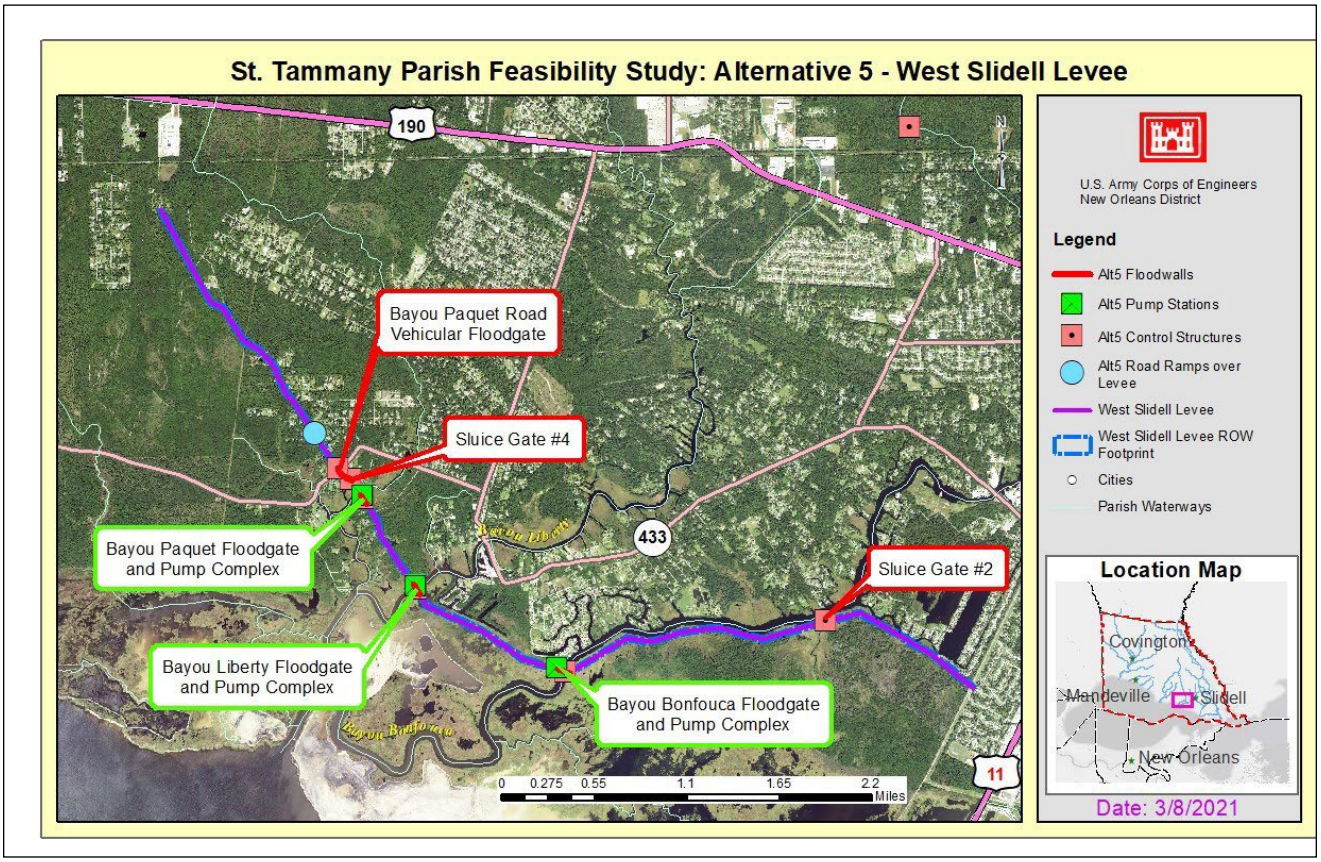


Figure D:1-7. Alternative 5 West Slidell Levee Focus

1.3.4.1.3 Floodwall Elevation and Location

There is one floodwall segment of approximately 0.08 mile (450 feet) in length within the Alternative 5 levee alignment. This segment would be located approximately 3,500 feet northwest of the Bayou Paquet Road floodgate. The floodwall would be at elevation 13 feet (total construction area for these floodwall segments would be one acre).

1.3.4.1.4 Typical Floodwall Section

The typical T-wall section consists of a 3-foot thick, 8.5-foot-wide slab with a 1.5-foot-thick stem. The height of the stem varies. Preliminary assumptions are two rows of 1H:3V battered steel H12 x 74 piles, 60-foot deep, spaced at 5 feet centers, and 30-foot-deep steel PZ sheetpile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The T-wall design, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.4.1.5 Pump Stations and Floodgates

There are three pump stations, three floodgates, and two sluicegates that are part of alternative 5, starting from the west:

- Sluicegate #4 is a 25-foot sluicegate serving a tributary of Bayou Paquet between Bayou Paquet Road and Jummonville Road (0.25 acres of construction area)
- Bayou Paquet Pump Station complex (500 cfs and 100 foot long) located between Jummonville Road and Mayer Drive. The complex includes a 20-foot navigable floodgate (construction area is 12.6 acres)
- Bayou Liberty Pump Station complex (3,200 cfs and 400 foot long). The complex includes a 20-foot navigable floodgate (construction area would be 12.6 acres)
- Bayou Bonfouca Pump Station complex (3,700 cfs and 300 foot long). The complex includes a 20-foot navigable floodgate (12.6 acres required for construction)
- Sluicegate #2 would be located at a crossing of Bayou Bonfouca with an unamend waterway where the bayou turns north. (0.25 acres of construction area). Note that Sluicegate #4 would not be at the same location as Sluicegate #3 in Alternative 4b.

1.3.4.1.6 Vehicular Floodgates and Ramps

Alternative 5 includes a vehicular road ramp at Cousins Road and a 30-foot vehicular floodgate at Bayou Paquet Road.

1.3.4.1.7 Structural Assumptions for Alternative 5

1.) Pump Stations:

a. Bayou Paquet at Bayou Liberty Pump Station
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.

b. Bayou Liberty Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.

c. Bayou Bonfouca Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.

2.) Floodwalls:
   a. 450-foot floodwall reach
      i. Assumed T-wall for this alignment reach as the alignment is adjacent to private property.
      ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W- 06B.5 Magnolia Pump Station T-walls.
      iii. Assumed 60 foot 1:3 battered H-piles based on geotechnical analysis.
      iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
      v. Assumed permanent access to be included in construction area.
3.) Access Floodgates:
   a. Paquet Road East floodgate
      i. Assumed floodgate.
      ii. Assumed rough ground elevation via Google Earth.
      iii. Assumed floodgate width based on road width.
4.) Control Floodgates:
   a. Sluicegates #2 and #4
      i. Assumed sluicegate.
      ii. Assumed width based on stream width.
      iii. Assumed sill is 5 feet below ground elevation.
   b. Bayou Bonfouca Detention Pond Weir:
      i. Assumed weir.
      ii. Assumed 100-foot long.
      iii. Assumed top of weir at 5 feet above ground level.

1.3.4.2 Bayou Bonfouca Detention Pond

Alternative 5 includes the construction of the Bayou Bonfouca Detention Pond to address rainfall and riverine flooding. This detention pond would be located south of the Interstate 12 (I-12). The construction of the detention pond would impact 109 acres and have a water detention capacity of 1,308 acre-feet. It is assumed that there is an existing average elevation of 12 feet NAVD 88. The depth of the pond would be 12 feet with 1V:3H side slopes. Approximately 125 acres would have to be cleared and grubbed prior to excavation. Approximately 2,500,000 cubic yards of excavated material is assumed. A 65-foot temporary right-of-way (ROW) (16 AC) would be needed around the perimeter for access during construction. The detention pond also includes the construction of a weir. Refer to Figure D:1-8.

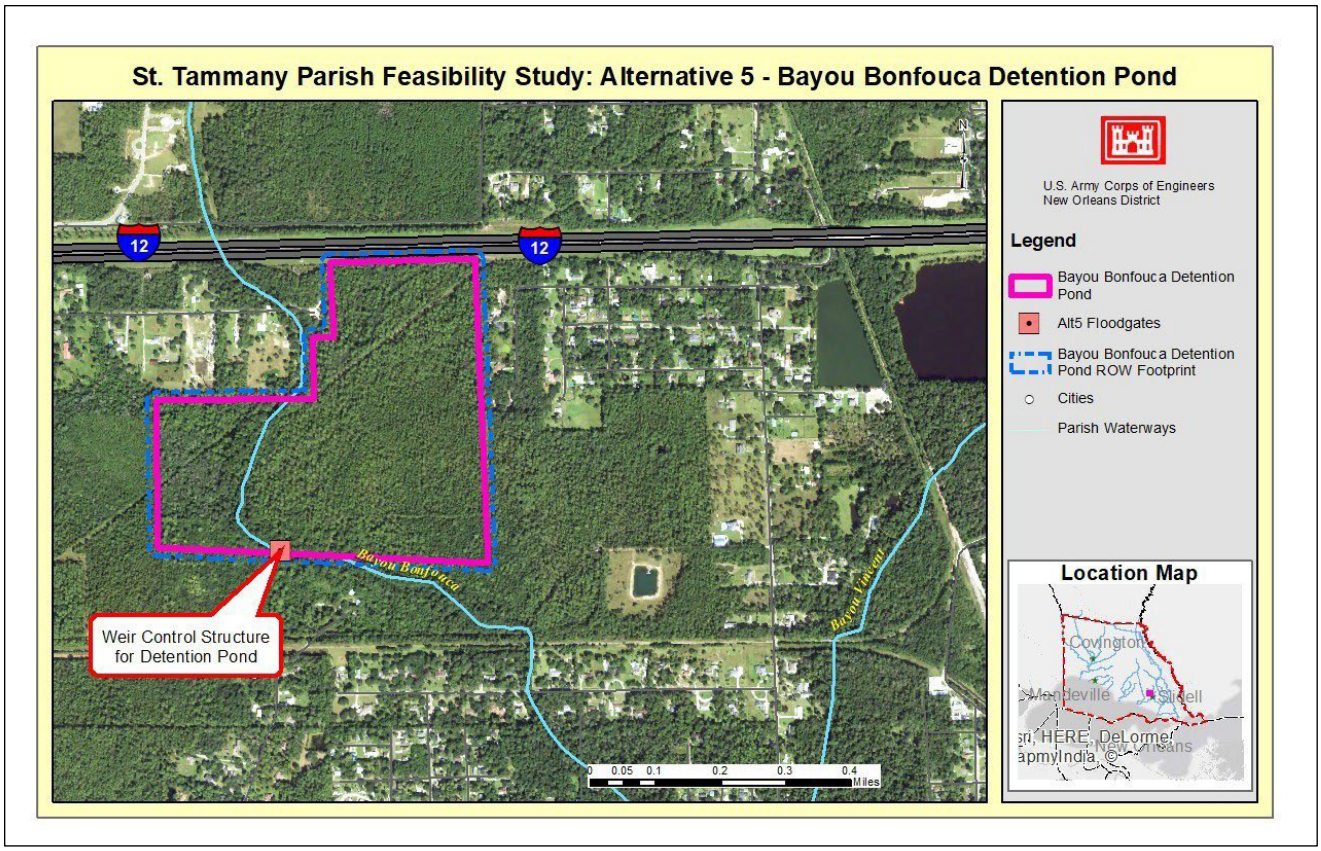


Figure D:1-8. Alternative 5 Bayou Bonfouca Detention Pond Focus

1.3.4.3 Bayou Patassat Channel Improvements

The Bayou Patassat channel improvements (Figure D:1-9) would be performed between Bayou Vincent Pump Station and U.S. Highway 11. Bayou Patassat is a small tributary of Bayou Bonfouca. The preliminary design of the Bayou Patassat channel improvements assumes that an existing bank elevation of 1 foot, a 10-ft bottom width at elevation (-) 5 feet NAVD 88, with bank side slopes of 1V:3H. The work would be located in Bayou Patassat between Bayou Vincent Pump Station and U.S. Highway 11. Land access to the site would be through Bayou Lane or the existing Bayou Vincent pump station.

The lands required for implementation of the Bayou Patassat channel improvements are all public property and owned by either St. Tammany Parish or the City of Slidell, LA. Possible staging areas include the city-owned land around the bayou and pump station or at the grassy area at the end of Bayou Lane. Access to the bayou is assumed via city-owned property along the channel. There would be enough ROW for two-way access on the north of the channel. If necessary, a temporary culvert can be placed in the channel to allow for crossing to the southernmost bank.

Approximately 0.17 miles (900 ft) of clearing and snagging would occur in Bayou Patassat. Material removed may include trees, debris, trash, or other obstructions within the channel. For Bayou Patassat channel improvements, approximately 2 acres of ROW would be required for a temporary easement within the channel. Approximate 0.6 acres of additional ROW would be tree-clearing, with the majority of the work taking place on the southernmost bank. All trees and debris cleared is anticipated to be chipped on site and then hauled to the nearest landfill. The nearest landfills are the Slidell Landfill (east of Interstate 10 and south of LA Highway 433) and Waste Management (2685 Gause Boulevard West, Slidell, LA 70460). The assumed haul distance is 15 miles.

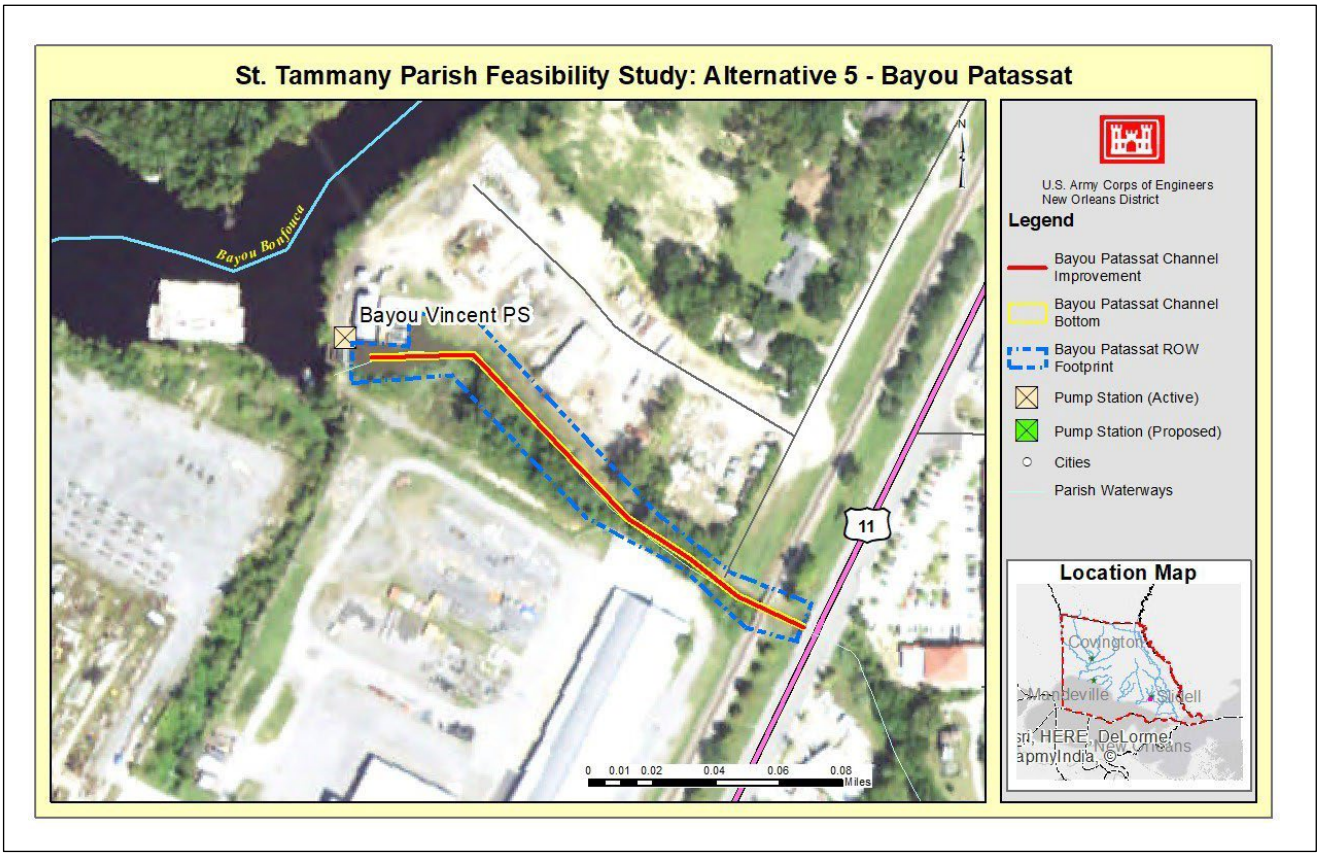


Figure D:1-9. Alternative 5 Bayou Patassat Focus

1.3.4.4 Bayou Liberty Channel Improvements

Alternative 5 includes the Bayou Liberty channel improvements (Figure D:1-10) to address rainfall and riverine flooding. The channel improvements run north-south, starting immediately south of the I-12, crossing U.S. Highway 190, the bridge that crosses the Tammany Trace, and LA Highway 433, and ending at the confluence with Bayou Bonfouca in the proximity of Lake Pontchartrain. The channel improvements

include clearing and snagging of 8 miles (41,232 feet) of the channel. The preliminary design of the channel improvements assumes an existing bank elevation of 1 foot, a 10-foot bottom width at elevation (-)5 ft NAVD 88, and bank side slopes of 1V:3H. Material removed may include trees, debris, or other obstructions within the waterway.

All trees and debris cleared would likely be chipped on site and then hauled to the nearest landfill. The nearest landfills are the Slidell Landfill (east of Interstate 10 and south of LA Highway 433) and Waste Management (2685 Gause Boulevard West, Slidell, LA 70460). The assumed haul distance is 15 miles.

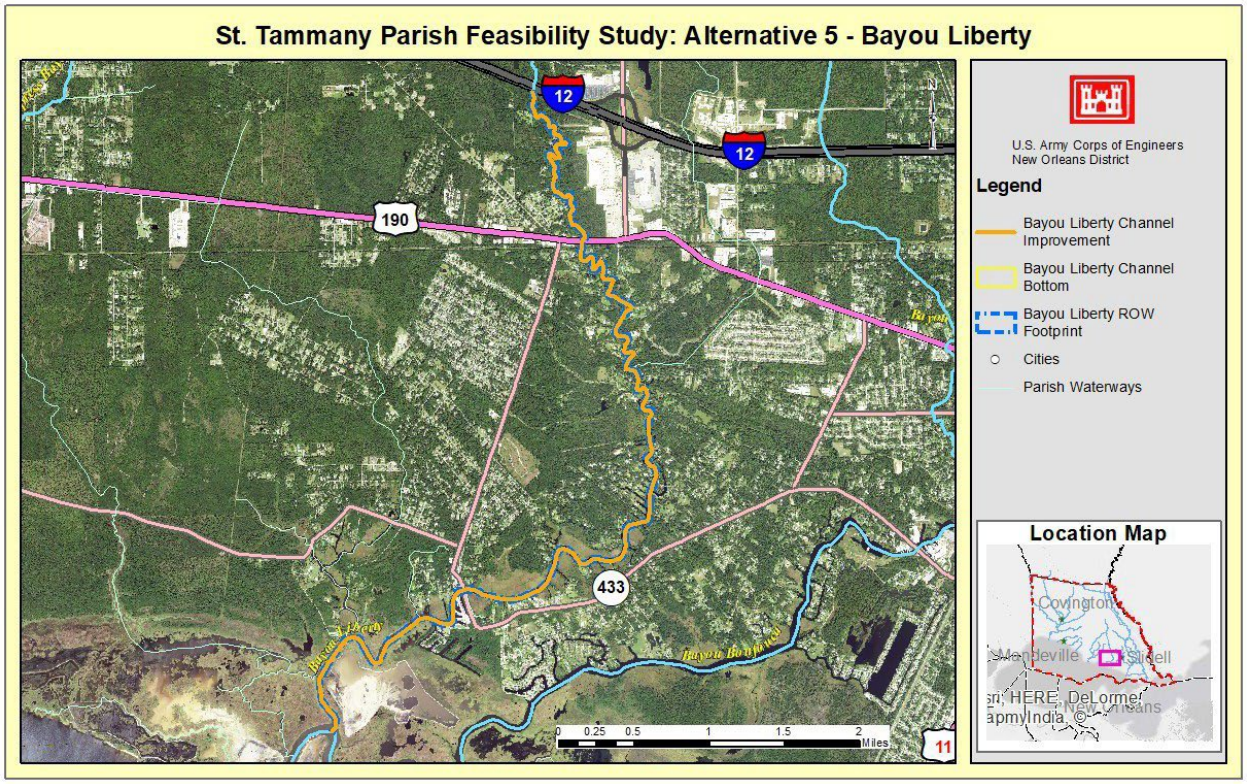


Figure D:1-10. Alternative 5 Bayou Liberty Focus

Due to the length of Bayou Liberty, the work was divided into four reaches. The first two reaches would be completed via the top of bank and the last two would be completed via floating plant.

1.3.4.4.1 Reach 1: I-12 to US Highway 190 (8,050 feet)

Access to Reach 1 would be from Frontage Road. A one-acre access corridor would be from Frontage Road to the bayou along an existing opening in the woods. Approximately 9 acres of clearing and snagging would occur from top-of-bank to top-of bank within the channel. If necessary, a temporary culvert could be placed in the channel to allow for crossing over to the opposite bank. Material removed may include trees, debris, trash, or other obstructions within the waterway.

1.3.4.4.2 Reach 2: US Highway 190 to the Tammany Trace (200 feet)

Access to Reach 2 would be from the trailhead off U.S. Highway 190. A 25-foot-wide access corridor (0.2 AC) would be located on the right descending bank side south of where the bayou intersects the highway. Clearing would be needed for this access corridor. An additional access corridor would be located along both sides of the channel, offset 25 feet from each top of bank from the intersection of U.S. Highway 190 to the bridge that crosses Tammany Trace. The combined acreage for access on both sides would be approximately 0.3 acres. This would also need to be cleared to be used for access. Approximately 0.2 acres of clearing and snagging would occur from top-of-bank to top-of bank within the channel. If necessary, a temporary culvert can be placed in the channel to allow for crossing to the opposite bank. Material removed may include trees, debris, trash, or other obstructions within the waterway.

1.3.4.4.3 Reach 3: Tammany Trace Bridge to the LA Highway 433 (22,726 feet)

Due to the boathouses along the bank of the bayou, access for construction would not work from the bank. All work would be done from the water (i.e., using a floating plant). This reach would be primarily snagging operations. There are two access points to reach 3. Access 1 (0.3 AC) would be via Elks Road. There is a boat launch and a staging area (0.4 AC). No clearing would be needed for this access corridor or staging area. Access 2 (0.15 AC) would be via Jefferson Avenue along with an accompanying staging area (0.15 AC) and boat launch. No clearing would be needed for this access corridor or staging area. Approximately 24 acres of clearing and snagging would occur from top-of-bank to top-of bank within the channel, although the primary operation would be snagging. Material removed may include trees, debris, trash, or other obstructions within the waterway.

1.3.4.4.4 Reach 4: LA Highway 433 to the Mouth (10,065 feet)

Due to the boathouses along the bank of Bayou Liberty, access for construction would not work from the bank. All work would be done from the water (i.e., using a floating plant). This reach would be primarily snagging operations. Access (0.05 AC) would be via Rivet Drive. There is a boat launch and a staging area (0.3 AC). No clearing would be needed for this access corridor or staging area. Approximately 11 acres of clearing and snagging would occur from top-of-bank to top-of bank within the Bayou Liberty channel, although the primary operation would be snagging. Material removed may include trees, debris, trash, or other obstructions within the waterway.

Assumptions for channel improvements include a ROW measured 65 feet from the centerline to each side of the channel as a general guideline (total width of 130 feet), which includes space for equipment access. All work would be within the project footprint. The temporary work easement would be within the ROW. The material requiring disposal would be trucked away from the site. Assumption is that all access would be through public lands.

1.3.5 Alternative 6: South Slidell

Alternative 6 consists of a combination of levees, floodwalls, and pump stations. This alternative also consists of floodgates, vehicular floodgates and ramps.

There are three existing ring levees in the City of Slidell: (1) the King's Point ring levee that consists of two ring levees on the northeast side of the City of Slidell, (2) the Lakeshore Estates ring levee on the southeast side of the City of Slidell, and (3) the Oak Harbor ring levee in the vicinity of the Eden Isle community (Eden Isle). The proposed levee and floodwall independent alternative alignments under Alternative 6 (Alternatives 6a, 6b, and 6c) tie into some of these existing ring levees as described herein. Existing levees are represented in yellow in Figure D:1-11.

There are three independent alternative levee and floodwall alignments in Alternative 6. These three alternatives (Alternatives 6a, 6b, and 6c) are stand-alone alternatives and cannot be combined with one another but can be combined with other justified measures in the Final Array.

Alternative 6a consists of the South Slidell levee alignment. Alternative 6b consists of a combination of the South Slidell levee alignment from Alternative 6a and the Eden Isle floodwall alignments. Alternative 6c consists of a combination of portions of levee from the proposed Alternative 5 (except for the western portion of alignment) and the South Slidell levee alignment proposed in Alternative 6a (except for the northwestern portion of alignment). The two alignments would be connected with a new railroad floodgate across the existing Norfolk Southern Railway Corp. railroad tracks.

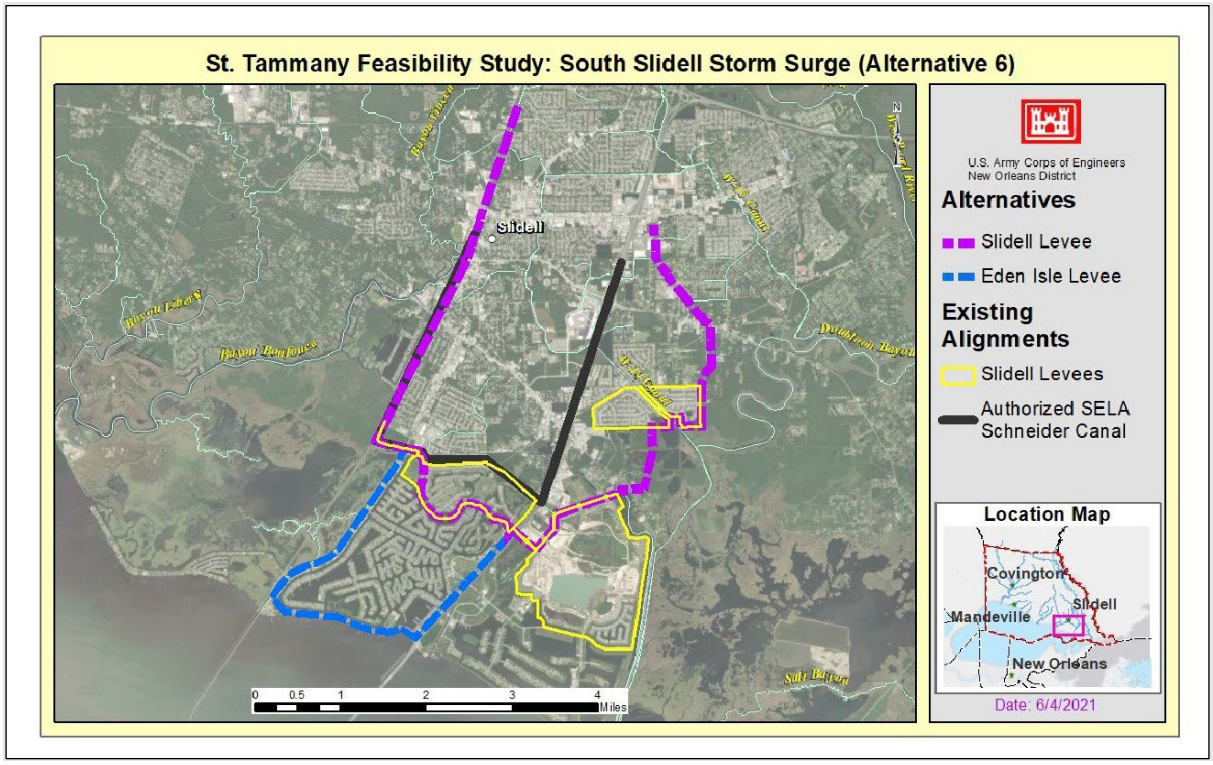


Figure D:1-11. Alternative 6 South Slidell

1.3.5.1 Alternative 6a: South Slidell

This alternative consists of 13 miles of alignment with a combination of approximately 7.3 miles of levees (38,500 feet) and approximately 5.9 miles (30,000 feet) of floodwall located in the City of Slidell, Louisiana. This alignment does not include Eden Isle. This alternative also consists of pump stations, floodgates, vehicular floodgates, and ramps. Refer to Figure D:1-12.

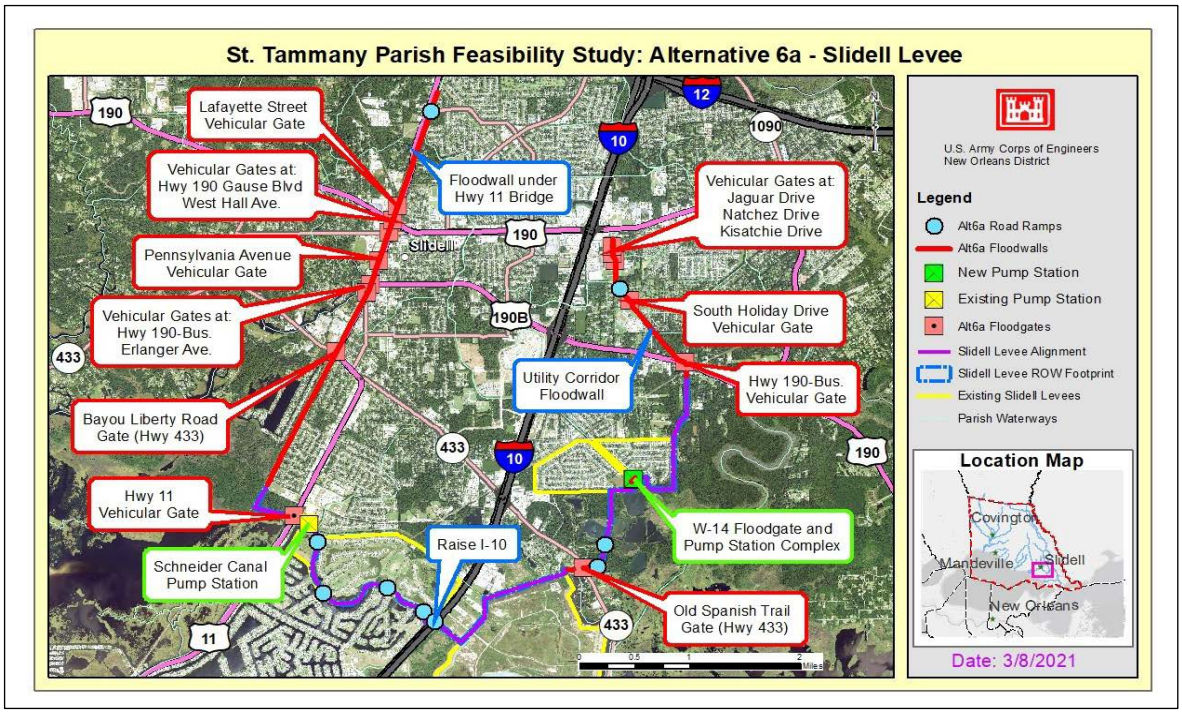


Figure D:1-12. Alternative 6a Slidell Levee

1.3.5.1.1 Description of Alignment

Starting on the northwest, the alternative consists of the construction of a floodwall extending on the east side of the Norfolk Southern Railway Corp. railroad tracks from Pinewood Country Club in a north to south direction. The floodwall would transition into a levee just south of First Baptist Church Christian School. Then the levee turns east and then south. This reach consists of a levee with floodwall segments. This reach includes the new Schneider Canal Pump Station (assume the same footprint as the existing facility). The new levee would tie to a segment of the existing Oak Harbor levee along Oak Harbor Boulevard (existing levee would be raised to elevation 15 feet), and then the I-10 would be raised to ramp over the new levee section. The new levee would tie to a section of the northern perimeter of the existing Lakeshore Estates levee (existing levee would be raised to Elevation 15 feet). The new levee would cross LA Highway 433 and then tie to a section of the existing King's Point west levee (existing levee would be raised to Elevation 15 feet).

The levee would tie to the new pump station at the W-14 canal and then tie to the existing King's Point east ring levee (existing levee would be raised to Elevation 15 feet). The new levee would turn north toward U.S. Highway 190 Business (Fremaux Avenue). The new levee would cross U.S. Highway 190 Business (Fremaux Avenue), where it would tie to a new floodwall across U.S. Highway 190 Business. The floodwall would run on the west side of the existing CLECO Corporate Holdings, LLC utility

corridor and cross South and North Holiday Drives. The floodwall would exit the CLECO Corporate Holdings, LLC utility corridor to continue on the east side of Carol Drive, continue north on the east side of Yaupon Drive, and terminate at Manzella Drive (one block south of Gause Boulevard).

CLECO Corporate Holdings, LLC has ROW use requirements pertaining to USACE work around their existing utility lines on the northeast corner of the floodwall alignment that would have to be met to provide clearance for construction activities (i.e. pile driving).

1.3.5.1.2 Interstate 10 Elevation

The I-10 would be raised to ramp over the new levee section by constructing ramps to the preliminary design elevation of 15 feet. The existing elevation of the I-10 at the proposed location is approximately 12.8 feet as per terrain raster dataset. This proposed location is the highest elevation of the I-10 in the vicinity of the proposed alignment. The I-10 elevation is lower (approximately 10 feet) on the adjacent areas.

1.3.5.1.3 Levee Design Section and Borrow Quantities

The elevation of the new South Slidell levee varies between 13 feet NAVD 88 and 15 feet NAVD 88 depending on the location. The elevations of the existing ground were used as per the terrain raster dataset. The preliminary assumptions are that the levee has a 10-foot-wide levee crown and side slopes of 1V:3H. The construction of the levee alignment would impact 88 acres. This levee alignment would require 851,000 cubic yards of fill/borrow (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

1.3.5.1.4 Floodwall Elevation and Location

The elevation for the floodwall segments would vary from 13.5 feet to 15 feet NAVD 88. The locations of the floodwall segments are:

There are approximately 4.1 miles (21,750 feet) of floodwall segment from Pinewood Country Club to just south of First Baptist Church Christian School (16 acres of construction area).

There is a 0.06 mile (300 feet) Old Spanish Trail floodwall segment (0.3 acres of construction area).

Across from LA Highway 433, there is a 0.09 mile (450 feet) Old Spanish Trail floodwall segment by Espirit du Lac Street (construction area is 0.5 acres).

There is 0.04-mile (200 feet) floodwall segment near Belaire Drive (0.2 acres of construction area).

The next floodwall segment is on the north side along U.S. Highway 190 Business (Fremaux Avenue) for 0.08 miles (430 feet) and then the floodwall turns into the CLECO Corporate Holdings, LLC utility corridor for approximately 1.4 miles (7,200 feet) before terminating at Manzella Drive. The total length of this floodwall alignment is 1.5 miles (7,700 feet) (total construction area is 9 acres).

1.3.5.1.5 Typical Floodwall Section

The typical T-wall section consists of a 3-foot thick by 8.5-foot-wide slab with a 1.5-foot-thick stem. The height of the stem varies. Preliminary assumptions are two rows of 1H:3V battered HP12 x 74 piles, 60-foot deep, spaced at 5-foot centers, and 30-foot-deep steel PZ sheet pile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The design of the T-wall, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.5.1.6 Pump Stations and Floodgates

The structural components for the Slidell Alternative 6a include, starting from the northwest:

- 1,200 cfs and 150-foot-long Schneider Canal Pump Station complex with a 30-foot floodgate (construction area is 12.6 acres)
- 1,200 cfs and 150-foot-long W-14 Pump Station complex with a 30-foot floodgate (construction area is 12.6 acres).

1.3.5.1.7 Vehicular Floodgates and Ramps

There would be eight vehicular ramps over the levee, 14 vehicular floodgates to provide access through the levee, and the I-10 roadway would be raised to ramp over the new levee section. Starting from the northwest:

Ramp on North Avenue

The following vehicular floodgates would be constructed:

- 50-foot, Lafayette Street,
- 75-foot, U.S. Highway 190 (Gause Boulevard),
- 40-foot, West Hall Avenue,
- 30-foot, West Pennsylvania Avenue,
- 30-foot, U.S. Highway 190 Business (Fremaux Ave),
- 30-foot, Erlanger Avenue,
- 40-foot, Bayou Liberty Road (LA Highway 433) west side crossing,
- 75-foot, Pontchartrain Drive (U.S. Highway 11). There would be ramps at:
- Cypress Lakes Drive (levee alignment).
- Mariner’s Cove Boulevard,

- Oak Harbor Country Club entrance,
- Grand Champions Lane.

The I-10 roadway would be raised to ramp over the new levee section.

There would be a 30-foot vehicular floodgate at LA Highway 433 east (Old Spanish Trail). There would be ramps at Fleur Du Lac Street and at Nunez Road. There would vehicular floodgates in the following roads:

- 50-foot, U.S. Highway 190 Business (east)
- 20-foot, South Holiday Drive

There would be a ramp at North Holiday Drive

- 20-foot, Jaguar Drive
- 20-foot, Natchez Drive
- 20-foot, Kisatchie Drive.

It is assumed that on the northwest side of the alignment, there would be no need for a vehicular floodgate at North Boulevard. On the northeast side of the alignment, it is assumed that there would be no need for a vehicular floodgate on North Holiday Drive.

1.3.5.1.8 Structural Assumptions for Alternative 6a and Alternative 6b

1.) Pump Stations:
   a. Schneider Canal Pump Station
      i. Assumed 1,200 cfs, based on previous feasibility studies.
      ii. Assumed complete redesign of all components.
      iii. Assumed navigable floodgate for recreational vessels.
      iv. Assumed same configuration as the USACE WSLP Pump Station.
   b. W-14 Pump Station:
      i. Assumed 1,200 cfs, based on hydraulic estimates.
      ii. Assumed all new construction.
      iii. Assumed navigable floodgate for recreational vessels.
      iv. Assumed same configuration as the USACE WSLP Pump Station.
2.) Floodwalls:
   a. 21,750 linear foot floodwall reach along Slidell railroad tracks
      i. Assumed alignment starts between the railroad tracks and Pinewood Country Club and ends behind First Baptist Church Christian School recreational area.
      ii. Assumed T-wall for this alignment reach as the alignment is adjacent to the railroad tracks and there is not enough room for levee.
      iii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
      iv. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.

Assumed 50-foot T-wall construction area to side opposite the railroad tracks

v. Assumed permanent access to be included in construction area.

b. 300 linear foot floodwall reach near Old Spanish Trail:
   i. Assumed T-wall for this alignment reach as the alignment is adjacent to a private warehouse near Old Spanish Trail
   ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
   iii. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.
   iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
   v. Assumed permanent access to be included in construction area.

c. 200 linear foot floodwall south of U.S. Highway 190 Business:
   i. Assumed T-wall for this alignment reach as the alignment is between an energy substation and private property near U.S. Highway 190 Business.
   ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
   iii. Assumed 60-foot 1:3 battered H-piles based on geotechnical analysis.
   iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
   v. Assumed permanent access to be included in construction area.

d. 7,700 linear foot floodwall reach along power easement north of U.S. Highway 190 Business:
   i. Assumed T-wall for this alignment reach is between U.S. Highway 190 Business and U.S. Highway 190 (Gause Boulevard).
   ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
   iii. Assumed 60-foot 1:3 battered H-piles based on geotechnical analysis.
   iv. Assumed T-wall construction area is 50 feet to either side from center line of alignment.
   v. Assumed permanent access to be included in construction area.

3.) Access Gates:

a. Vehicular Roller floodgates
   i. Assumed all Alternative 6a vehicular floodgates to be roller floodgates.
   ii. Assumed sill to be at existing ground level.

1.3.5.2 Alternative 6b: South Slidell with Eden Isle

Alternative 6b consists of the Slidell levee and floodwall system and incorporates an Eden Isle floodwall. This alternative would reduce the risk of storm surge to Slidell, including Eden Isle. This alternative consists of 17.1 miles of alignment with a combination of levee and floodwall. The alignment would have 5.2 miles of levees

(27,400 feet). The alignment has approximately 6 miles (31,000 feet) of floodwall at Eden Isle and 5.9 miles (30,000 feet) of floodwall in the Slidell levee alignment. The floodwall alignment totals 11.9 miles (61,000 feet). This alternative also consists of floodgates, navigable floodgate, vehicular floodgates and ramps. Refer to Figure D:1-13.

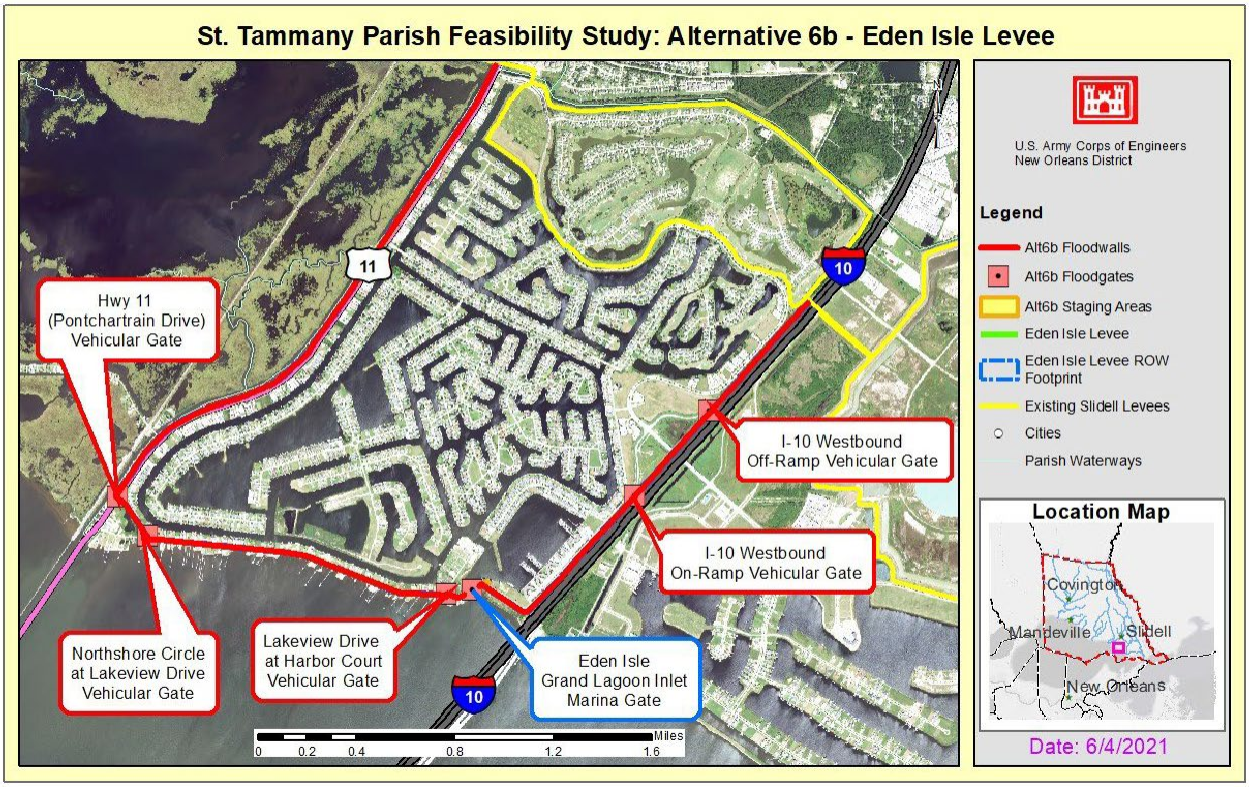


Figure D:1-13. Alternative 6b Eden Isle Levee

1.3.5.2.1 Description of Alignment

Starting on the northwest, the alternative consists of the construction of a floodwall on the east side of the Norfolk Southern Railway Corp. railroad tracks from Pinewood Country Club in a north to south direction. The floodwall would transition into a levee just south of First Baptist Church Christian School. The levee would then transition into floodwall at Eden Isle. There would be approximately 6 miles (31,000 feet) of floodwall. This floodwall would start on the west side of Oak Harbor Drive and follow along the west side of U.S. Highway 11, would turn southeast on Lakeview Drive and would cross Oak Harbor Marina, where it would continue parallel to the I-10. The I-10 roadway would be raised to the preliminary design elevation of 15 feet NAVD 88. The alignment would transition into a levee and tie to a section of the northern perimeter of the existing Lakeshore Estates levee. The new levee would cross LA Highway 433 and then tie to a section of the existing King's Point west levee (existing levee would be raised to elevation 15 feet). The alignment would connect to the new W-14 pump station at the

W-14 canal and then tie to the existing King's Point east levee (existing levee would be raised to elevation 15 feet). The new levee would turn north toward U.S. Highway 190 Business (Fremaux Ave). The new levee would cross U.S. Highway 190 Business (Fremaux Ave) would transition into a floodwall across U.S. Highway 190 Business. The floodwall would continue on the west side of the CLECO Corporate Holdings, LLC utility corridor and would cross South and North Holiday Drives. The floodwall would exit the utility corridor to continue on the east side of Carol Drive, would continue north on the east side of Yaupon Drive, and would terminate at Manzella Drive (one block south of Gause Boulevard).

CLECO Corporate Holdings, LLC has ROW use requirements pertaining to USACE work around their existing utility lines on the northeast corner of the floodwall alignment that would have to be met to provide clearance for construction activities (i.e., pile driving).

1.3.5.2.2 Interstate 10 Elevation

The I-10 would be raised to ramp over the new levee section by constructing ramps to the preliminary design elevation of 15 feet. The existing elevation of the I-10 at the proposed location is approximately 12.8 feet as per terrain raster dataset. This proposed location is the highest elevation of the I-10 in the vicinity of the proposed alignment. The I-10 elevation is lower (approximately 10 feet) on the adjacent areas. This feature would be designed in the feasibility level of design for the study. Refer to Section 10.4.4 for design of this feature.

1.3.5.2.3 Levee

For this alternative, a section of the new Slidell levee (alternative 6a) would not be constructed (11,000 feet of levee and 800 feet of floodwall). This section would be between U.S. Highway 11 (Pontchartrain Drive) on the west side and the I-10 on the east side. A significant part of the alignment that would not be constructed is part of the existing Oak Harbor levee.

1.3.5.2.4 Levee Design Section and Borrow Quantities

The elevation of the new Slidell levee for Alternative 6b varies between 13 feet NAVD 88 and 15 feet NAVD 88 depending on the location. The elevations of the existing ground were used as per the terrain raster dataset. The preliminary assumptions are that the levee would have a 10-foot-wide levee crown and side slopes of 1V:3H. The levee alignment would impact 63 acres of construction area. This levee alignment would require 742,000 cubic yards of fill (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

1.3.5.2.5 Floodwall Elevation and Location

For Eden Isle, the preliminary design elevation of the new floodwall varies from 13.5 feet to 21 feet NAVD 88 depending on the location. For the remaining portion of the Slidell levee and floodwall system, the new floodwall elevation would vary from 13.5 feet to 16.5 feet NAVD 88. For the Eden Isle portion, the alignment would consist entirely of new floodwall. Note that on Lakeview Drive, the floodwall would be located at the center of the road with one lane on either side of the floodwall to allow access to homes.

For the Slidell portion of the alignment, there would be approximately 4.1 miles (21,750 feet) of floodwall segment from Pinewood Country Club to just south of First Baptist Church Christian School (construction area is 16 acres). There would be a 0.06 mile (300 feet) Old Spanish Trail floodwall segment (construction area is 0.3 acres). Across from LA Highway 433, there would be a 0.09 mile (450 feet) Old Spanish Trail floodwall segment by Espirit du Lac Street (construction area is 0.5 acres). There would be a 0.04-mile (200 feet) floodwall segment near Belaire Drive (construction area would be 0.2 acres). The next floodwall segment would be on the north side along U.S. Highway 190 Business (Fremaux Ave) for

0.08 miles (430 feet) and then the floodwall would turn into the CLECO Corporate Holdings, LLC utility corridor for approximately 1.4 miles (7,200 feet) and would terminate at Manzella Drive. The total length of this floodwall alignment would be 1.5 miles (7,700 feet) long (total construction area would be 9 acres).

1.3.5.2.6 Typical Floodwall Section

The typical T-wall section consists of a 3-foot thick by 8.5-feet wide slab with a 1.5-foot-thick stem. The height of the stem would vary. Preliminary assumptions are two rows of 1H:3V battered HP12 x 74 piles, 60-foot deep, spaced at 5-foot centers, and 30-foot-deep steel PZ sheet pile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The design of the T-wall, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.5.2.7 Pump Stations and Floodgates

For Eden Isle:

- 100-foot-long Eden Isle (Grand Lagoon) Oak Harbor Marina navigable floodgate structure would be constructed (construction area would be 2 acres).

For the Slidell levee portion:

- 1,200 cfs and 150-foot-long Schneider Canal Pump Station complex with a 30-foot floodgate (construction area is 12.6 acres)
- 1,200 cfs and 150-foot-long W-14 Pump Station complex with a 30-foot floodgate (construction area is 12.6 acres).

1.3.5.2.8 Vehicular Floodgates and Ramps

For Eden Isle, there would be 5 vehicular floodgates for access in the floodwall locations starting from the northwest:

- 75-foot, Pontchartrain Drive (U.S. Highway 11)
- 50-foot, Northside Circle at Lakeview Drive vehicular floodgate
- 50-foot, Lakeview Drive (Harbor View Court) floodgate
- two 50-foot Interstate 10 access road floodgates (westbound on-ramp and I-10 off- ramp)

For the Slidell levee portion, there would be four vehicular ramps over the levee, 13 vehicular floodgates to provide access, and the Interstate 10 roadway would be raised to ramp over the new levee section. Starting from the northwest, there would be a North Avenue ramp.

The following vehicular floodgates would be constructed:

- 50-foot, Lafayette Street
- 75-foot, U.S. Highway 190 (Gause Boulevard)
- 40-foot, West Hall Avenue
- 30-foot, West Pennsylvania Avenue
- 30-foot, U.S. Highway 190 Business (Fremaux Ave)
- 30-foot, Erlanger Avenue
- 40-foot, Bayou Liberty Road (LA Highway 433) west side crossing.
- The I-10 would be raised to ramp over the new levee section.
- There will be a 30-foot vehicular floodgate at LA Highway 433 east (Old Spanish Trail)

There would be two vehicular ramps at:

- Fleur Du Lac Street
- Nunez Road

There would vehicular floodgates at:

- 50-foot, US Highway 190 Business (east)
- 20-foot, South Holiday Drive Ramp on North Holiday Drive

There would vehicular floodgates at:

- 20-foot, Natchez Drive
- 20-foot, Kisatchie Drive

It is assumed that on the northwest side of the alignment, there would be no need for a vehicular floodgate at North Boulevard. On the northeast side of the alignment, it is assumed that there would be no need for a vehicular floodgate on North Holiday Drive.

1.3.5.3 Alternative 6c: South Slidell Storm Surge with West Slidell

Alternative 6c consists of a combination of portions of the West Slidell levee alignment proposed in Alternative 5 and the South Slidell levee and floodwall system alignment proposed in Alternative 6a (except for the northwestern portion of that alignment) with the two alignments being connected by a new railroad gate across the existing Norfolk Southern Railway Corp. railroad tracks.

Alternative 6c consists of a total of 16.3 miles (85,900 feet) of a levee and floodwall alignment, with approximately 14 miles (73,700 feet) of levees constructed in two separate (non-continuous) segments, and 2.3 miles (12,200 feet) of two separate (non-continuous) segments of a floodwall. This alignment would include 49,100 feet of South Slidell segment and 36,800 feet of West Slidell segment of the Final Array of Alternatives. This Alternative also consists of pump stations, floodgates, vehicular floodgates, and ramps. See Figure D:1- 14.

1.3.5.3.1 Description of Alignment

Starting on the western side, the levee alignment would start on the south side of U.S. Highway 190 from southwest of Bayou Paquet, would cross Bayou Liberty, Bayou Bonfouca, along the northern perimeter on the Big Branch Marsh NWR and would meet the Norfolk Southern Railway Corp. railroad tracks west of U.S. Highway 11 in the vicinity of Dellwood Pump Station in Slidell. The alignment would cross the railroad tracks and continue into South Slidell. Then the alignment would transition into a floodwall along the east side of the railroad tracks from Dellwood Pump Station (Sun Valley Drive) in a north to south direction. The floodwall would turn into a levee just south of First Baptist Church Christian School.

Then the levee would turn east and then south. This reach would consist of a levee alignment with floodwall segments. This reach would include the new Schneider Canal Pump Station (assume the same footprint as the existing facility). Then the levee would tie to a section of the existing Oak Harbor levee along Oak Harbor Boulevard, and then the I-10 would be raised to ramp over the new levee section. The new levee would follow a section of the northern perimeter of the existing Lakeshore Estates levee. The new levee would cross LA Highway 433 and would tie to a section of the existing King’s Point west levee (existing levee would be raised to Elevation 15 feet). The alignment would tie to the new pump station at the W-14 canal and would tie to the existing King’s Point east levee (existing levee would be raised to Elevation 15 feet). The new levee would turn north toward U.S. Highway 190 Business (Fremaux Ave). The new levee would cross U.S. Highway 190 Business (Fremaux Ave) and would transition into a floodwall across U.S. Highway 190 Business. The floodwall would continue on the west side of the CLECO Corporate Holdings, LLC’s utility corridor and would cross South and North Holiday Drives. The floodwall would exit the utility corridor along the east side of Carol Drive and would continue north on the east side of Yaupon Drive, and ultimately terminating at Manzella Drive (one block south of Gause Boulevard).

1.3.5.3.2 Interstate 10 Elevation

The I-10 would be raised to ramp over the new levee section by constructing ramps to the preliminary design elevation of 15 feet. The existing elevation of the I-10 at the proposed location is approximately 12.8 feet as per terrain raster dataset. This proposed location is the highest elevation of the I-10 in the vicinity of the proposed alignment. The I-10 elevation is lower (approximately 10 feet) on the adjacent areas.

1.3.5.3.3 Levee Design Section and Borrow Quantities

Alternative 6c levee elevation would vary depending on location. Preliminary assumptions are that the new levee would have a 10-feet wide levee crown and side slopes of 1V:3H. The existing elevations were obtained from the terrain raster dataset. Elevation of the West Slidell portion would vary between 13 feet and 17 feet, depending on the location. Likewise, elevation of the South Slidell levee would vary between 13 feet and 15 feet depending on the location. Construction of the total levee alignment would impact approximately 169 acres. This levee alignment would require approximately 1,528,000 cubic yards of fill (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

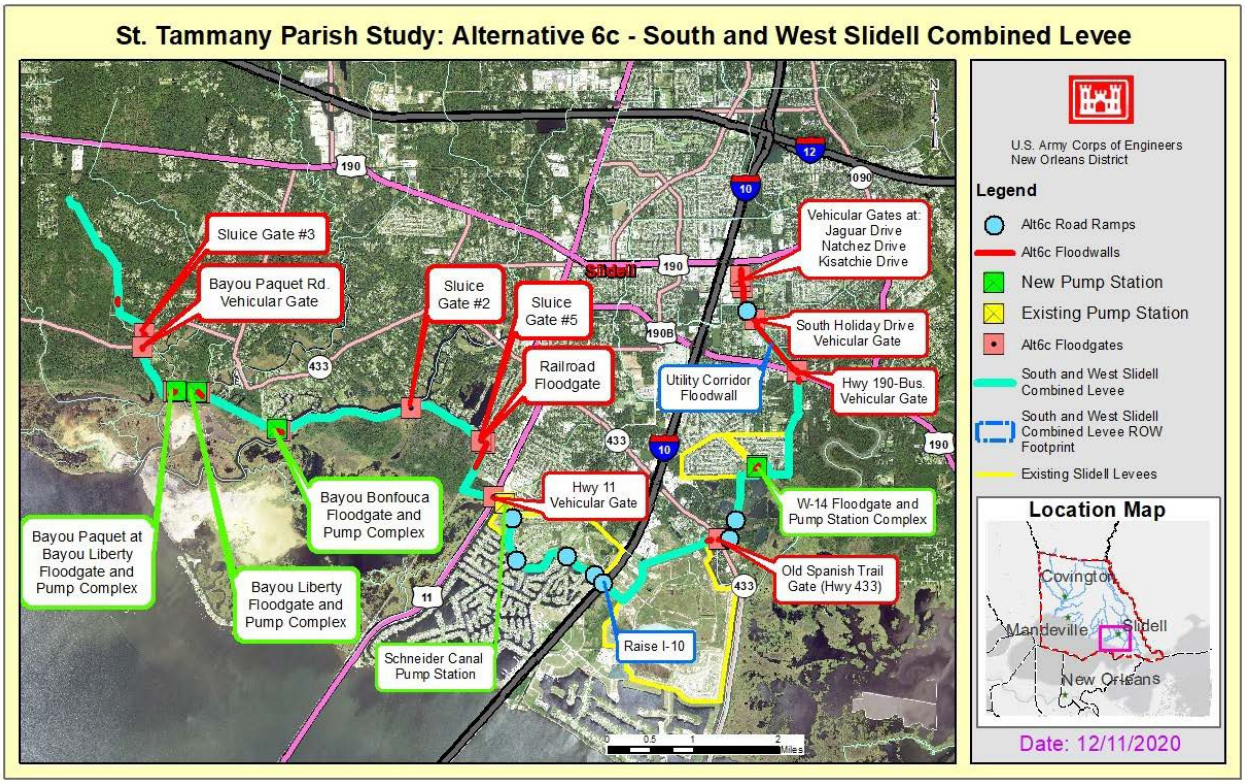


Figure D:1-14. Alternative 6c South and West Slidell Combined Levee

1.3.5.3.4 Floodwall Elevation and Location

For this alignment, elevation of floodwall segments would vary from 13.5 feet to 17 feet. Starting on the west:

- 0.07 miles (350 feet) of floodwall segment going through a group of properties. Top of wall elevation of 17 feet. The construction area would be 0.4 acres.
- On the east side of the railroad tracks:
- 0.3 miles (1,600 feet) of T-wall along railroad tracks between Dellwood Pump Station and Baptist Church (this is a change that is not part of alt 6a)
- 0.06 miles (300 feet) of Old Spanish Trail Floodwall segment (construction area would be 0.3 acres).
- Across from LA Highway 433, there would be a 0.09 mile (450 feet) Old Spanish Trail Floodwall segment by Espirit du Lac Street (construction area would be 0.5 acres).
- 0.04 miles (200 feet) of floodwall segment near Belaire Drive (construction area would be 0.2 acres).
- The next floodwall segment would be on the north side along U.S. Highway 190 Business (Fremaux Ave) for 0.08 miles (430 feet) and then the floodwall would turn into the CLECO Corporate Holdings, LLC utility corridor for approximately 1.4 miles (7,200 feet) and would terminate at Manzella Drive. The total length of this floodwall alignment would be 1.5 miles (7,700 feet) long (total construction area would be 9 acres).

1.3.5.3.5 Typical Floodwall Section

The typical T-wall section would consist of a 3-foot thick by 8.5-foot-wide slab with a 1.5- foot-thick stem. The height of the stem would vary. Preliminary assumptions are two rows of 1H:3V battered HP12 x 74 piles, 60 foot deep, spaced at 5-foot centers, and 30-foot-deep steel PZ sheet pile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The design of the T-wall, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.5.3.6 Pump Stations and Floodgates

There would be a series of pump stations and sluicegates as part of this alternative. Starting on the west:

- Bayou Paquet Pump Station complex (500 cfs and 100 foot long) located between Jummonville Road and Mayer Drive. The complex includes a 20-foot navigable floodgate (construction area is 12.6 acres)

- Bayou Liberty Pump Station complex (3,200 cfs and 400 foot long). The complex includes a 20-foot navigable floodgate (construction area would be 12.6 acres)
- Bayou Bonfouca Pump Station complex (3,700 cfs and 300 foot long). The complex includes a 20-foot navigable floodgate (12.6 acres required for construction
- On the east side of the railroad tracks:
- Schneider Canal Pump Station complex (1,200 cfs and 150 foot long) with a 30- foot floodgate (construction area would be 12.6 acres)
- W-14 Pump Station complex (1,200 cfs and 150 foot long) with a 30-foot floodgate (construction area would be 12.6 acres).

1.3.5.3.7 Sluicegates, Vehicular Floodgates and Ramps

There would be a total of three sluicegates, eight vehicular floodgates, one railroad floodgate, and seven ramps. The I-10 would be raised to ramp over the new levee section.

Starting on the west of the alignment:

- Sluicegate #3 at Bayou Paquet (0.25 acres of construction area)
- 30-foot vehicular floodgate at Bayou Paquet Road
- Sluicegate #2 (located east of three major proposed pump stations complexes with floodgates)
- Sluicegate #5 (on the opposite side of the railroad tracks from Dellwood Pump Station), (0.25 acres of construction area). Further refinement would be needed at this location.
- 60-foot railroad floodgate (added for 6c)
- 75-foot Pontchartrain Drive vehicular floodgate (US Highway 11)
- Ramp at Cypress Lakes Drive
- Ramp at Mariner’s Cove Boulevard
- Ramp at the Oak Harbor Country Club entrance
- Ramp at Grand Champions Lane
- The I-10 would be raised to ramp over the new levee section
- 30-foot vehicular floodgate at LA Highway 433 East (Old Spanish Trail)
- Ramp at Fleur Du Lac Street
- Ramp at Nunez Road
- 50-foot vehicular floodgate at U.S. Highway 190 Business (East)
- 20-foot vehicular floodgate at South Holiday Drive
- Ramp at North Holiday Drive
- 20-foot vehicular floodgate at Jaguar Drive
- 20-foot vehicular floodgate at Natchez Drive
- 20-foot vehicular floodgate at Kisatchie Drive

For Item 5 above, the railroad double-swing floodgate was added for Alternative 6c. The analysis was based on Mississippi River levee (MRL) Carrollton Railroad Gate. On the northeast side of the alignment, it is assumed that there would be no need for a vehicular floodgate on North Holiday Drive.

1.3.5.3.8 Structural Assumptions for Alternative 6c

1.) Pump Stations:

a. Bayou Paquet at Bayou Liberty Pump Station
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
b. Bayou Liberty Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
c. Bayou Bonfouca Pump Station:
   i. Assumed to have similar components and configuration as WSLP Pump Station.
   ii. Assumed sill elevation to be equal to inlet at Lake Pontchartrain.
   iii. Assumed integrated navigable floodgate for recreational vessels.
d. Schneider Canal Pump Station:
   i. Assumed 1,200 cfs based on previous feasibility studies.
   ii. Assumed complete redesign of all components.
   iii. Assumed navigable floodgate for recreational vessels.
   iv. Assumed same configuration as the USACE WSLP Pump Station.
e. W-14 Pump Station:
   i. Assumed 1,200 cfs based on hydraulic estimates.
   ii. Assumed all new construction.
   iii. Assumed navigable floodgate for recreational vessels.
   iv. Assumed same configuration as the USACE WSLP Pump Station.

2.) Floodwalls:

Assumed all Alternative 6a and West Slidell floodwalls will be included in Alternative 6c except the floodwall portions of 6a situated adjacent to the east side of the Norfolk Southern Railway Corp. railroad tracks north of the railroad crossing:

a. 0.07 miles (350 feet) floodwall segment passing through several properties. Top of wall elevation of 17 feet. The construction area would be 0.4 acres.

i. Assumed T-wall for this alignment reach as the alignment is adjacent to private property.
ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
iii. Assumed 60 foot 1:3 battered H-piles based on geotechnical analysis.
iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
v. Assumed permanent access to be included in construction area. On the east side of the railroad tracks:

b. 0.3 miles (1,600 feet) of T-wall along the railroad tracks between Dellwood Pump Station and Baptist Church

i. Assumed T-wall for this alignment reach as the alignment is adjacent to the railroad tracks and there is not enough room for levee.
ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
iii. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.
iv. Assumed 50-foot T-wall construction area to side opposite the railroad tracks.
v. Assumed permanent access to be included in construction area.

c. 0.06 miles (300 feet) Old Spanish Trail Floodwall segment (construction area would be 0.3 acres).
i. Assumed T-wall for this alignment reach as the alignment is adjacent to a private warehouse near Old Spanish Trail
ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
iii. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.
iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
v. Assumed permanent access to be included in construction area.

d. Across from LA Highway 433, there would be 0.09 miles (450 feet) Old Spanish Trail Floodwall segment by Espirit du Lac Street (construction area would be 0.5 acres).
i. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.
ii. Assumed 60-foot, 1:3 battered H-piles based on geotechnical analysis.
iii. Assumed T-wall construction area 50 feet to either side from center line of alignment.
iv. Assumed permanent access to be included in construction area.

e. 200 linear foot floodwall south of U.S. Highway 190 Business:
i. Assumed T-wall for this alignment reach as the alignment is between an

energy substation and private property near U.S. Highway 190 Business.

ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.

iii. Assumed 60-foot 1:3 battered H-piles based on geotechnical analysis.

iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.

v. Assumed permanent access to be included in construction area.

f. 7,700 linear foot floodwall reach along power easement north of U.S. Highway 190 Business:

i. Assumed T-wall for this alignment reach is between U.S. Highway 190 Business and U.S. Highway 190 (Gause Boulevard).

ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W-06B.5 Magnolia Pump Station T-walls.

iii. Assumed 60-foot 1:3 battered H-piles based on geotechnical analysis.

iv. Assumed T-wall construction area is 50 feet to either side from center line of alignment.

v. Assumed permanent access to be included in construction area.

3.) Access Floodgates:

a. Paquet Road East floodgate

i. Assumed floodgate.

ii. Assumed rough ground elevation via Google Earth.

iii. Assumed floodgate width based on road width.

b. Vehicular Roller Access Floodgates

i. Assumed all Alternative 6c vehicular floodgates to be roller floodgates.

ii. Assumed sill to be at existing ground level.

4.) Control Floodgates:

a. Sluicegates #2 and #4

i. Assumed sluicegate.

ii. Assumed width based on stream width.

iii. Assumed sill is 5 feet below ground elevation.

1.3.6 Alternative 7: Eastern Slidell

Alternative 7 would include a levee, diversion channel, and channel improvements to address flooding. This alternative also consists of pump stations, floodgates, vehicular floodgates, and ramps. The features in this Alternative are all separate and combinable and could all be implemented if justified. Refer to Figures D:1-15; D;1-16; D:1-17; D:1-18; and D:1-19.

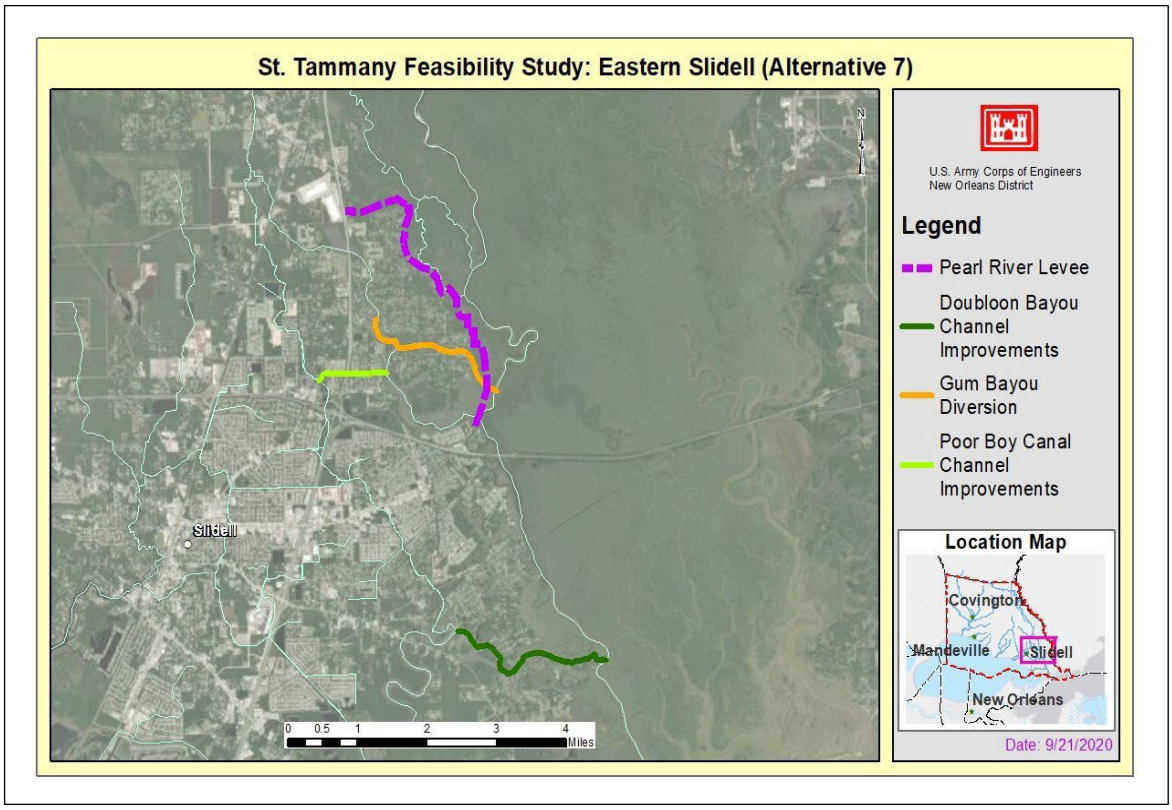


Figure D:1-15. Alternative 7 Eastern Slidell

1.3.6.1 Levee

The overall length of the Pearl River levee would be approximately 4.8 miles (25,000 feet). The levee would extend from I-59 continuing east and would turn south extending along the Pearl River Tributaries to the intersection of the West Pearl River and Gum Bayou. The Pearl River levee alternative would reduce risk of riverine flooding. The alignment would stay clear from existing residential streets and houses. This alternative also would maintain a buffer distance from the closest tributary channel for Pearl River. The Pearl River levee has been adjusted to push the alignment to the east out of the residential neighborhoods wherever possible but keeping the Pearl River Tributary as a constraining factor.

1.3.6.2 Levee Design Section and Borrow Quantities

The new levee is designed using a preliminary design elevation ranging from 16.2 feet to 19 feet NAVD 88 and uses the elevations of the existing ground obtained from the terrain raster dataset. The preliminary assumptions are that the levee would have a 10-foot-wide levee crown and side slopes of 1V:3H. The construction of this levee alignment would impact approximately 57 acres. This levee alignment would require 350,000 cubic yards of fill/borrow (includes 30 percent contingency). Berm sections would be determined once data is available for analysis.

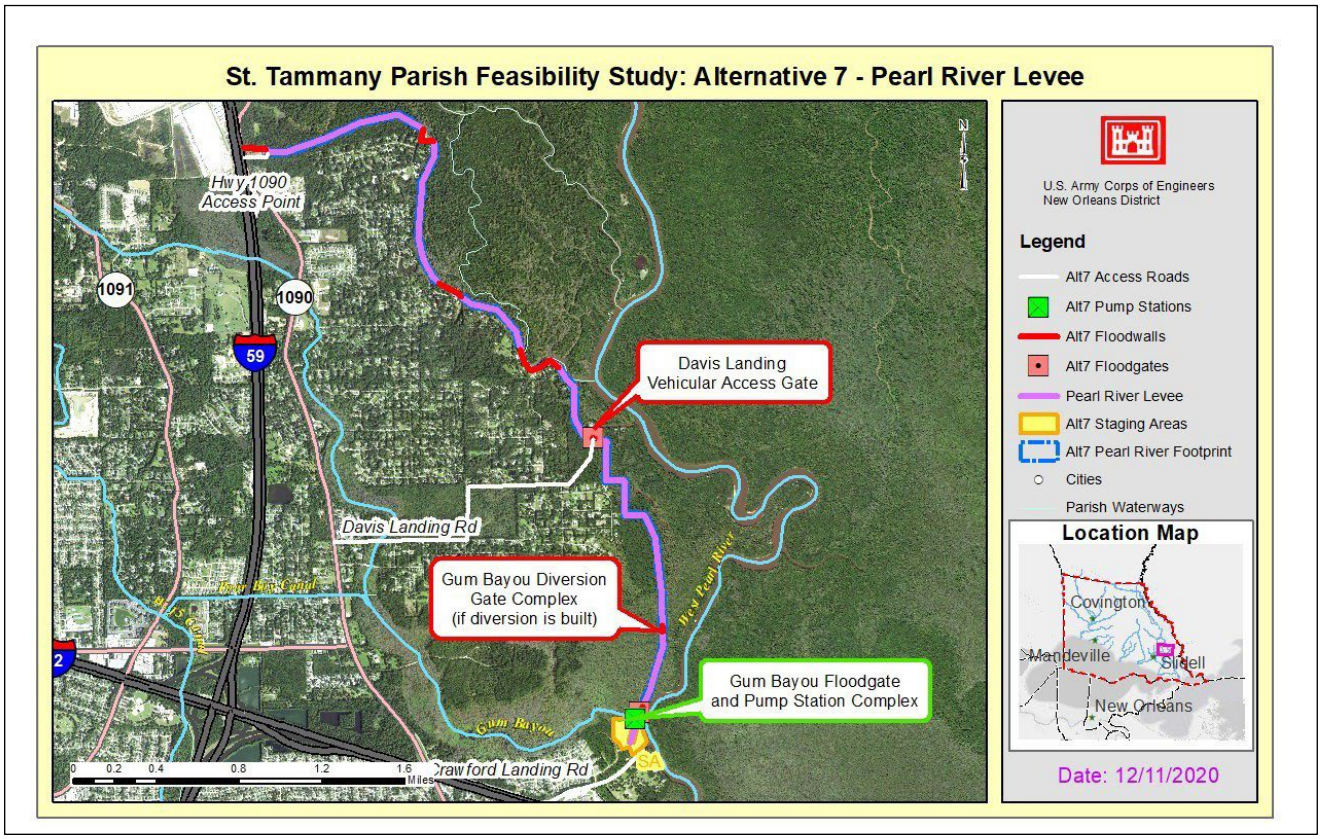


Figure D:1-16. Alternative 7 Pearl River Levee

1.3.6.3 Floodwall Elevation and Location

For this alignment, the elevation of the floodwall segments would be 15 feet.

There would be four floodwall sections for a total of 0.64 miles (3,400 feet). Three of them would remain in residential neighborhoods, at locations where the houses are built adjacent to the Pearl River Tributary. The construction of these floodwall segments would impact approximately 3 acres.

There would be 0.11-mile (600 feet) floodwall segment near I-59. The floodwall segment starts at the tie-in to Highway 190 (which is on the east side of I-59) passing on the north side of a residential property that has access to US Highway 190.

There would be two 0.12-mile (650 feet) floodwall segments. One is around a cluster of several properties at the northeast end of Forest Drive in Morgan Bluff neighborhood. The levee alignment turns from east to south near this location. Earthen levee continues south along Old Pearl River tributary. The other floodwall segment would be located behind several residential properties near the end of Houmas Court in Morgan Bluff neighborhood.

There would be a 0.27-mile (1,500 feet) floodwall segment behind several residential properties near the eastern end of Smith Baggert Rd in Morgan Bluff neighborhood.

1.3.6.4 Typical Floodwall Section

The typical T-wall section would consist of a 3-feet thick, 8.5-feet wide slab with a 1.5-feet thick stem. The height of the stem would vary. Preliminary assumptions are two rows of 1H:3V battered HP12 x 74 piles, 60 feet deep, spaced at 5 feet centers, and 30 feet deep steel PZ sheet pile. Approximately 1,850 square feet of slope protection would be provided at floodwall/levee tie-ins (6-inch-thick slope pavement). The design of the T-wall including the foundation is subject to change once detailed geotechnical investigations are conducted.

1.3.6.5 Pump Stations and Floodgates

There would be a 600-cfs pump station where Gum Bayou and West Pearl River intersect, and a 30-foot floodgate.

There would be a 30-foot sluicegate.

1.3.6.6 Vehicular Floodgates and Ramps

There would be a 30-foot vehicular floodgate at Davis Landing Road.

1.3.6.7 Gum Bayou Diversion

This alternative would include the unmanned Gum Bayou Diversion, to target rainfall and riverine flooding by diverting Gum Bayou to the Pearl River through a new channel. The diversion would extend from Oak Alley Drive and would run to the West Pearl River. The length of the diversion would be 1.8 miles (9,300 feet). The upstream end of the diversion channel would tie into the existing channel invert of Gum Bayou (+4.48 feet NAVD 88). The invert would decline a total of 5 feet before tying into the West Pearl River (-0.48 feet NAVD 88).

The existing ground elevation is between 8 feet NAVD 88 and 10 feet NAVD 88. The preliminary diversion channel design assumes an existing bank elevation 10 feet NAVD88, a 10-foot bottom width, and a bank with 1V:3H slope. A maximum of 100,000 cubic yards of material would be removed to form the alignment. For the diversion, approximately 35 acres of ROW would be needed for a temporary easement. A key assumption includes a 50-foot access corridor on the top of bank of each side of the Gum Bayou channel. Material disposal would be trucked away from the site or sidecast along the bankline of the channel. Refer to Figure D:1-17.

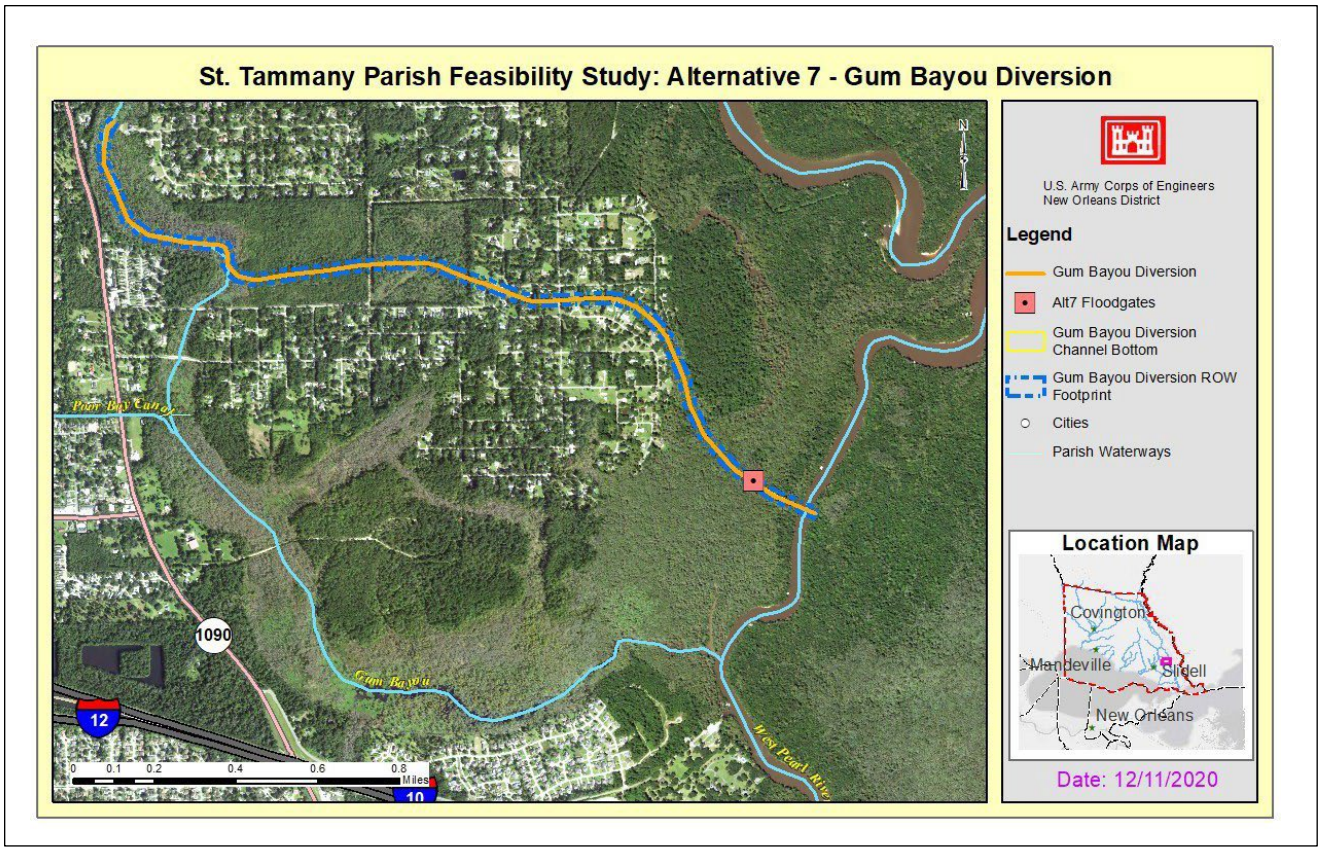


Figure D:1-17. Alternative 7 Gum Bayou Diversion

1.3.6.8 Poor Boy Canal Channel Improvements

The Poor Boy Canal channel improvements would address rainfall and riverine flooding. The channel improvements in Poor Boy Canal would extend from LA Highway 1091, would cross LA Highway 59 and North Military Road, and would terminate into Gum Bayou. This alternative consists of channel improvements of approximately 1 mile (5,288 feet) of channel. The preliminary design of the channel improvements assumes an existing bank elevation of 1 foot, and a 10-feet bottom width. The bank would be at 1V:3H side slope. The improvements would include clearing and grubbing, and mechanical dredging of the Poor Boy Canal channel. The channel bottom would be lowered by 5 feet. Approximately 12 acres of Poor Boy Canal channel would be cleared and grubbed prior to mechanical dredging. An assumed maximum of 80,000 cubic yards of material is anticipated to be removed. Material removed may include sediment, trees, debris, or other obstructions within the waterway. For the channel improvements, approximately 16 acres of temporary ROW would be needed for a temporary easement for construction. Potential staging areas would be at the intersection of the channel and North Military Road on the right descending bank east of the road (0.6 AC), the corner of canal street and N Military Road on the right descending bank of the channel (0.3 AC), and along LA Highway 1090 adjacent to the channel (0.7 AC).

Assumptions for channel improvements include ROW at 65 feet from the centerline to each side of the Poor Boy Canal channel as a general guideline (total width of 130 feet), which includes space for equipment access. All work would be within the project footprint. The material requiring disposal would be trucked away from the site. Assumption is that all access would be through public lands. Refer to Figure D:1-18.

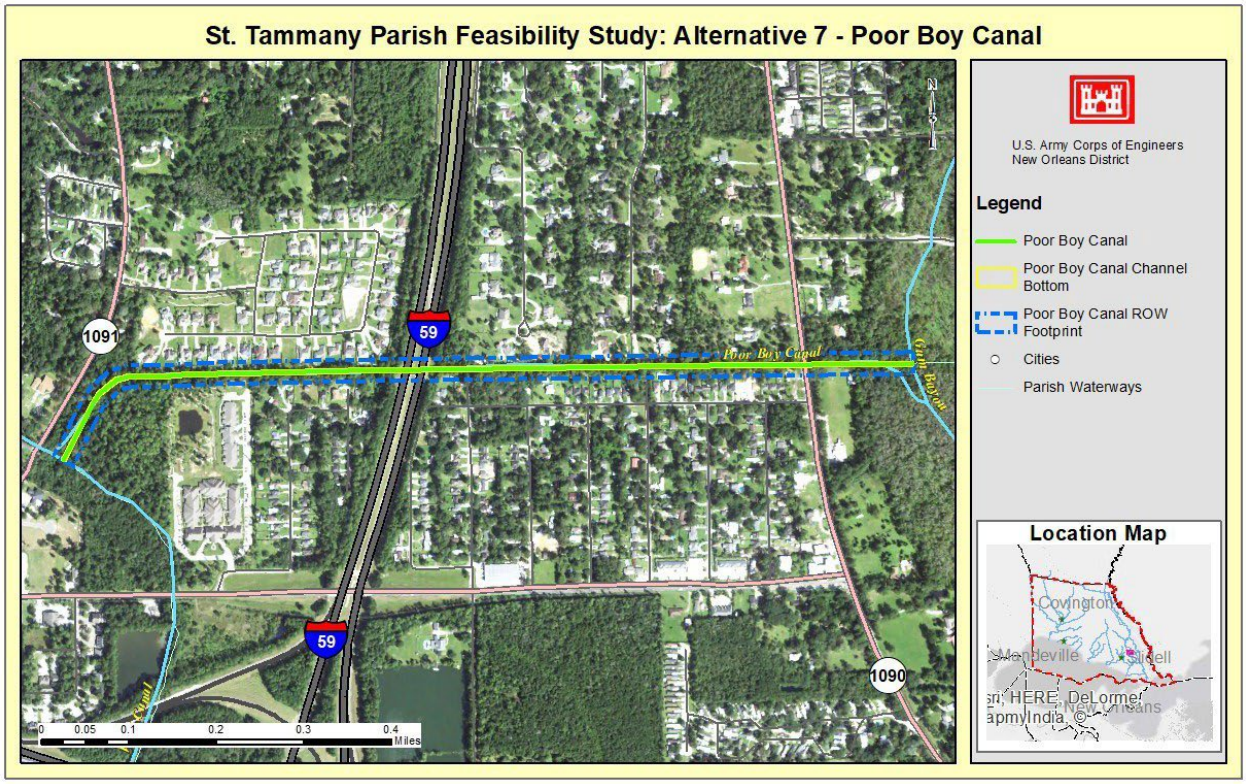


Figure D:1-18. Alternative 7 Poor Boy Canal Channel Improvements

1.3.6.9 Doubloon Bayou Channel Improvements

This alternative includes the Doubloon Bayou channel improvements to address rainfall and riverine flooding. The Doubloon Bayou channel improvements would extend from the intersection of Doubloon Bayou and W-15 Canal and end on West Pearl River. This alternative would consist of channel improvements of approximately 3 miles (13,500 feet) of channel. The preliminary design of the channel improvements assumes an existing bank elevation of 1 foot, and a 25-feet bottom width. The bank would be at 1V:3H slope. The improvements would include clearing and snagging and mechanical dredging of the Doubloon Bayou channel. The channel bottom would be lowered by 5 feet. Approximately 30 acres of channel would be cleared and grubbed prior to mechanical dredging. An assumed maximum of 190,000 cubic yards of material may be

removed from Doubloon Bayou. Material removed may include sediment, trees, debris, or other obstructions within the waterway. Method for removal can be by a small hydraulic dredge (10-inch to 14-inch cutterhead or suction depending on the material) or by clamshell bucket. The lower portion of the bayou would be hydraulically dredged whereas the upper portion of Doubloon Bayou would need to be mechanically dredged. The material would need to be pumped to a disposal area or pumped/placed into a barge for hauling away and disposed of downriver. The disposition of the 190,000 cubic yards of material assumed to be removed from the Doubloon Bayou channel is as follows:

- 20 percent hydraulic dredged (38,000 cubic yards) (2,700 linear feet)
- 35 percent mechanically dredged and hauled away (66,500 cubic yards) (4,725 linear feet)
- 45 percent mechanically dredged and side cast (85,500 cubic yards) (6,075 linear feet)

For the channel improvements, approximately 40 acres of ROW would be needed for a temporary easement. Assumptions for channel improvements include 65 feet from the centerline of each side of the Doubloon Bayou channel for ROW as a general guideline (total width of 130 feet), which includes space for equipment access. All work would be within the project footprint. The temporary work easement would be within ROW. Refer to Figure D:1- 19.

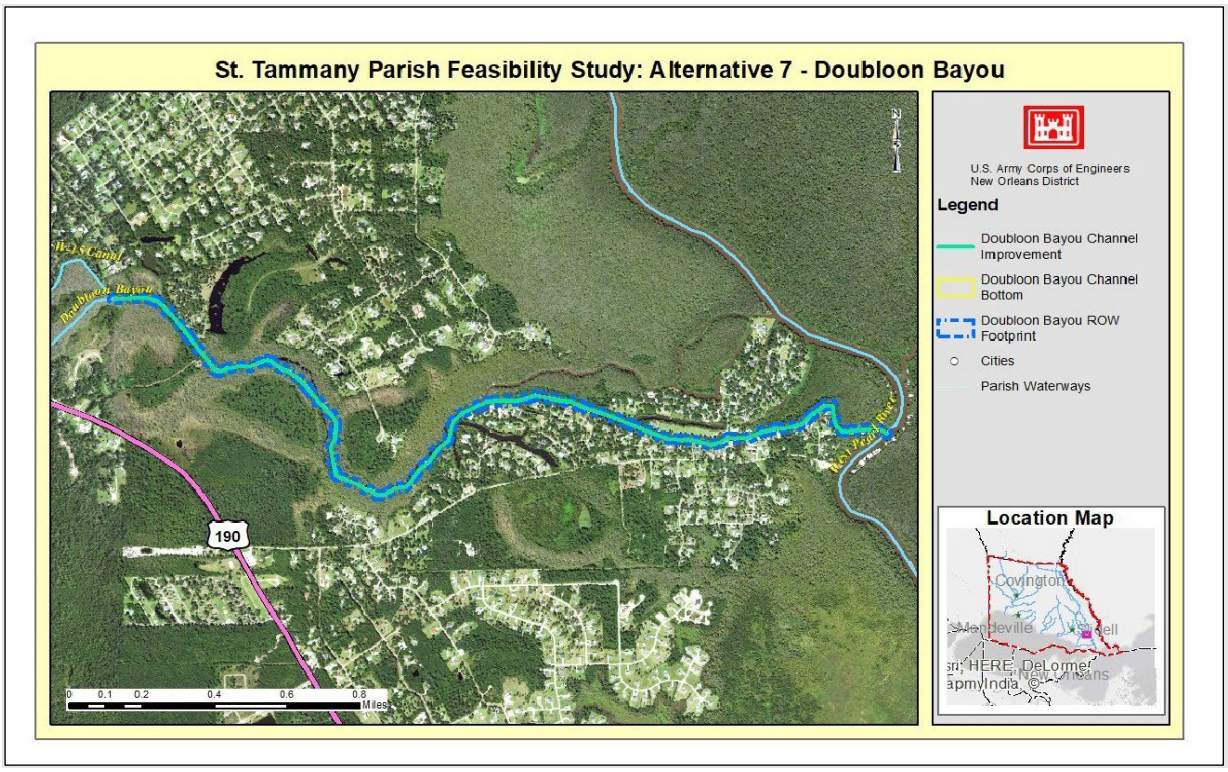


Figure D:1-19. Alternative 7 Doubloon Bayou

1.3.6.10 Structural Assumptions for Alternative 7

1.) Pump Stations:
   a. Gum Bayou Pump Station
      i. Assumed new construction.
      ii. Assumed no navigable floodgate for recreational vessels.
      iii. Assumed location at existing Gum Bayou outlet.

2.) Floodwalls:
   a. 600-foot floodwall near I-59
      i. Assumed T-Wall as alignment is too close to development.
      ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W- 06B.5 Magnolia Pump Station T-walls.
      iii. Assumed 60-feet 1:3 battered H-piles based on geotechnical analysis.
      iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
      v. Assumed permanent access to be included in construction area.
   b. 650-foot floodwall near Forest Drive:
      i. Assumed T-Wall as alignment is too close to development.
      ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W- 06B.5 Magnolia Pump Station T-walls.
      iii. Assumed 60-feet 1:3 battered H-piles based on geotechnical analysis.
      iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
      v. Assumed permanent access to be included in construction area.
   c. 650-foot floodwall near Houmas Court:
      i. Assumed T-Wall as alignment is too close to development.
      ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W- 06B.5 Magnolia Pump Station T-walls.
      iii. Assumed 60-feet 1:3 battered H-piles based on geotechnical analysis.
      iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.
      v. Assumed permanent access to be included in construction area.
   d. 1,500-foot floodwall near Smith Baggert Road:
      i. Assumed T-Wall as alignment is too close to development.
      ii. Assumed T-wall dimensions and configurations similar to NOV-NF-W- 06B.5 Magnolia Pump Station t-walls.
      iii. Assumed 60-feet 1:3 battered H-piles based on geotechnical analysis.
      iv. Assumed T-wall construction area 50 feet to either side from center line of alignment.

v. Assumed permanent access to be included in construction area.

3.) Floodgates:

a. Vehicular Roller Access Floodgate at Davis Landing Road:

i. Assumed 30-foot roller floodgate.

ii. Assumed rough ground elevation via Google Earth.

iii. Assumed floodgate width based on road width.

4.) Control Floodgates:

a. Control floodgate at Gum Bayou Diversion:

i. Assumed 30-foot sluicegate.

1.3.7 Alternative 8- Upper Tchefuncte/Covington Channel

The Upper Tchefuncte/Covington Channel alternative includes measures to reduce rainfall and riverine flooding in the upper reaches of the Tchefuncte and Bogue Falaya Rivers. The measures in this alternative are all separate. They are combinable within this alternative or could also be combined with other alternatives. If justified, all the above measures could be implemented. Refer to Figure D:1-20.

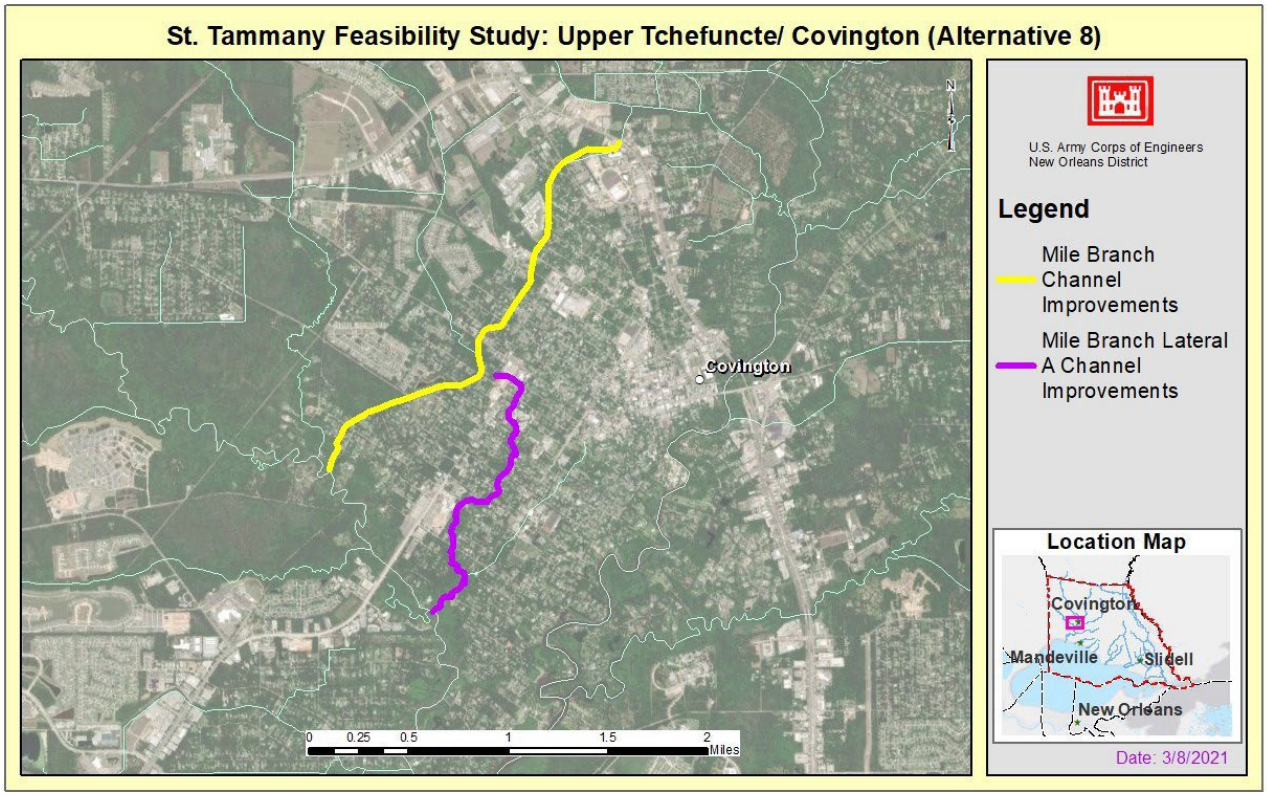


Figure D:1-20. Alternative 8 Upper Tchefuncte/Covington

1.3.7.1 Mile Branch Channel Improvements

The Mile Branch channel improvements starts at the intersection of Mile Branch and U.S. Highway 190, crossing U.S. Highway 190 Business, and would end at the

intersection of Mile Branch and the Tchefuncte River. This measure consists of channel improvements on the lower 2.15 miles (11,341-foot channel) of Mile Branch in Covington, LA.

The improvements would include clearing and grubbing and mechanical dredging of the channel. The channel bottom would be lowered by 5 feet. Approximately 20 acres of channel would be cleared and grubbed prior to mechanical dredging. An assumed maximum of 130,000 cubic yards of material may be mechanically dredged from the channel. The preliminary design assumes an existing bank elevation of 1 foot, and a 10-feet bottom width. The bank would be at 1V:3H slope. Material removed may include sediment, trees, debris, or other obstructions within the waterway. For the channel improvements (includes 20 acres of clear and grubbing), approximately 34 acres of ROW would be needed for temporary work areas. The material will be hauled away from the site.

The Mile Branch channel improvements may include bridge replacements or culverts (starting from north to south) at 29th, 28th, 25th, 23rd, 21st, 19th, and 18th Avenues. No work would be anticipated at the 15th and 11th Avenue channel crossings as those bridges have been replaced prior to this study (and the new bridges were designed to safely pass higher flows on Mile Branch). Refer to Figure D:1-21.

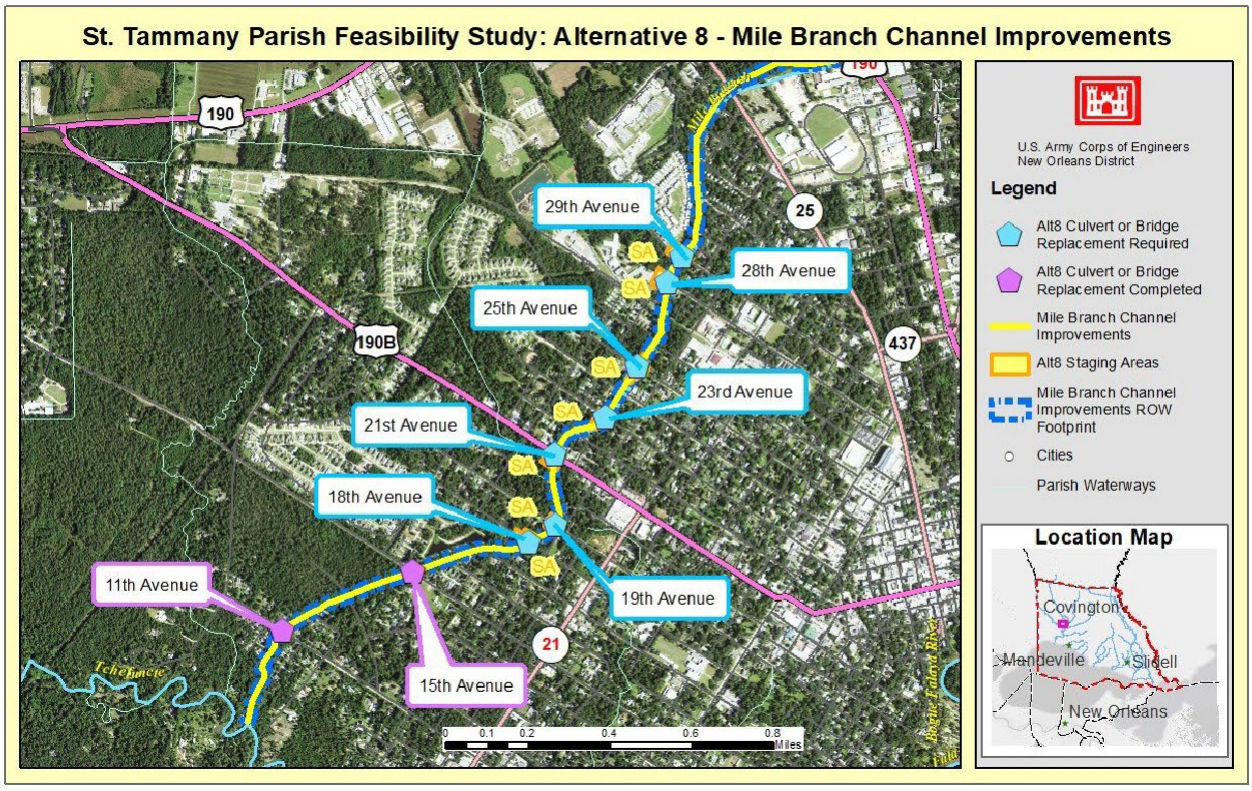


Figure D:1-21. Alternative 8 Mile Branch Channel Improvements

1.3.7.2 Lateral A Mile Branch Channel Improvements

This alternative includes channel improvements to the Lateral A Mile Branch to address rainfall and riverine flooding. The work would extend from just southwest of the intersection of U.S. Highway 190 and LA Highway 21. The channel improvements would include clearing and snagging approximately 1.73 miles (9,129 feet channel) of Lateral A Mile Branch. The preliminary design of the channel improvements for Lateral A Mile Branch assumes an existing bank elevation of 1 foot, and a 10-feet bottom width. The bank would be at a 1V:3H slope. The channel bottom would be lowered by 5 feet. Approximately 16 acres of channel would be cleared and grubbed prior to mechanical dredging. An assumed maximum of 104,000 cubic yards of material may be removed from the channel. Material removed may include sediment, trees, debris, or other obstructions within the waterway. For the channel improvements, approximately 28 acres of ROW would be needed for a temporary easement. The material will be hauled away from the site.

The Lateral A Mile Branch channel improvements may include bridge replacements or culverts (starting from north to south) at the intersection of Tyler Street and LA Highway 21, and at 19th, 17th, 16th, 15th, 14th, 13th, 11th, and 8th Avenues. Assumptions for channel improvements include a ROW limit measured 65 feet from the centerline to each side of the channel for as a general guideline (total width of 130 feet), which includes space for equipment access. All work would be within the project footprint. Temporary work easement would be within ROW. The material requiring disposal would be trucked away from the site. Assumption is that all access would be through public lands. Refer to Figure D:1-22.

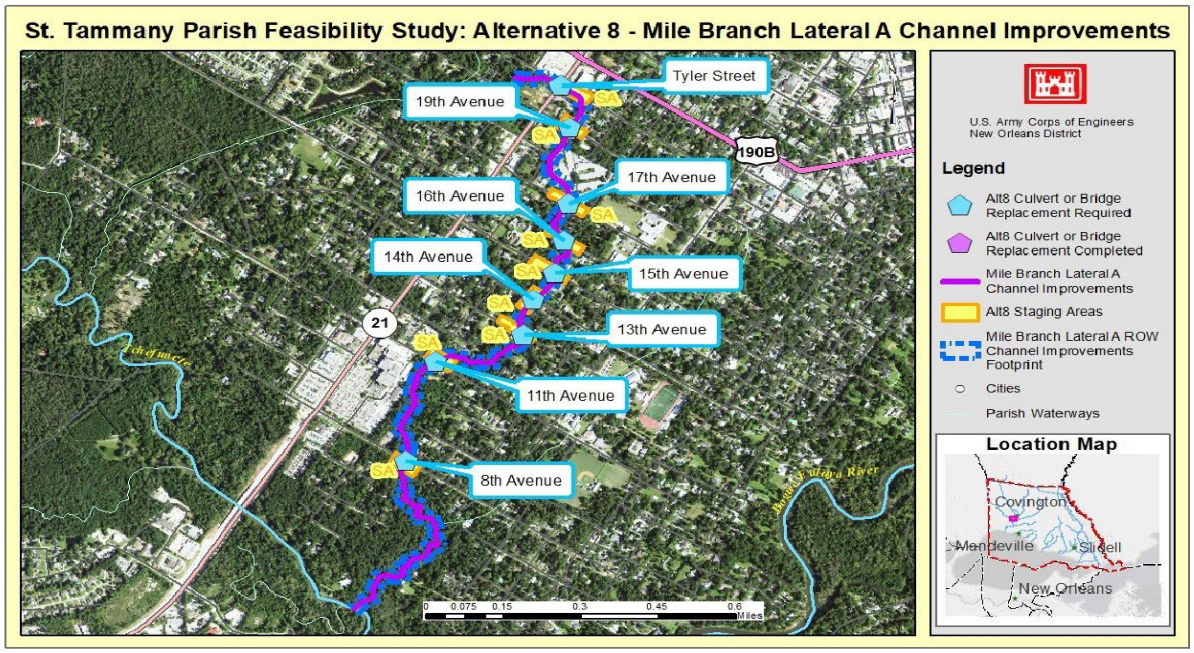


Figure D:1-22. Alternative 8 Mile Branch Lateral A Channel Improvements

1.3.7.2.1 Structural Assumptions for Alternative 8

1.) Pump Stations:
   a. Assume none
2.) Floodwalls:
   a. Assume none
3.) Access Floodgates:
   a. Assume none
4.) Control Floodgates:
   a. Assume none

1.3.8 Alternative 9 Mandeville Lakefront

This alternative is proposed to reduce the risk from a coastal storm. The following variations to Alternative 9 would be mutually exclusive within the alternative. This means that only one variation within alternative 9 could be implemented, if justified, to become part of the TSP. Refer to Figure D:1-23.

- Alternative 9a would replace the existing lakefront seawall to elevation 7.3 feet NAVD 88 and would add a passive drainage option on Bayou Ravine Aux Coquilles and Little Castine Bayou.
- Alternative 9b would replace the existing seawall to elevation 7.3 feet NAVD 88 and would add pump stations at the lakefront at Girod Street and Ravine Aux Coquilles.
- Alternative 9c would replace the existing seawall to elevation 18 feet NAVD 88 and would add pump stations at the lakefront at Girod Street and Ravine Aux Coquilles.

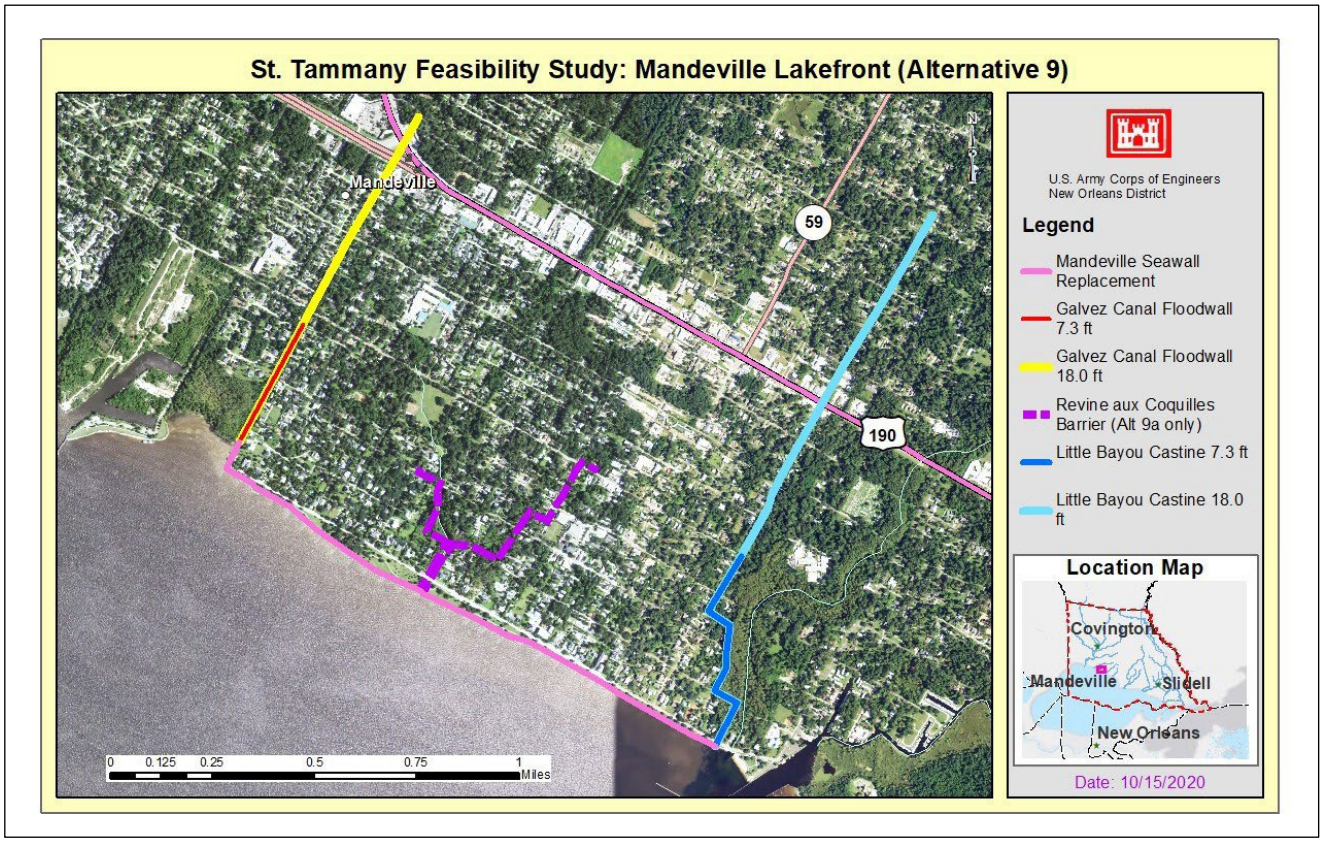


Figure D:1-23. Alternative 9 Mandeville Lakefront

1.3.8.1 Alternative 9a Mandeville Lakefront – Seawall Passive Drainage

Alternative 9a would consist of a passive system in conjunction with the new 7.3 feet seawall at the lakefront of Mandeville, Louisiana. There would be an opening in the lakefront seawall at Ravine Aux Coquilles. The design elevation for the seawall would be 7.3 feet NAVD 88, which is above the 20-year surge level in the planned project completion year of 2032.Elevation 7.3 feet would be 2 feet higher than the existing seawall. Local interests communicated a strong preference for this elevation. Refer to Figures D:1-24, D:1-25, D:1- 26, and D:1-27.

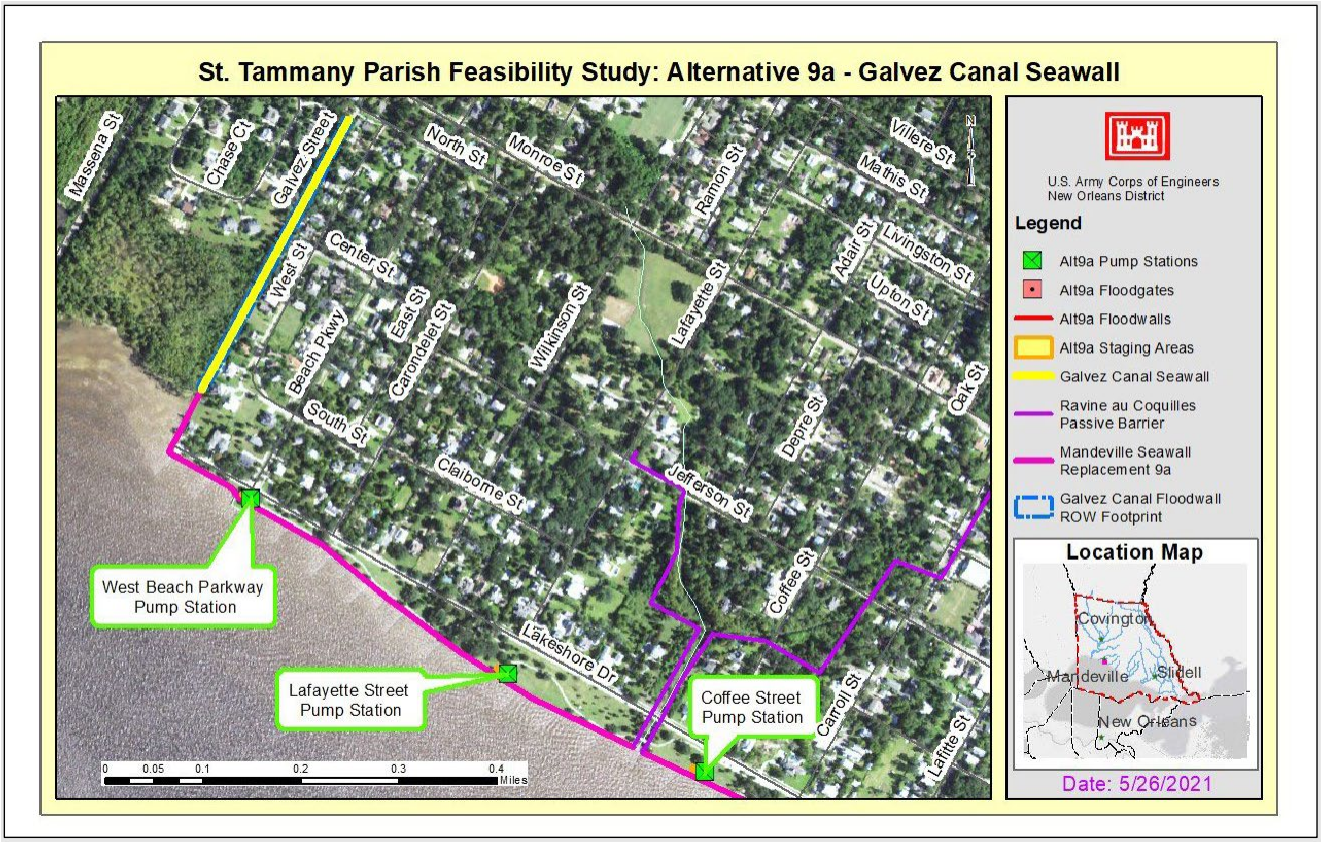


Figure D:1-24. Alternative 9a Galvez Canal Seawall

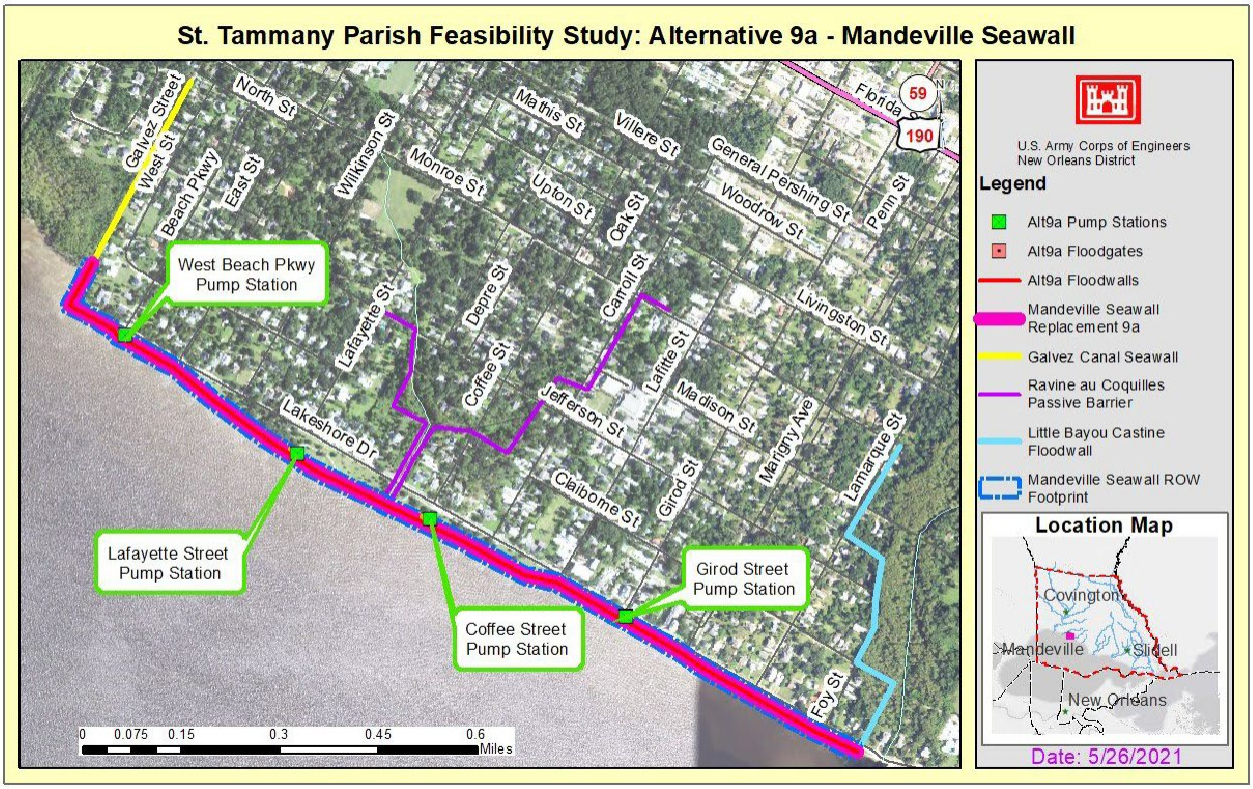


Figure D:1-25. Alternative 9a Mandeville Seawall

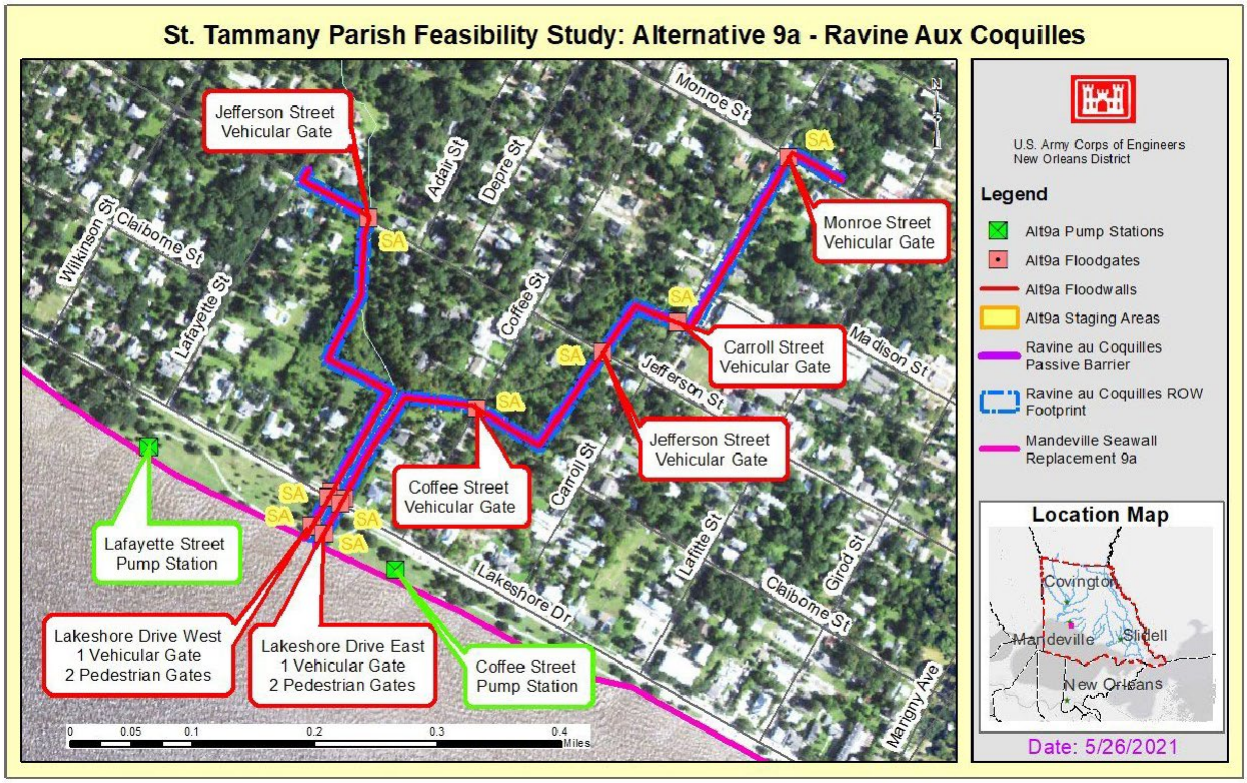


Figure D:1-26. Alternative 9a Ravine Aux Coquilles

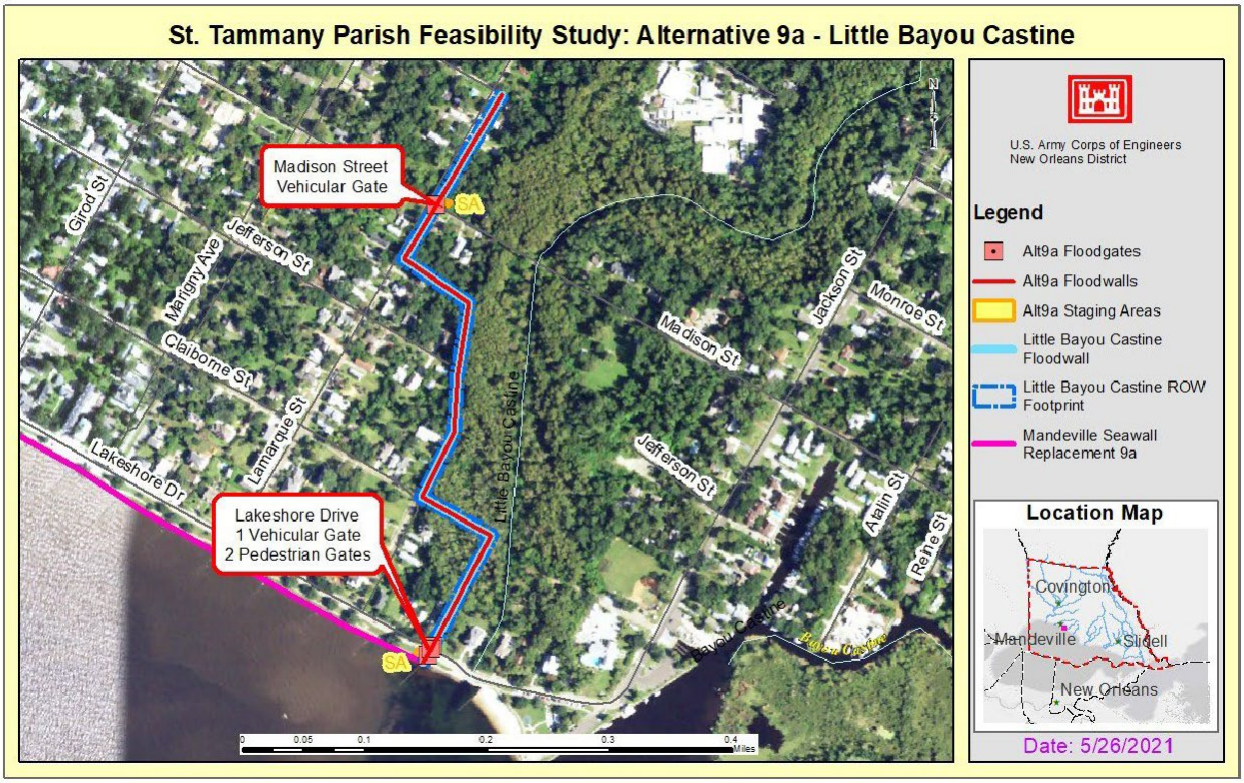


Figure D:1-27. Alternative 9a Little Bayou Castine

1.3.8.1.1 Description of Alignment

For Alternative 9a, the seawall would be replaced in its entirety on the same alignment as the existing seawall. The seawall would be parallel to and on the south side of Lakeshore Drive. On the west side it would continue on the Galvez Canal to North Street, connecting to floodwalls (passive barrier) on the west and east sides of Ravine Aux Coquilles, and ending on the west side of Little Bayou Castine where it would connect to the floodwall (passive barrier) for Little Bayou Castine. This new floodwall would be along the Little Bayou Castine and would end on Monroe and Lamarque Streets.

1.3.8.1.2 Floodwall Elevation and Location

For elevation 7.3 feet NAVD 88, the new seawall would be approximately 1.5 miles long (7,703 feet). The new floodwall at Galvez Canal would be at elevation 7.3 feet NAVD 88 and 0.3 miles (1,740 feet) long. The total floodwall length would be approximately 18,000 feet. This length includes Ravine Aux Coquilles West Passive Barrier (length

2,067 feet), Ravine Aux Coquilles East Passive Barrier (length 3,485 feet), and Little Bayou Castine West Passive Barrier (length 3,000 feet).

1.3.8.1.3 Typical Floodwall Section

The typical I-wall section for the seawall would consist of a 30-feet deep steel PZ sheet pile with a concrete cap. The new seawall would impact 10.3 acres of construction area. The

typical passive floodwall at Ravine Aux Coquilles would be a combination of T-wall and I– wall. The floodwall at Galvez Canal would be an I-wall. The existing seawall would be demolished prior to the construction of the new seawall. It was assumed that for the existing seawall, the concrete cap is 3 feet by 1.5 feet and the length of the wall is 7,500 feet. The existing vinyl sheet pile at Galvez Canal (length of 2,050 feet) would be demolished and would be replaced with seawall. The design of the floodwalls, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

The new passive alignment floodwalls would keep storm surge from reaching the developed areas adjacent to them, while maintaining natural drainage in the bayous. The passive drainage floodwalls would be located at Ravine Aux Coquilles West Passive Barrier, Ravine Aux Coquilles East Passive Barrier, and Little Bayou Castine West Passive Barrier.

More detailed information on the new floodwalls includes:

- Ravine Aux Coquilles West Passive Barrier: length of 0.4 miles or 2,067 feet (1,817 feet of T-Wall and 250 feet of I-Wall), and 2.4 acres of construction area.
- Ravine Aux Coquilles East Passive Barrier: length of 0.7 miles or 3,485 feet (2,562 feet of T-Wall and 923 feet of I-Wall), and 4 acres of construction area.
- Little Bayou Castine West Passive Barrier: length of 0.6 miles or 3,000 feet (1,300 feet of T-Wall and 1,700 feet of I-Wall), 3.9 acres of construction area.

1.3.8.1.4 Pump Stations and Floodgates

Four pump stations would be constructed at the lakefront seawall on West Beach Parkway (116 cfs), Lafayette Street (33 cfs), Coffee Street (106 cfs), and Girod Street (139 cfs). Each pump station would need a construction area of 0.009 acres for a total 0.03 acres for the four pump stations. A 100-feet by 50-feet staging area for seawall within the lakefront was assumed. There would be no pump station at Foy Street. There would be no floodgates on Galvez Canal for the elevation 7.3 feet NAVD 88 of the floodwall.

1.3.8.1.5 Vehicular Floodgates and Ramps

There would be a total of nine vehicular floodgates and six pedestrian floodgates for Alternative 9a. At Ravine Aux Coquilles East floodwall, there would be vehicular floodgates for access, one on each street, for a total of four on each of the following streets: Coffee Street, Jefferson Street, Carroll Street, and Monroe Street (LA Highway 1087), one vehicular floodgate and two pedestrian floodgates at Lakeshore Drive East. There would be designated staging areas adjacent to the construction areas for all the floodgates.

At Ravine Aux Coquilles West floodwall, there would be a vehicular floodgate at Jefferson Street, one vehicular floodgate and two pedestrian floodgates at Lakeshore Drive West.

There would be designated staging areas within each of the floodgates.

At Little Bayou Castine, there would be four floodgates from south to north, there would be a vehicular gate at Madison Street, one vehicular floodgate and two pedestrian floodgates at Lakeshore Drive. There would be designated staging areas adjacent to the construction areas for all the floodgates.

1.3.8.1.6 Structural Assumptions for Alternative 9a

1.) Pump Stations:

a. West Beach Parkway Pump Station
   i. Assumed 116.1 cfs based on hydraulic analysis.
   ii. Assumed pump station to be located adjacent to seawall.
   iii. Assumed pump station to be enclosed in prefabricated metal building.
   iv. Assumed same configuration as the USACE Westshore Lake Pontchartrain Prescott Road Pumping Station.

b. Lafayette Street Pump Station:
   i. Assumed 32.9 cfs based on hydraulic analysis.
   ii. Assumed Pump station to be located adjacent to seawall.
   iii. Assumed pump station to be enclosed in prefabricated metal building.
   iv. Assumed same configuration as the USACE Westshore Lake Pontchartrain Prescott Road Pumping Station.

c. Coffee Street Pump Station:
   i. Assumed 105.6 cfs based on hydraulic analysis.
   ii. Assumed pump station to be located adjacent to seawall.
   iii. Assumed pump station to be enclosed in prefabricated metal building.

iv. Assumed same configuration as the USACE Westshore Lake Pontchartrain Prescott Road Pumping Station.

d. Girod Street Pump Station:
   i. Assumed 138 cfs based on hydraulic analysis.
   ii. Assumed pump station to be located adjacent to seawall.
   iii. Assumed pump station to be enclosed in prefab metal building.
   iv. Assumed same configuration as the USACE Westshore Lake Pontchartrain Prescott Road Pumping Station.

2.) Floodwalls:
   a. Ravine Aux Coquilles western floodwall
      i. Assumed T-wall with sill at Elevation 0 to extend from seawall to Elevation 5.
      ii. Assumed I-wall for alignment elevations from 5 to 7.3.
      iii. Assumed alignment to avoid infringing on private property.
      iv. Assumed alignment to cross at two pedestrian paths, Lakeshore Drive, and Jefferson Street.
   b. Ravine Aux Coquilles eastern floodwall
      i. Assumed T-wall with sill at Elevation 0 to extend from seawall to Elevation 5.
      ii. Assumed I-wall for alignment elevations from 5 to 7.3.
      iii. Assumed alignment to avoid infringing on private property.
      iv. Assumed alignment to cross at two pedestrian paths, Lakeshore Drive, Coffee Street, Jefferson Street, Carroll Street, and Monroe Street.
   c. Little Bayou Castine western floodwall
      i. Assumed T-wall with sill at Elevation 0 to extend from seawall to Elevation 5.
      ii. Assumed I-wall for alignment elevations from 5 to 7.3.
      iii. Assumed alignment to avoid infringing on private property.
      iv. Assumed alignment to cross at two pedestrian paths, Lakeshore Drive, and Madison Street.
   d. Lakefront Seawall
      i. Assumed complete replacement of existing I-wall seawall
      ii. Assumed complete replacement of Galvez Canal seawall
      iii. Assumed Seawall to extend from North Street tie-in to Little Bayou Castine inlet.

3.) Access Floodgates:
   a. Vehicle Roller floodgates
      i. Assumed vehicle roller floodgates at intersection of proposed alignment and streets.
   b. Pedestrian Roller floodgates
      i. Assumed pedestrian swing floodgates at intersections of proposed

alignment and pedestrian paths.

4.) Control Floodgates:

a. Assumed none.

1.3.8.2 Alternative 9b Mandeville Lakefront – Seawall and Pump Stations

Alternative 9b would consist of a series of inlet pumps in conjunction with the new seawall at the lakefront in Mandeville, Louisiana. This alternative would include a pump station at Bayou Ravine Aux Coquilles. For the seawall replacement, elevation of 7.3 feet NAVD 88 was analyzed. The design elevation for the seawall would be 7.3 feet NAVD 88, which is above the 20-year surge level in the planned project completion year 2032. Elevation 7.3 feet would be 2 feet higher than the existing seawall elevation. Local interests communicated a strong preference for this elevation. Refer to Figures D:1-28, D:1-29, D:1- 30, and D1-31.

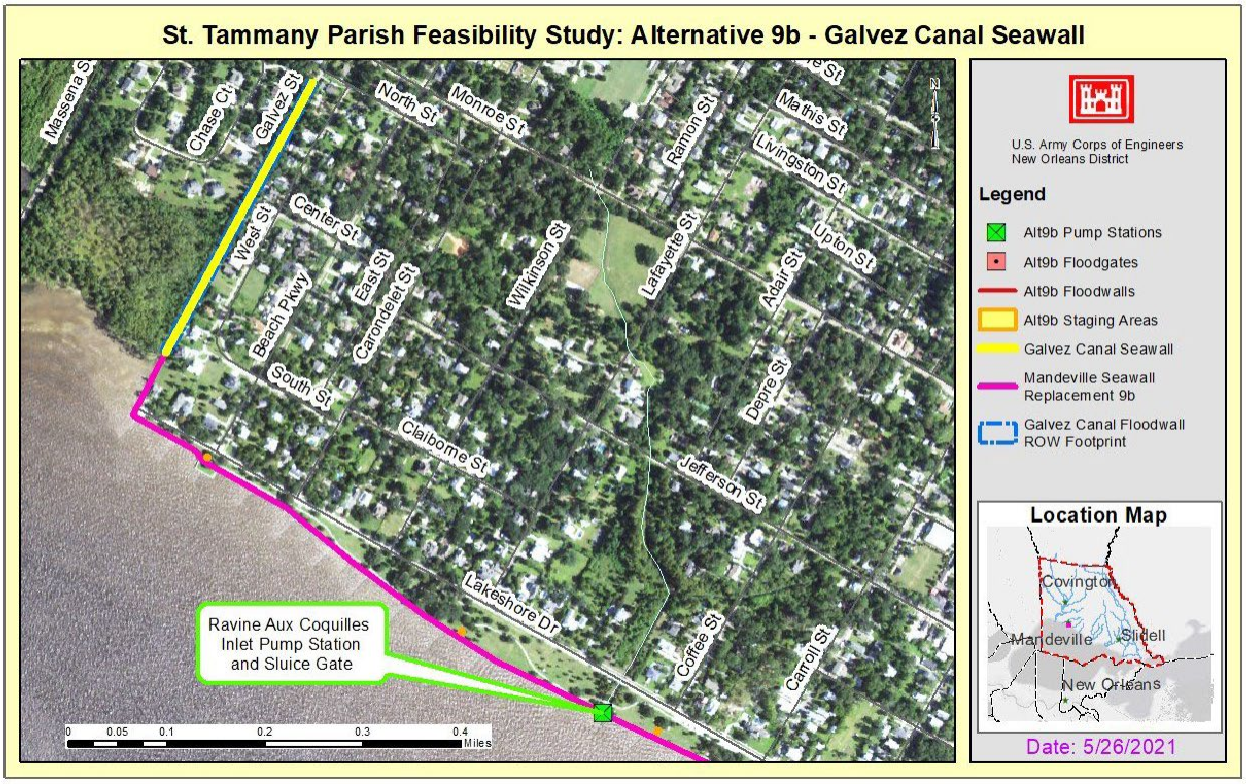


Figure D:1-28. Alternative 9b Galvez Canal Seawall

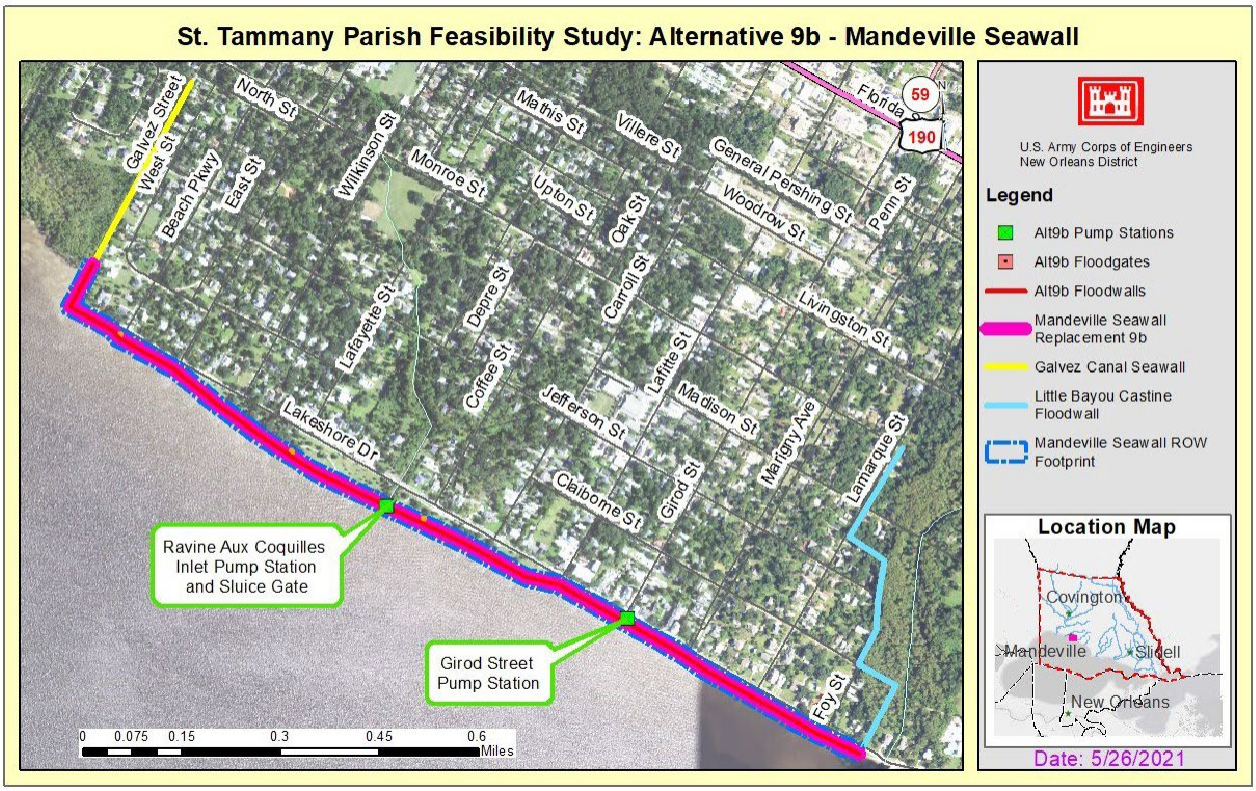


Figure D:1-29. Alternative 9b Mandeville Seawall

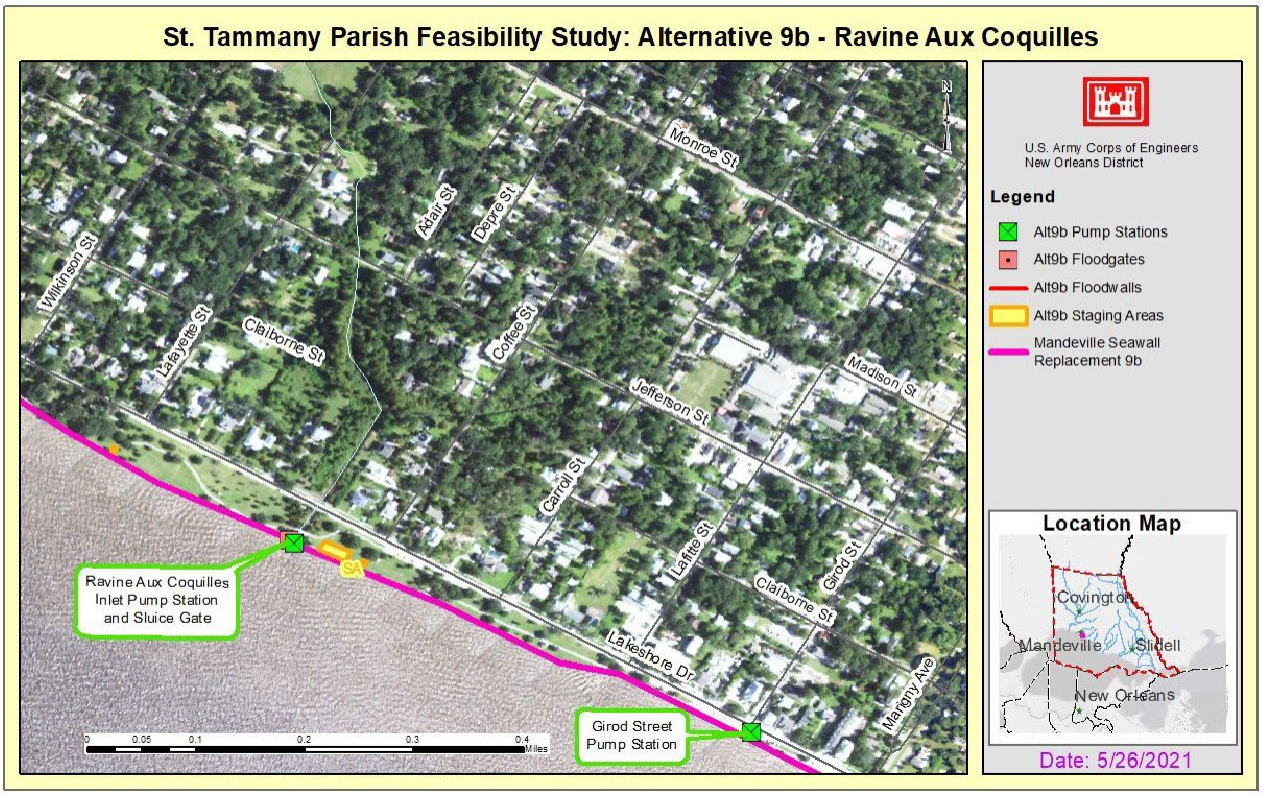


Figure D:1-30. Alternative 9b Ravine Aux Coquilles

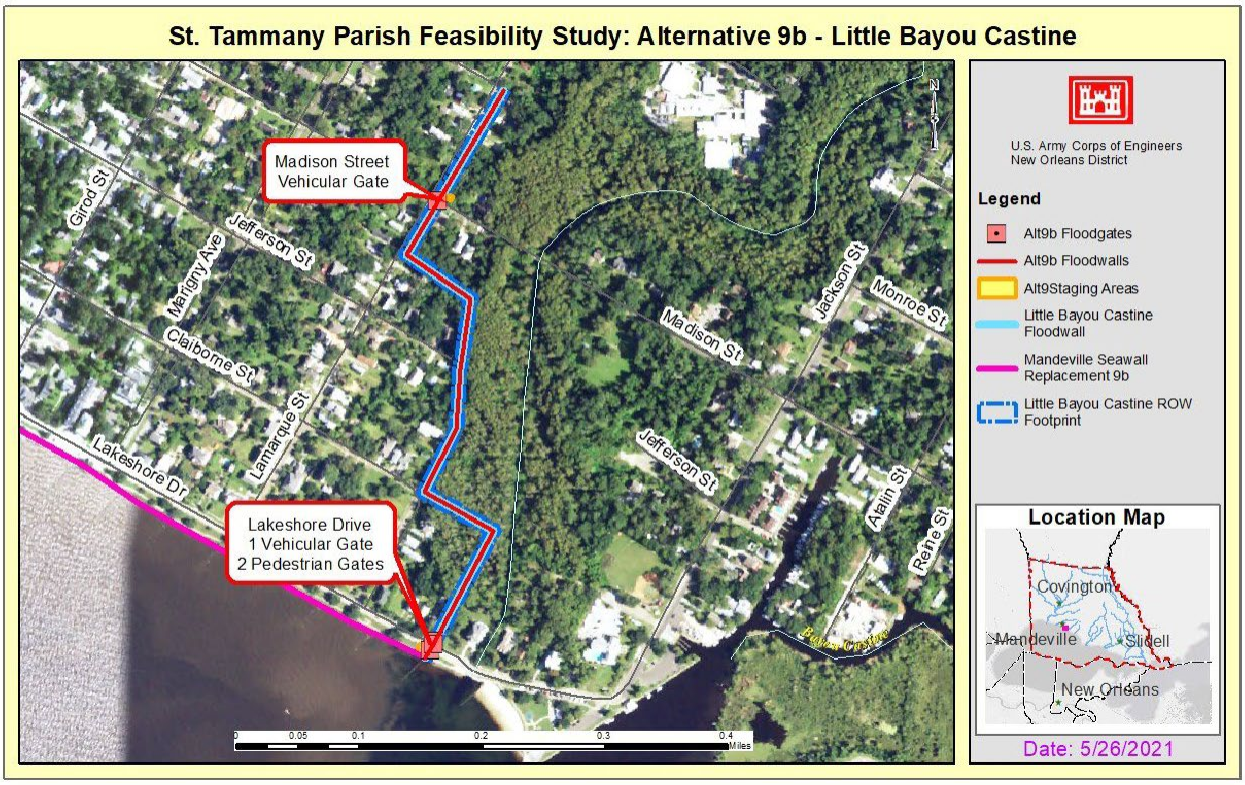


Figure D:1-31. Alternative 9b Little Bayou Castine

1.3.8.2.1 Description of Alignment

For Alternative 9b, the seawall would be replaced in its entirety on the same alignment as the existing seawall. The new seawall would be parallel to and on the south side of Lakeshore Drive; on the west side it would continue on the Galvez Canal to North Street, and the new seawall will end on the west side of Little Bayou Castine where it would connect to the new floodwall on the bayou. This new floodwall would continue along the Little Bayou Castine and would end on Monroe and Lamarque Streets.

1.3.8.2.2 Floodwall Elevation and Location

For Elevation 7.3 feet NAVD 88, the new seawall would be approximately 1.5 miles long (7,703 feet). The new floodwall in Galvez Canal would be at elevation 7.3 feet NAVD 88 and 0.3 miles (1,740 feet) long. The new floodwall at Little Bayou Castine would be at elevation 7.3 feet NAVD 88 and 0.64 miles (3,400 feet) long. Total floodwall would be approximately 12,900 feet.

1.3.8.2.3 Typical Floodwall Section

The typical I-wall section for the new seawall would consist of a 30-foot-deep steel PZ sheet pile with a concrete cap. The new seawall would impact 10.3 acres of construction area. The typical floodwall at Little Bayou Castine would be a combination of T-wall and I–wall. The floodwall at Galvez Canal would be an I-wall. The existing seawall would be demolished prior to the construction of the new seawall. It was assumed that, for the existing seawall, the concrete cap would be 3 feet by 1.5 feet and the length of the wall would be 7,500 feet. The existing vinyl sheet pile at Galvez Canal (length of 2,050 feet) would be demolished and would be replaced with a new seawall. The design of the floodwalls, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.8.2.4 Pump Stations and Floodgates

A pump station would be constructed at the lakefront seawall on Girod Street (preliminary estimated capacity of 200 cfs) with a construction area of 0.009 acres. Assumption was 100- foot by 50-foot staging area for the seawall within the riverfront park.

A 500 cfs pump station and 20-foot sluicegate would be constructed at Ravine Aux Coquilles at the lakefront (construction area is 2 acres). There would be no floodgates on Galvez Canal for elevation 7.3 feet NAVD 88 of the floodwall.

1.3.8.2.5 Vehicular Floodgates and Ramps

At Little Bayou Castine, there would be four vehicular floodgates from south to north, there would be a 30 feet vehicular floodgate at Madison Street, one 75-feet vehicular floodgate and two 10-feet pedestrian floodgates at Lakeshore Drive. There would be designated staging areas adjacent to the construction areas for all the floodgates.

Note that for Alternative 9b, there would be no pump station or floodgate at Little Bayou Castine as the risk reduction ends on the west side of this bayou. There would be no new floodwalls in the interior of Ravine Aux Coquilles for this alternative.

1.3.8.2.6 Structural Assumptions for Alternative 9b

1.) Pump Stations:
   a. Ravine Aux Coquilles Pump Station
      i. Assumed 500 cfs based on hydraulic analysis.
      ii. Assumed pump station to be located at outlet of Ravine Aux Coquilles.
      iii. Assumed pump station to be enclosed in prefab metal building.
   b. Girod Street pump station
      i. Assumed 200 cfs based on hydraulic analysis.

ii. Assumed pump station to be located adjacent to seawall.
iii. Assumed pump station to be enclosed in prefab metal building.

2.) Floodwalls:
a. Little Bayou Castine western floodwall
i. Assumed T-wall with sill at Elevation 0 to extend from seawall to Elevation 5.
ii. Assumed I-wall for alignment elevations from 5 to 7.3.
iii. Assumed alignment to avoid infringing on private property.
iv. Assumed alignment to cross at 2 pedestrian paths, Lakeshore Drive, and Madison Street.
b. Lakefront Seawall:
i. Assumed complete replacement of existing I-wall seawall.
ii. Assumed complete replacement of Galvez Canal seawall.
iii. Assumed Seawall to extend from North Street tie-in to Little Bayou Castine inlet.

3.) Access Floodgates:
a. Vehicle Roller floodgates
i. Assumed vehicle floodgates at intersection of proposed alignment and streets.
b. Pedestrian Roller Floodgates
i. Assumed pedestrian swing floodgates at intersections of proposed alignment and pedestrian paths.

4.) Control Floodgates:
a. Ravine Aux Coquilles Outlet Floodgate
i. Assumed 20-feet sluicegate.

1.3.8.3 Alternative 9c Mandeville Lakefront – 18 ft

Alternative 9c would consists of a series of inlet pumps in conjunction with the new seawall at the lakefront in Mandeville, Louisiana. This alternative would include a pump station at Bayou Ravine Aux Coquilles. For the seawall replacement, elevation of 18 feet (NAVD 88) was analyzed. The elevation to provide 1 percent risk reduction (100-year) in future conditions in the year 2082 (planned project completion year 2032) was analyzed. The 1 percent risk reduction would require the new seawall at elevation 18 feet NAVD 88, according to preliminary New Orleans District (MVN) Engineering Division (ED) Hydraulics, Hydrology and Coastal Branch (HH&C) data. Refer to Figure D:1-32, D:1-33, and D:1-34.

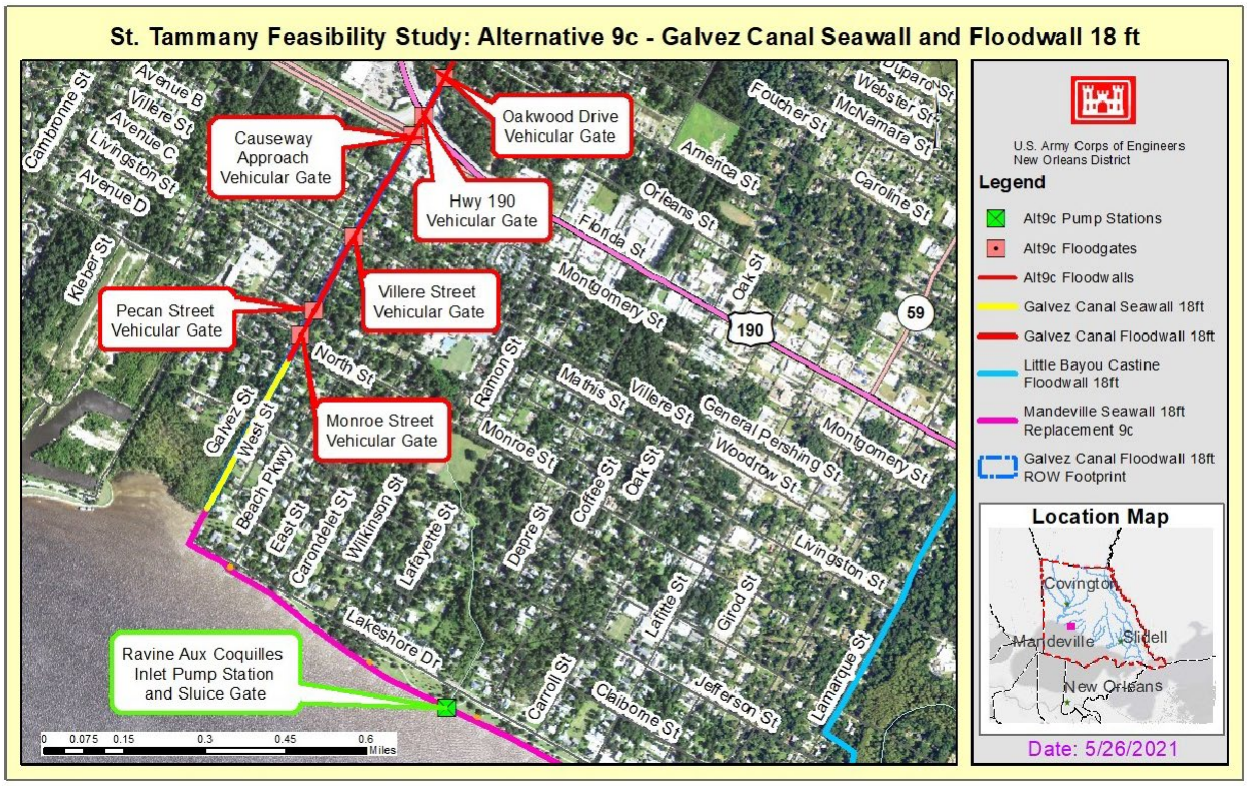


Figure D:1-32. Alternative 9c Galvez Canal Seawall and Floodwall 18 Feet

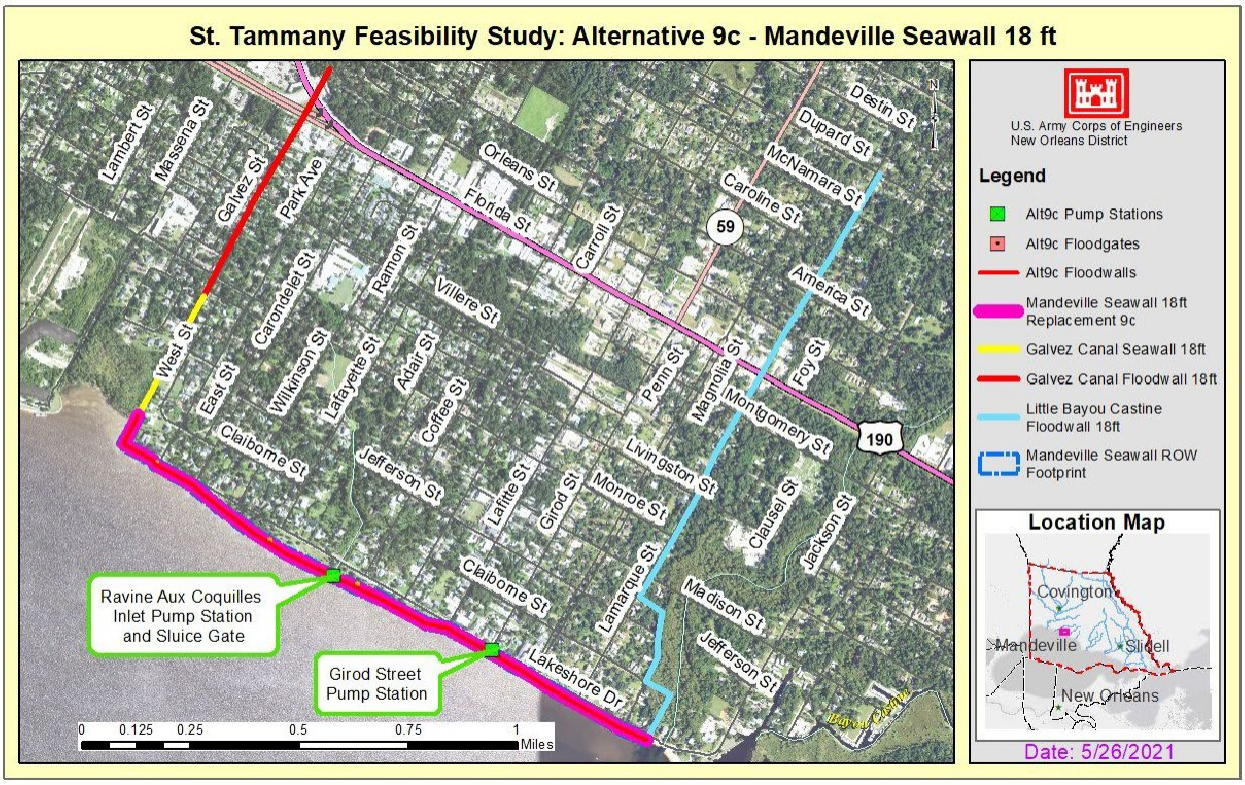


Figure D:1-33. Alternative 9c Mandeville Seawall 18 Feet

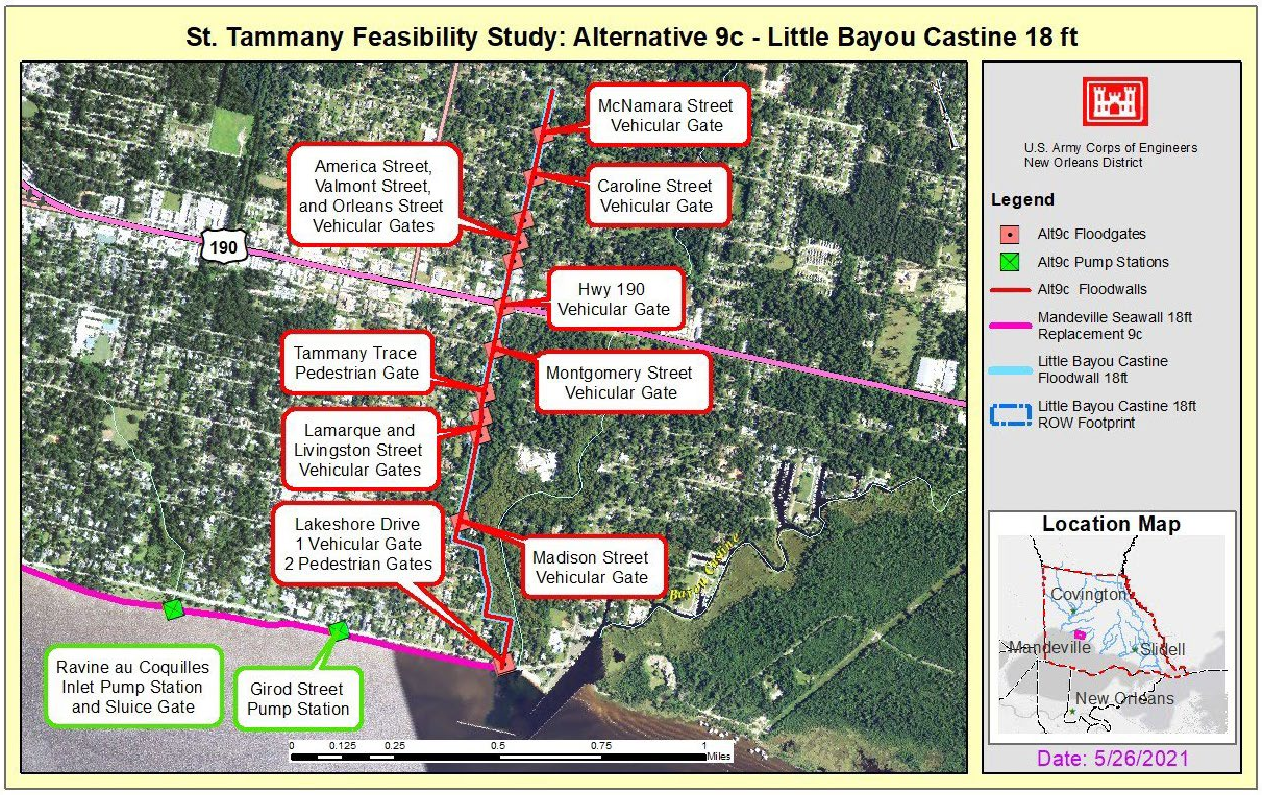


Figure D:1-34. Alternative 9c Little Bayou Castine 18 Feet

1.3.8.3.1 Description of Alignment

For Alternative 9c, the seawall would be replaced in its entirety on the same location as the existing seawall. The seawall would be parallel to and on the south side of Lakeshore Drive. On the west side, it would continue on the Galvez Canal to North Street, where it would transition into a floodwall. The new floodwall (T-wall) on Galvez Canal would cross the Causeway Approach, where the T-wall would transition into an I-wall and would end at Oakwood Drive. On the east side of Lakeshore Drive, the new seawall would end on the west side of Little Bayou Castine, where it would connect to the floodwall on the bayou. This floodwall (T-wall) would continue along the Little Bayou Castine, continue on Monroe and Lamarque Streets, and would cross U.S. Highway 190, where it would transition into an I-wall and would end on Dupard and Lamarque Streets.

1.3.8.3.2 Floodwall Elevation and Location

The new seawall would be approximately 1.8 miles long (9,600 feet) and would be built to elevation 18 feet NAVD 88. The new floodwall in Galvez Canal would be at elevation 18 feet NAVD 88 and would be 0.5 miles (2,700 feet) long. The new floodwall at Little Bayou Castine would be at elevation 18 feet NAVD 88 and would be 1.7 miles (9,000 feet) long. The total floodwall length would be approximately 21,000 feet.

1.3.8.3.3 Typical Floodwall Section

The typical T-wall section for the seawall would consist of a 30 feet deep steel PZ sheet pile with a concrete cap. The foundation would be pile-founded. The new seawall would impact 10.3 acres of construction area. The typical floodwall at Little Bayou Castine would be a combination of T-wall and I–wall. The floodwall at Galvez Canal would be a combination of a T-wall and I-wall. The existing seawall would be demolished prior to the construction of the new seawall. It was assumed that, for the existing seawall, the concrete cap is 3 feet by 1.5 feet and the length of the wall is 7,500 feet. The existing vinyl sheet pile at Galvez Canal (length of 2,050 feet) would be demolished and would be replaced with the seawall. The design of the new floodwalls, including the foundation, is subject to change once detailed geotechnical investigations are conducted.

1.3.8.3.4 Pump Stations and Floodgates

A pump station would be constructed at the lakefront seawall on Girod Street (preliminary estimated capacity of 450 cfs) with a construction area of 0.009 acres. Assume a 100-feet-by-50-feet staging area for seawall within the riverfront park.

A 500 cfs pump station complex with a 20-foot sluicegate would be constructed at Ravine Aux Coquilles at the lakefront (construction area would be 2 acres).

1.3.8.3.5 Vehicular Floodgates and Ramps

At Galvez Canal for the 18 feet NAVD 88 floodwall, there would be six 30-feet vehicular floodgates, from south to north: Monroe Street, Pecan Street, Villere Street, East Causeway Approach, LA Highway 190, and Oakwood Drive.

At Little Bayou Castine, there would 11 vehicular floodgates and three pedestrian floodgates, starting from south to north as follows:

- 10-foot Lakefront Drive Pedestrian Floodgate #1,
- 75-foot Lakeshore Drive,
- 10-foot Lakeshore Drive Pedestrian Floodgate #2,
- 30-foot Madison Street,
- 30 feet Livingston Street,
- 30-foot Lamarque Street,
- 30-foot Tammany Trace Pedestrian Floodgate,
- 30-foot Montgomery Street,
- 60-foot US Highway 190/Florida Street,
- 30-feet Orleans Street,
- 30-foot Valmont Street,
- 30-foot America Street,
- 30-foot Caroline Street, and
- 30-foot McNamara Street.

There would be designated staging areas adjacent to the construction areas for all floodgates. Note that on the north side of U.S. Highway 190, there would be floodgates at alternate street crossings to reduce the total number of floodgates. This plan means that Orleans Street and Caroline Street would not cross the floodwall and two additional floodgates may be needed. Note that for Alternative 9c, there would be no pump station or floodgate at Little Bayou Castine as the risk reduction would end on the west side of this bayou. There would be no floodwalls in the interior of Ravine Aux Coquilles for this alternative.

1.3.8.3.6 Structural Assumptions for Alternative 9c

1.) Pump Stations:
   a. Ravine Aux Coquilles Pump Station
      i. Assumed 500 cfs based on hydraulic analysis.
      ii. Assumed pump station to be located at outlet of Ravine Aux Coquilles.
      iii. Assumed pump station to be enclosed in prefab metal building.
   b. Girod Street Pump Station
      i. Assumed 400 cfs based on hydraulic analysis.
      ii. Assumed pump station to be located adjacent to seawall.
      iii. Assumed pump station to be enclosed in prefab metal building.

2.) Floodwalls:
   a. Little Bayou Castine western floodwall
      i. Assumed T-wall with sill at elevation 0 to extend from seawall to elevation 15.
      ii. Assumed I-wall for alignment elevations from 15 to 18.
      iii. Assumed alignment to avoid infringing on private property.
      iv. Assumed alignment to cross at two pedestrian paths, Lakeshore Drive, Madison Street and all streets and pedestrian paths up Lamarque Street to and excluding Dupard Street.
   b. Lakefront Seawall
      i. Assumed complete replacement of existing I-wall seawall.
      ii. Assumed complete replacement of Galvez Canal seawall.
      iii. Assumed Seawall to extend from North Street tie-in to Little Bayou Castine inlet.
   c. Galvez Canal seawall extension
      i. Assumed T-wall with sill at Elevation 0 to extend from seawall to Elevation 15.
      ii. Assumed I-wall for alignment elevations from 15 to 18.
      iii. Assumed alignment to cross all streets along Galvez Street across US Highway 190 to dead-end at Oakwood Drive.

3.) Access Floodgates:
   a. Vehicle Floodgates
      i. Assumed vehicle roller floodgates at intersection of proposed alignment and streets.
   b. Pedestrian Roller Floodgates
      i. Assumed pedestrian swing floodgates at intersections of proposed alignment and pedestrian paths.

4.) Control Floodgates:

a. Ravine Aux Coquilles Outlet Floodgate
   i. Assumed 20 feet sluicegate.

1.4 ASSUMPTIONS FOR EXISTING NFS LEVEES

For the alternative milestone and the feasibility level design, the PDT made the following assumptions regarding the new alignment tying to portions of the NFS existing levees. During PED, USACE will request and evaluate design information from the NFS on the existing levees. If the existing NFS levees are found to be insufficient in design, they will be degraded and a new levee built in the same location to USACE standards and specifications. MVN is not aware of any existing O&M issues or performance concerns on the existing NFS levees.

If the existing NFS levees are found to be constructed to USACE standards, then additional fill will be added to the hydraulic design elevations. The additional fill will be placed to USACE standards and specifications. These portions of the existing NFS levees will become part of the federal levee.

The following design information on the existing levees will be requested to the NFS: As builts, Plans and Specifications that clearly indicate construction methods, including soil properties of embankment material used/borrow material, moisture/density/compaction requirements for embankment, requirements for placement and processing of borrow material, embankment testing requirements and test results, horizontal lift construction with initial and final lifts, requirements of any geotextile fabric used and installation methods of geotextile. USACE will request any construction completion reports documenting any changes to the plans and specifications, or any other approved changes, reason for change, and engineering approval for any changes, like post-construction surveys showing compliance with plans and specifications and validated benchmarks, complete set of design criteria, and assumptions. USACE will request subsurface investigations and lab testing (both before and after construction), soil parameters for embankment fill and foundations (including rationale for selected properties). In addition, USACE will request a complete set of analyses to include, but not limited to slope stability, I-wall analyses (if applicable) showing it meets USACE criteria, theoretical pile capacities with any pile load test results (consolidation, settlement, and seepage). If applicable, USACE will request geotechnical and structural evaluation of the impacts of the buried I-walls within the system, and regulatory permits, and survey data complying with USACE minimum survey standards.

1.5 COST ASSUMPTIONS FOR ALTERNATIVE MILESTONE AND FEASIBILITY LEVEL

The cost analysis performed during alternative milestone and for feasibility level of the study did not include costs for degrading the existing levees. The civil quantities for the levee alignment were calculated using the existing ground elevations obtained from World Imagery. The levee calculations performed for the recommended plan assumed that the existing levees were constructed to USACE standards and that they will remain in place and will be lifted to the new hydraulic design elevations. The levee quantities were provided to Cost Team. The cost analysis used these levee quantities.

For geotechnical investigations, relocations, and surveys during PED, the cost analysis assumed that the whole levee alignment will be designed as a new levee.

The Relocations Team identified, to the best of their ability, all utilities within or near the proposed alignment and assumed the utilities will have to be relocated based on that alignment, regardless of whether or not it was located on an existing NFS levee.

1.6 EXISTING LEVEES

According to the National Levee Database (https://levees.sec.usace.army.mil), there are 4 existing levee systems in St. Tammany Parish:

- Oak Harbor Ring Levee (system ID 1605252002)
- Oak Harbor Ring Levee (system ID 1605252001), referred to as Lakeshore Estate in the study
- Kingspoint Ring Levees (system ID 1605252004), referred to as Kingspoint West in the study
- Voters Road Levee (system ID 300005009100), referred to as Kingspoint East in the study.

1.7 NON-PROJECT SEGMENTS

Screening-level risk assessment will be conducted for non-project segments (interstate highway embankment) that will become part of the federal levee as a result of this project. The project will follow the steps to add the non-project segments to the National Levee Database.

SECTION 2

General Assumptions for Levees for Final Array

The levee cross sections in the Final Array of Alternatives were estimated using engineering judgment based on the limited data available during this study. Throughout this document, they will be referred to as “levees.” The PDT used the Hurricane and Storm Damage Risk Reduction System (HSDRRS) standards, the latest USACE guidance documents (i.e., engineer manuals, engineer circulars, etc.), as well as updated hydraulic modeling techniques as applicable and appropriate for the features presented herein.

The HSDRRS guidelines may be found at:

https://www.mvn.usace.army.mil/Missions/Engineering/Hurricane-Design-Guidelines/Hurricane-Design-Guidelines/

The levee crown elevation would vary from elevation +10 to +15 throughout. Final elevations were determined through hydraulic analysis. The design levee section would be 10-feet crown width and 1V:3H slopes for all levees. Due to the lack of survey and geotechnical data at the time of the proposed design, a 30 percent contingency was added to the levee quantities. The width of the proposed levee at the toes would vary up to 100 feet. With the addition of a clear zone width of 15 feet on each side, the width of the levee footprint would be a maximum 130 feet. Allowing for 10 feet on each side for access, a 150 feet ROW for the levee construction and maintenance would be needed. Levee reaches that require berms will need a wider footprint to include the necessary area for berm construction. A berm section was not included at this phase and project width is subject to change once engineering data is available and analysis is complete. The alignment of the various levee alternatives is subject to change for various reasons ranging from obtaining ROW to potential environmental concerns.

SECTION 3

General Assumptions for Structures for Final Array

For structures of the Final Array (excluding pump stations), the analysis comprises scoping level engineering estimates for the 1 percent future (year 2082) hydraulic design elevation for each structure (See Appendix E: Hydrologic and Hydraulics). No structural superiority was added to the computed design elevations. Structural superiority will be considered in the PED phase.

3.1 PUMP STATIONS

The analysis of the pump stations comprises scoping level engineering estimates for the 1 percent future (year 2082) hydraulic design elevation with an additional 2 feet of structural superiority added to the computed design elevations.

Structural superiority was only considered for design and quantity take-offs for pump stations. These designs and quantities were based off of WSLP pump station designs.

For additional information on the pump station assumptions, refer to USACE WSLP: https://www.mvn.usace.army.mil/About/Projects/BBA-2018/West-Shore-Lake-Pontchartrain/

SECTION 4

4.1 RELOCATIONS FOR FINAL ARRAY

4.1.1 GENERAL

The Fifth Amendment to the Constitution of the United States provides for compensation to be paid for the acquisition of private property for public use. This acquisition of an interest in real estate is necessary for the Federal Government to subordinate such interest in real estate. In publicly owned roads and utility systems, the Federal courts have held that the liability of the United States for such acquisition is the cost of providing substitute facilities where substitute facilities are, in fact, necessary. This is the basis of the facility and utility relocation process. Therefore, it was incumbent that an investigation of the existing public utilities and facilities located within the study area of the Final Array of Alternatives be conducted, while accounting for the current design requirements for the Draft and Optimized TSP. In the event that a facility, utility, cemetery, or town would affect the construction, operation, maintenance, repair, replacement or rehabilitation of a USACE project or study, then the appropriate disposition of the impacted facility must be determined. Some facilities may require either a permanent or temporary physical adjustment or displacement to support project activities, engineering requirements, and operation and maintenance needs.

Investigating, identifying, and verifying public facilities and utilities located for the Final Array within the study area was performed. Database research included the National Pipeline Database, State Online Natural Resources Information System, Louisiana Department of Natural Resources, Hydrographic and Impairment Statistics (HIS), Inc. dataset, and Penwell data. Based on the research and investigations conducted as part of the study effort, multiple facilities or utilities located within the study area of the alternatives for the Final Array would be expected to be impacted. Refer to Annex D2 for maps of the various utilities in the study area of each alternative of the Final Array.

4.1.2 METHODOLOGY

A review of multiple pipeline databases was used to investigate the facilities located within the study area for the Final Array of Alternatives. A site visit was not performed. The facilities that could be potentially impacted by the Final Array of Alternatives were the pipelines, overhead electrical transmission lines, and electrical distribution lines shown in Annex D2.

The impacts on the pipelines assumed the HSDRRS Design Guidelines, dated February 2012, which describes the following as acceptable methods of pipeline crossing: directional drilling, structural elevated support, T-wall construction and direct contact, would be used. It was decided to use the T-wall and direct contact methods (up and over) for this study.

With the direct contact method (up and over), the pipeline owner has the option of placing the pipeline in direct contact with the surface of the newly constructed levee. This would require the owner to relocate the pipeline when the levee is raised because of settlement or change in design grade. The owners must also determine that the pipeline can sustain the settlement and resulting stresses that are associated with it. Slope pavement or other approved armoring methods must be installed over the pipeline throughout the transition area. This method was assumed for single or dual pipelines that have enough space to bypass or re-route up-and-over the new levee design section.

If applicable, the T-wall construction method focuses on passing the pipeline through the T- wall, with the existing pipeline remaining in place. This method consists of constructing a pile-founded, inverted T-wall flanked by a sheet pile wall on either side to provide seepage reduction for flood control. The T-wall is built around the in-situ pipeline. This method is more conducive for pipelines that are approximately 20 feet or less apart and are unable to bypass their ROW on a temporary basis.

Electric transmission lines were assumed to meet the minimum clearance criteria over the proposed levee crossings, which is 22 feet at 50 kV, plus 0.4 inches for every 1.0 kV above 50 kV.

4.1.2.1 RAILROAD CONSIDERATIONS

A key preliminary assumption is that, when a feature crosses or impacts a RR, in some instances there are communications lines and/or electrical lines that service the RR within the RR ROW. These lines can be used for the RR signal lights or track switches. There are also cases where utilities, such as underground water or sewer lines, run under the RR to service nearby buildings. Another preliminary assumption is that all utilities servicing the Dellwood Pump Station do not run under the RR, so there could be communication lines or electric lines near the Norfolk Southern Railway Corporation RR tracks. Further refinement of the utility investigation will be needed.

4.1.3 CONSIDERATIONS FOR THE UTILITY CORRIDOR

CLECO Corporate Holdings, LLC has ROW use requirements pertaining to USACE work around their existing utility lines on the northeast corner of the floodwall alignment that would have to be met to provide clearance for construction activities (i.e., pile driving).

4.1.4 RESULTS

Relocation costs for the Final Array are provided in Table D:4-1. The results of the facility relocations investigations are shown in Tables D:4-2 through D:4-5, which includes a description of the only facilities located within the study area for the respective Final Array of alternatives. Refer to Annex D5 for additional cost information.

Table D:4-1. Relocation Costs for Final Array

Alternative	**Relocations Cost- 2020 Dollars (Includes 28% contingency)**
4a	$25,860,000
4a.1	$18,302,000
4b	$13,323,000
5	$933,000
6a	$16,000
6b	$16,000
6c	$887,000

Table D:4-2. Alternative 4a - Bayou Lacombe Levee

Utility Owner	Utility Type	Method of Relocation
Kinder Morgan	Natural Gas 26-inch pipeline	Up and Over Pipeline Relocation
Kinder Morgan	Natural Gas 24-inch pipeline	Up and Over Pipeline Relocation
Kinder Morgan	Natural Gas 20-inch pipeline	Up and Over Pipeline Relocation
Southern Natural Gas	Natural Gas 20-inch pipeline	Up and Over Pipeline Relocation
Southern Natural Gas	Natural Gas 24-inch pipeline	Up and Over Pipeline Relocation
Ferrier Estates Street	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Monique Street	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Dalmas Street	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Pontchartrain Drive #1	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
24th Street	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Pontchartrain Drive #2	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Barringer Road #1	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Barringer Road #2	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Barringer Road #3	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Barringer Road #4	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation
Lake Road/LA Highway 434	12 ft. Roadway w/ 4 ft. shoulders w/ Distribution powerlines, 6-inch waterline, 6 - inch sewerline on one side	Up and Over Roadway Relocation

Assumptions for Table D:4-2:

- Assumed that proposed protection would be 10-foot-wide levee crown, side slopes 1V:3H and Elevation 12.5 feet.

- 500 feet of each utility would be affected by the alternative to be finalized once USACE meets with utility owner during preconstruction engineering and design (PED).

Table D:4-3. Alternative 4b - Combined Levee from Lacombe to West Slidell

Utility Owner	Utility Type	Method of Relocation	Proposed Effective Length of Utility
Kinder Morgan	Natural Gas 26-inch pipeline	Up and Over Pipeline Relocation	500 ft
Kinder Morgan	Natural Gas 24-inch pipeline	Up and Over Pipeline Relocation	500 ft
Kinder Morgan	Natural Gas 20-inch pipeline	Up and Over Pipeline Relocation	500 ft
Southern Natural Gas	Natural Gas 20-inch pipeline	Up and Over Pipeline Relocation	500 ft
Southern Natural Gas	Natural Gas 24-inch pipeline	Up and Over Pipeline Relocation	500 ft
ExxonMobil	Natural Gas 16-inch pipeline	Up and Over Pipeline Relocation	500 ft
Pontchartrain Drive #2	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation	1,500 ft
Barringer Road #1	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation	1,500 ft
Barringer Road #2	12 ft. Roadway w/ distribution power poles & lines, 6-inch waterline, 6-inch sewerline on one side	Up and Over Roadway Relocation	1,500 ft
Lake Road/LA Highway 434	12 ft. Roadway w/ 4 ft. shoulders w/ Distribution powerlines, 6-inch waterline, 6 - inch sewerline on one side	Up and Over Roadway Relocation	1,500 ft

Assumption for Table D:4-3:

- Assumed that the levee would have a 10-foot-wide crown, side slopes 1V:3H and crown elevation of 12.5 feet.

Table D:4-4. Alternative 5 - Bayou Liberty/Bayou Vincent/Bayou Bonfouca

Utility Owner	Type/Size of Utility	Method of Relocation	Proposed Effective Length of Utility
ExxonMobil	Natural Gas 16-inch pipeline	Up and Over Pipeline Relocation (Levee)	500 ft
Boardwalk	Natural Gas 6-inch pipeline	Pipeline Protection (Clear and Snag Bayou Liberty Channel)	500 ft
Boardwalk	Natural Gas 6-inch pipeline	Pipeline Protection (Excavation on Either Side of Pipelines)	500 ft
Boardwalk	Natural Gas 6-inch pipeline (assumed)	Pipeline Protection (Excavation on Either Side of Pipelines)	500 ft

Assumption for Table D:4-4:

- Assumed that the levee would have a 10-foot-wide levee crown, side slopes 1V:3H and Elevation 12.5 feet.

Table D:4-5. Alternative 6 - South Slidell Levee

Utility Owner	Type/Size of Utility	Method of Relocation
CLECO Corporate Holdings, LLC	Transmission Lines - 240KV	De-energizing Powerlines

Assumption for Table D:4-5:

- Assumed that levee would have a 10-foot-wide levee crown, side slopes 1V:3H and Elevation 12.5 feet.

4.1.5 PIPELINE OWNERS

There are multiple pipelines within the study area of the alternatives mentioned previously. These pipelines cross access corridors or run parallel to the proposed features and their alignments. Refer to Tables D:4-2 through D:4-5, as well as Annex D2, for more information.

4.1.6 CONCLUSION

Based on the preliminary findings of the relocations investigation, it was determined that the existing pipelines within the area of the Final Array of Alternatives would be impacted, either requiring relocation of the utilities affected, or requiring pipeline protection over the affected utilities during construction. In such situations, USACE would incorporate the relocations process toward compensability and coordinate with utility owners throughout the design and development of the plans and specifications during PED.

SECTION 5

Geotechnical Investigations for Final Array

5.1 BACKGROUND

This section summarizes the limited data available from the preliminary geotechnical design results for the Final Array of alternatives. The geotechnical investigations, associated testing, and geologic profiles was limited. Soil borings were not taken and soil testing was not performed for this study. Soil unit weights and shear strengths were assigned based on USACE geotechnical experience in the region and limited boring information in the vicinity of the Final Array alignments. The results presented are only intended for feasibility level cost estimating purposes and determining the technical feasibility of proposed alternatives.

Earthwork stability analyses, settlement, and lift schedules were developed on levee features and used for elimination of alternatives in TSP identification. Pile capacity analyses and Lane's weighted creep analyses were performed on structural features of alternatives to assist in quantity estimates for Final Array structural pile and sheet pile lengths. Refer to Annex D3.

5.2 GEOLOGY

Neither geologic profiles nor geologic maps could be obtained for the Final Array alternatives. Geological conditions are inferred from the limited boring information obtained from Eustis Engineering LLC St. Tammany Parish job reports. More information on these geotechnical investigations can be found in Section 5.3.

The western part of the alignment designated as West Slidell starts at the Shannon Drive lift gate and terminates at the Norfolk Southern Railway Corporation RR tracks west of U.S. Highway 11 in the vicinity of Dellwood Pump Station in Slidell. These marsh deposits make up the top 2 to 10 feet of the subsurface and are comprised of soft organic clays with some sand layers. Below marsh deposits and in some places at the surface, Pleistocene Prairie Terrace deposits are present. These deposits span at least 40 feet below and are composed of predominantly medium to very stiff clay; however, some borings show an approximately 10-foot-thick sand layer around 20 feet below the ground surface.

As the alignment runs eastward into the area designated as South Slidell, starting at the floodwall parallel to the RR tracks and including the tie-ions with Oak Harbor levee and the Lakeshore Estates levee, the marsh deposits transition into areas of fill deposits overlaying shallow Pleistocene deposits. This is due to a filling system employed to develop the Slidell and Eden Isles areas. The top 5 to 15 feet of fill consists of silt and clay with some sands. A compressed marsh section mat exists under the fill section

depending on how development occurred, and this is underlain by Pleistocene Prairie Terrace Deposits.

A geologic profile should be developed for the project alignment during PED, when a thorough geotechnical investigation is planned to be conducted along the alignment.

5.3 FURNISHED INFORMATION AND SOIL DESIGN

USACE has very limited boring information in St. Tammany Parish. Eustis Engineering LLC furnished USACE with all job reports for projects previously completed by the company in the vicinity. Projects that had boring information were taken and plotted in Google Earth.

Borings of appropriate depth were considered. The closest available projects of levee or structural features that had geotechnical investigations were used to develop parameters. Levees were not broken into individual geotechnical reaches. Sections analyzed were based on hydraulic reaches. Whichever Eustis Engineering LLC project was most appropriate was used to get soil information for development of geotechnical parameters. Parameters were applied uniformly throughout an entire levee or structural feature. Future analyses (during PED) shall take site specific information. More refined parameter selection process should be developed during PED. This could result in more geotechnical reaches needing additional analyses within individual levee features in PED. Many levee features given in the alternatives are several miles long, so a levee feature may require breakdown into multiple reaches requiring checks instead of one per feature as done for the Final Array of Alternatives during feasibility. The lack of subsurface information and the consequential number of assumptions required are noted in the study risk register.

Alternatives that have features requiring geotechnical input are provided below. The Eustis Engineering LLC project number, boring information including number and depth, proximity to feature and assumptions are contained within.

5.3.1 Alternative 4: Bayou Lacombe Levee

The levee alignment consists of approximately 8 miles (39,000 ft) in the City of Lacombe, Louisiana to reduce coastal flooding. The levee extends on the south side of Highway 190 from Shelby Drive to east of Cypress Bayou. There is also a Bayou Lacombe floodgate as part of this alternative.

The closest boring to the Bayou Lacombe alignment is two 40 ft borings taken at the Mandeville Marina as part of Eustis Job No. 09318. This boring is 4.75 miles away from the westernmost section of the Bayou Lacombe alignment and is used for the entirety of the Bayou Lacombe alignment. The closest boring to the Bayou Lacombe floodgate is an 80 ft boring taken at Dellwood Pump Station as part of Eustis Job No. 13965. This boring is 3.7 miles away.

5.3.2 Alternative 5: Bayou Liberty/Bayou Vincent/Bayou Bonfouca and West Slidell Levee

This alternative includes levee with floodgate sections, channel improvements, and detention ponds. The alignment includes an 8.5-mile alignment consisting of approximately 6.5 miles (34,000 ft) of levees and 2 miles (9,400 ft) of floodwall. This alignment is located on the west side of the City of Slidell, Louisiana. The levee extends on the south side of Highway 190 from southwest of Bayou Paquet to west of Highway 11.

The closest boring is an 80 ft boring taken at Dellwood Pump Station as part of Eustis Engineering LLC job No. 13965. This boring is right at the western terminus of the alignment and is used for the entire 8.5-mile alignment.

The same Dellwood pump station boring was used for all the structural features along the West Slidell alignment. The 80 ft boring taken at Dellwood Pump Station boring is closest to the West Slidell sluice gate at about 1 mile away. It is about 2.3 miles away from the Bayou Bonfouca floodgate, about 3.3 miles from the Bayou Liberty area, and about 4 miles from the Bayou Pacquet area.

No parameters were developed for channel improvements or detention ponds for this Final Array alternative.

5.3.3 Alternative 6: South Slidell Storm Surge Risk Reduction

Alternative 6 reduces storm surge risk to Slidell, including Eden Isles. The alignment comprises 20.5 miles of levee/floodwall combination. The alignment will consist of 8.5 miles of levees (44,500 ft), and approximately 6 miles (31,000 ft) of floodwall at Eden Isles and 5.9 miles (30,800 ft) of floodwall within the Slidell levee alignment.

There are geotechnical investigations in the Slidell area and three different Eustis Engineering LLC projects were used for three different levee sections: one for Eden Isles levees, one for CPRA ring levee/Slidell levee segments, and one for Oak Harbor. The closest boring to the Eden Isles levee alignment is an 80 ft boring taken at St Tammany Event Center project as part of Eustis Engineering LLC Job No. 16613. This boring is 0.75 miles away from the eastern side of the Eden Isles alignment. The closest boring to the CPRA/Slidell levee alignment is an 80 ft boring taken 0.20 miles away at the Eden Isles expansion project as part of Eustis Engineering LLC Job No. 10120. Please note that this is closer to the Slidell levee alignment than the Eden Isles levee alignment as currently laid out. The two levee sections had the same overlapping alignment just north of Eden Isles for the Eustis Engineering LLC project versus the study. The Oak Harbor borings were taken from Eustis Engineering Job No. 11044 and make up the southeastern portion of the Oak Harbor alignment. There are nine borings in total that are 40 to 45 feet in depth.

There are seven main structural features in four areas: the South Slidell surge reduction on the western side, the W-14 floodgate to the east, the I-10 gates to the southeast, and the Eden Isles sections to the south.

The same boring at Dellwood Pump Station as part of Eustis Engineering LLC Job No. 13965 was used for the South Slidell Surge reduction wall pile capacities. These boring locations are closer to this feature than the Lacombe/West Slidell wall features. The boring is approximately 60 feet from the alignment's southern end.

The W-14 floodgate and Old Spanish Trail floodgate and any unmarked walls in their vicinity used the Oak Harbor Pump Station borings from Eustis Engineering LLC Job No. 10463.

This has borings ranging from 40 to 100 feet. The geotechnical investigations from Oak Harbor are 1.7 miles away from Old Spanish Trail floodgate and 1.9 miles away from W-14 floodgate.

The I-10 access gates and the Eden Isles features, including the marina gate, use the Event Center borings, which are 60 feet and taken from Eustis Engineering LLC Job No. 16613.

Deeper than 60 feet use the Oak Harbor geotechnical investigation information to inform on pile capacities. The event center boring is approximately 3,500 feet from the I-10 floodgates, 1.5 miles from Marina floodgate and Lakeshore Drive gate and 2.8 miles from the Pontchartrain Drive gate. This is actually just as close as the Eden Isles borings in Eustis Engineering LLC Job No. 10120 and run 10 feet deeper, so the decision was made to use the Event Center boring.

5.3.4 Alternative 7: Eastern Slidell (Pearl River Levee)

Alternative 7 includes a levee, diversion channel and channel improvements to address riverine flooding. The features in this alternative are all separate and combinable and could all be implemented if justified. The overall length of the Pearl River levee is approximately 4.8 miles (25,000 ft). The levee extends from I-59 running east and turns south running along the Pearl River Tributaries to the intersection of the West Pearl River and Gum Bayou.

The closest borings are four 40 ft borings taken at Slidell Memorial Hospital as part of Eustis Engineering LLC Job No. 13418. These borings are 4 miles away from the southern terminus of the Pearl River levee alignment and is used for the entire 4.8 miles. There are four floodwall sections for a total of 0.5 miles (2,750 ft) for this alternative along the Pearl River alignment. The same Memorial Hospital borings from Eustis Engineering LLC Job No. 13418 were also used for structural analyses.

No parameters were developed for channel improvements or diversion channel for this stage of the Final Array.

5.3.5 Alternative 9: Mandeville Lakefront Seawall

Alternative 9 replaces the existing lakefront seawall to elevation 7.3 ft NAVD88 and adds a passive drainage option on Bayou Ravine Aux Coquilles and Little Castine Bayou. Sub alternatives include pump stations at Girod Street and Ravine Aux Coquilles.

There is an existing Eustis Engineering LLC Geotechnical Report for the Mandeville seawall. Eustis Engineering LLC Job No. 12464, which has bulkhead design for three reaches. This report was directly referenced to inform for the array.

5.4 METHODOLOGY AND ASSUMPTIONS

HSDRRS design criteria was used as a reference to establish study design criteria. For construction design development in PED, a full geotechnical exploration plan shall be developed with investigations taken at every 500 feet off alignment centerline with borings alternating between 5-inch continuous Shelby tube borings (undisturbed) and 3-inch Shelby tube borings (general type) or cone penetrometer tests (CPT). A comprehensive development of site-specific geotechnical parameters shall be developed based on testing and analysis from these explorations. Based on site specific geotechnical parameter development, the project areas across the alignment would be further refined resulting in division into numerous reaches based on subsurface profiles, topography, and design elevations.

Stability and seepage analysis follows HSDRRS guidelines for factor of safety requirements. Levees were analyzed for 2032 elevation with some overbuild to account for settlement.

Study scope only includes an evaluation of Q-case (i.e., undrained) parameters for top of levee, 2032 design elevation, and low water cases. S-case (i.e. drained) analysis were also performed for low water cases. The same cases were checked for 2082 elevations with some overbuild to inform future right of way (ROW) considerations.

For structures, HSDRRS criteria was followed to create pile capacity curves for 12x73 H- piles, 14x74 H-piles, and 18-inch pipe piles. These were used to help inform pile lengths for Final Array quantity estimates. Lane’s weighted creep analyses were performed assuming water loads to the top of the wall for 2082 design elevations. These were used to help inform sheet pile lengths for Final Array quantity.

5.4.1 DESIGN INFORMATION

A typical levee section was developed and analyzed to evaluate stability and settlement. The typical section consisted of a 10-foot crown with 1V:3H slopes. Stability was checked at the highest 2032 design elevation. With more site-specific data and application of a full HSDRRS analysis, study designs may no longer be feasible and stability berms, reinforcement geotextile, or deep-soil mixing may be necessary to meet applicable criteria.

Settlement was checked on the typical levee section defined herein with overbuild that was 2.5 feet above the 2032 design elevation and plotted on a 50-year gradient line from 2032 to 2082 design elevations.

Soil test information taken from the borings defined in Section 5.3 were used to create stability analysis parameters. Applicable test information can be found in Annex D3 (section 2). Seven different strength lines were developed for the various alignment features (floodgates/levees) in different alternatives. Developed strength lines were

plotted based on data acquired from unconfined compression tests (UCT) and one-point unconsolidated, undrained, overburden tests taken from the borings. Borings are 3-inch as they were not taken for USACE specific projects. Please note, for typical USACE risk reduction project under design for construction in PED, 5-inch undisturbed borings would be taken and three- point unconsolidated undrained (Q) triaxial compression tests would be performed.

Atterberg limit tests were also performed from Eustis selected boring samples. Boring locations can be found in the Table D:5-1. Note that Eustis Engineering LLC reports had one set of coordinates per job, therefore jobs having more than one boring have the same coordinates. Available Eustis boring information can be found in Section 2 of Annex D3.

Table D:5-1. Boring Information for the Final Array of Alternatives

Eustis Job No. 09318 Used for Bayou Lacombe				
Boring No.	**Latitude**	**Longitude**	**Ground EL**	**Depth**
B-1 (EE 09318)	30° 20’ 59.68” N	90° 3’ 33.02” W	NA	40’
B-2 (EE09318)	30° 20’ 59.68” N	90° 3’ 33.02” W	NA	40’
Eustis Job No. 13965 Used for West Slidell Levee, Bayou Lacombe Floodgate, Bayou Lacombe Floodgate, Bayou Bonfouca Floodgate, Bayou Liberty Floodgate, Bayou Vincent Floodgate, Bayou Pacquet Floodgate, and South Slidell Surge Reduction Wall				
Boring No.	**Latitude**	**Longitude**	**Ground EL**	**Depth**
B-1 (EE 13965)	30° 15’ 15.73” N	89° 47’ 18.18” W	NA	80’
Eustis Job No. 16613 + 10463 from -54 to -100 Used for Eden Isles Levee, Eden Isles Marina Gate, Eden Isles South Floodwall, Eden Isles Southwest Floodwall, and Eden Isles West Floodwall				
Boring No.	**Latitude**	**Longitude**	**Ground EL**	**Depth**
B-1 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	1.6	50’
B-2 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	2.15	100’
B-3 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	5	50’
B-27 (EE 16613)	30° 13’ 4.89” N	89° 46’ 18.33” W	NA	60’
B-28 (EE 16613)	30° 13’ 4.89” N	89° 46’ 18.33” W	NA	60’
B-29 (EE 16613)	30° 13’ 4.89” N	89° 46’ 18.33” W	NA	60’
B-30 (EE 16613)	30° 13’ 4.89” N	89° 46’ 18.33” W	NA	60’
Eustis Job No. 11044 Used for Oak Harbor Levee				
B-1 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	-1.5	40’
B-2 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	1	40’

B-3 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	6.78	45’
B-4 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	0.5	40’
B-5 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	-1.5	40’
B-6 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	5.55	45’
B-7 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	-1.5	40’
B-8 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	7.17	45’
B-9 (EE 11044)	30° 12’ 50.07” N	89° 45’ 58.38” W	1.25	40’
Eustis Job No.13418 Used for Pearl River Levee and Pearl River Floodgate				
B-1 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
B-2 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
B-3 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
A-1 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	10’
A-2 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	10’
Eustis Job No. 10120 Used for South Slidell Levee				
B-1 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-2 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-3 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-4 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-5 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-6 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-7 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-8 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-9 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-10 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-11 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-12 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-13 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-14 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
sectB-15 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-16 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-17 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-18 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-19 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-20 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
Eustis Job No. 10463 Used for W-14 Floodgate, Old Spanish Trail Floodgate				

Boring No.	Latitude	Longitude	Ground EL	Depth
B-1 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	1.6	50’
B-2 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	2.15	100’
B-3 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	5	50’

No settlement test data was available, so correlations which use water contents from the strength and compression tests and the liquid limits from the Atterberg tests were taken in some areas.

The initial void ratio (or e0) was calculated by multiplying the water content results from the strength tests by the specific gravity and assuming the soil is saturated. The specific gravity was assumed to be 2.7. The resulting equation is:

$$\mathrm{E0} = \mathrm{wGs/S\ or\ 2.7w/100}$$

The compression index (or Cc) was calculated as a function of the in-situ water content. For fat and lean clays (CH and CL) which all clays in analysis are classified as such, which is typical for southeast Louisiana. The Cc correlation data points were based on a Plasticity Index correlation found in “Correlation of Compression Index and Soil Properties of New Orleans Area Clays,” Table 16, dated 04 September 2011. The resulting equation is:

$$\mathrm{Cc} = 0.017\mathrm{w} - 0.299$$

Recompression index values were chosen as 20% of the Compression Index (or Cr) as recommended in the “Correlation of Compression Index and Soil Properties of New Orleans Area Clays” report. All soil was assumed as normally consolidated for this study, so the overconsolidation ratio (or OCR) is assumed as 1. This is a conservative assumption as OCR values are typically overconsolidated near the surface resulting in lower settlement estimates. Settlement analysis unit weights are obtained from the same design values within strength tests of the stability parameters.

The Cv correlation data points were based on a water content correlation which can be found in NAVFAC DM-7. 1, and is included as Figure D:5-1 below. Since the correlation is only provided in graphical form in the NAVFAC DM, an equation was estimated for the CV correlation using Microsoft Excel. The image was first imported into Microsoft Excel and points were manually added to match the CV correlation line. Next, a trend line was added to these points and the power function option was selected. The resulting equation is provided below above Figure D:5-1. In areas where no liquid limit values are available, values between 10 and 50 are assumed, which is in the Settle/3D recommended value range and among the high average of tested values from the available Atterberg test information. CV and CVr values were set to be equal to each other in the analysis.

$$\mathrm{Cv} = 9809.9\mathrm{LL}^{-2.847}$$

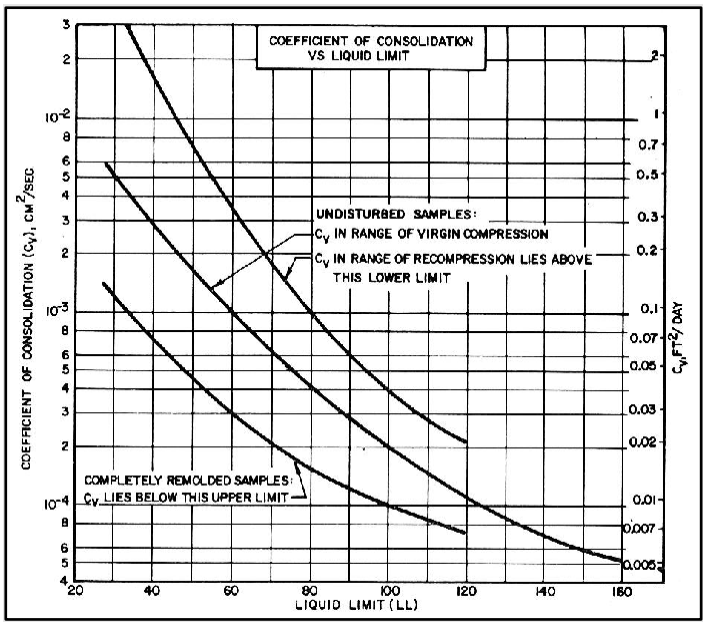


Figure D:5-1. Approximate Correlations for Consolidation Characteristics of Silts and Clays

Note: Figure D:5-1 is Figure 4 from the NAVFAC DM, 7.1-144.

Design parameters and strength plots can be found in Section 3 of Annex D3.

5.5 STABILITY ANALYSIS

Stability of earthen levees was analyzed in SLOPE/W version 10.0.0.17401 from the Geostudio Suite of programs using the Spencer Method to determine typical cross sections for cost estimating purposes. Entry-exit searches with tension cracks applied to the driving side were checked for top of levee loading and low water at ground surface. Stability results can be found in Table D:5-2 below.

Table D:5-2. Stability Results for Final Array of Alternatives

Alternative	Feature	2032 Design Elevation Analyzed	Top of Levee FoS	Low Water FoS
Alternative 4	Bayou Lacombe	+12.5	3.68	3.69
Alternative 5	West Slidell Levee Section	+14.5	1.75	1.81
Alternative 6	South Slidell Levee Section	+15.0	2.33	2.35
Alternative 6	Eden Isles Levee	+14.0	1.87	2.49
Alternative 6	Oak Harbor Levee	+13.5	2.58	2.57
Alternative 7	Pearl River Levee	+15.0	2.09	1.57
Alternative Berm Section	Alignment West of Dellwood Pump Station	+14.5	1.40	1.40

5.6 SETTLEMENT ANALYSIS

Settle3D Version 4.013 by Rocscience Inc. was used for settlement analysis. The typical levee section defined herein was used to model soil loading. No settlement test data was available; therefore correlations were utilized from the test data. The lift schedules were created with a family of settlement curves based on CEMVN's experience with soft soils in southeast Louisiana. Lift schedules were created for each levee feature except for the Pearl River alignment in which only settlement was calculated. The settlement output can be found in Section 5 of Annex D3. The family of curves and lift schedule can be found in Section 6 of Annex D3.

Alternative 4: Bayou Lacombe Levee

The preliminary design elevation is 12.5 ft NAVD88 for the levee. A general levee section of 15 ft (2.5 ft above the 2032 design elevation) with an assumed ground surface of 3 (taken from terrain raster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The first lift is estimated to elevation +16.0 in 2035 followed by a lift to elevation +16.5 in 2040, elevation +17.0 in 2053, and elevation +17.5 in 2068.

Alternative 5: Bayou Liberty/Bayou Vincent/Bayou Bonfouca and West Slidell Levee

The West Slidell levee section has two different 2032 design elevations. The preliminary design elevation is 13.0 ft NAVD88 for the South Slidell and West Slidell sections and 14.5 ft NAVD88 for the central section. A general levee section of 15.5 ft for the South Slidell and West Slidell section and 17 for the central section with an assumed ground surface of 2 for all of the Slidell Levee (taken from the terrain roster dataset), 1 on 3 slopes and a 10-foot crown was used as a cross section in the initial lift in 2032. South and West Slidell sections have three successive lifts afterwards: elevation +16.5 in 2039, elevation +17.5 in 2057, and elevation +18 in 2068. Central section has three successive lifts also: elevation +18.0 in 2037, elevation +19.0 in 2050, and elevation +20 in 2065.

Alternative 6: South Slidell Storm Surge Risk Reduction.

Alternative 6 has three different levee alignments: the South Slidell alignment, the Eden Isles levees, and the Oak Harbor levees. There are four different preliminary design elevations between them. The preliminary design elevation for the South Slidell levee section is 15.0 ft NAVD88 for the levee. A general levee section of +17.5 ft (2.5 ft above the 2032 design elevation) with an assumed ground surface of 0 (taken from terrain roster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The f lift is estimated to elevation +18.5 in 2038 followed by a lift to elevation +19.5 in 2051, and elevation +20.5 in 2067.

The Eden Isles levee section has two different 2032 design elevations. The preliminary design elevation is +13.5 ft NAVD88 for the southeastern sections and 14.0 ft NAVD88 for the Eastern section. A general levee section of +16.0 ft for the southeastern section and +16.5 ft for the central section with an assumed ground surface of 5 for all of the Slidell Levee (taken from terrain roster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The southeastern section has five successive lifts afterwards: elevation +17.5 in 2037, elevation +19.0 in 2047, elevation +20.5 in 2057, elevation +22 in 2067, and elevation +23 in 2075. The eastern section has three successive lifts: elevation +17.5 in 2042, elevation +18.5 in 2057, and elevation +19 in 2072.

The preliminary design elevation for the Oak Harbor levee section is 13.5 ft for the levee. A general levee section of +16.0 ft (2.5 ft above the 2032 design elevation) with an assumed ground surface of 0 (taken from terrain raster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The first lift is estimated to elevation +17.0 in 2037 followed by a lift to elevation +18.0 in 2047, a lift to elevation +19.0 in 2062, and elevation +19.0 in 2077.

Alternative 7: Pearl River Levee

The preliminary design elevation for the Pearl River levee section is 15.0 ft for the levee. A general levee section of +17.0 ft (2.0 ft above the 2032 design elevation) with an

assumed ground surface of 0 (taken from terrain roster dataset), 1 on 3 slopes and a 10-foot crown was used as a cross section in the initial construction in 2032. No lift was estimated to stay above the design grade in the future.

5.7 SEEPAGE

As previously noted, limited geotechnical information is located on the project alignment. The threat of seepage is highly dependent on the presence of aquifers and material with a granular makeup that allows water to freely flow through or underneath the levee section. Since project scope is large and location, extent, depth and proliferation is largely unknown; the seepage analysis consisted of different iterations to investigate what kind of blanket would be required to meet factor of safety requirements against threats of seepage and piping at levee toes.

Two different embankment sections were checked. A 15-foot embankment section with the largest head differential of 15 feet like in South Slidell or Pearl River and a 9-foot embankment with the smallest head differential of 9 feet like in Bayou Lacombe and Eden Isle. Two different aquifer sizes were checked. The first was a 12-foot aquifer which represented the larger end of sand strata found in geotechnical boring information existing on the northern shores of Lake Pontchartrain. The second was a 20-foot aquifer representing a large stratum of sand encountered in southeast Louisiana.

Seepage parameters were estimated based upon Figure 2-5, “Approximate range in coefficient of permeability of soils and rocks” from EM 1110-2-1901, Seepage Analysis and Control for Dams. Conservative values for sand, silt, and clay were chosen and are shown in Table D:5-3 below. A Ky’/Kx’ anisotropy ratio of 0.25 was used for silt and clay, and an anisotropy ratio of 1.0 was used for sand.

Table D:5-3. Seepage Parameters

MATERIAL	COEFFICIENT OF PERMEABILITY (cm/sec)	COEFFICIENT OF PERMEABILITY (ft/sec)
Sand	5.00E-02	1.64E-03
Silt	6.00E-05	1.97E-06
Clay	5.00E-07	1.64E-08

Analyses aimed to achieve a 1.6 factor of safety and aquifers were adjusted to various elevations until a factor of safety of 1.6 was achieved. Overburden used for calculations was 115 pounds per cubic foot (pcf)for total stress or 52.6 pcf for effective stress. Levee density is constructed 115 pcf and is the low average of densities observed in the first 30 feet of available boring information. Effective steady-state stress analysis was performed on two representative levee sections.

The 9-foot embankment section met minimum factor of safety with a 9-foot clay blanket and the 20-foot embankment section met minimum factor of safety with a 22-foot clay

blanket. Results were identical for the 12-foot aquifer and the 20-foot aquifer. Once a full geotechnical exploration program is developed during PED, special attention should be made to the location of the sand. If it is located less than 9 feet down or overburden appears to trend less than 115 pcf total stress, then a seepage berm may be required. For the larger levee sections with potentially 15 feet of head differential, any sand encountered less than 22 feet down or similar weaker overburden should anticipate the need for a seepage berm. The levees would be constructed of compacted clay and erosion is not expected to occur at the toes.

Seepage results may be found in Section 7 of Annex D3.

5.8 H-PILES AND SHEET PILES

Pile capacities were performed on 12x73 and 14x74 H-piles for quantity estimates for the Final Array. A proprietary spreadsheet was used to calculate pile capacities. Densities were derived from the material tests from the individual Eustis Engineering LLC reports.

5.8.1 Alternative 4: Bayou Lacombe Floodgate

The preliminary design elevation for the Bayou Lacombe floodgate is 14.5 ft. Soil properties such as unit weight, cohesion, and material type were taken from the boring at Dellwood pump station and used for analyses. Pile capacity analyses and Lane's Weighted Creep Analysis were performed to help inform on pile lengths for quantity estimates in the Final Array. Sheetpile length was recommended to be 35 ft.

5.8.2 Alternative 5: Bayou Liberty/Bayou Vincent/Bayou Bonfouca/ Bayou Paquet Structures

The preliminary design elevation for the Bayou Paquet floodgate and the floodwall is 14.5 ft. The preliminary design for the Bayou Paquet and liberty floodwall is 16 ft. The preliminary design for Bayou Liberty is 17 ft. The preliminary design for Bayou Bonfouca was 16.5. All of these structural features also utilized the boring at Dellwood pump station for soil properties such as unit weight, cohesion, and material type. Pile Capacity analyses and Lane's weighted Creep analyses were performed to determine sheet pile lengths and pile lengths for quantity estimates. Sheet pile length was recommended to be 35 ft.

5.8.3 Alternative 6: South Slidell Storm Surge Risk Reduction

Alternative 6 alignment would have approximately 6 miles (31,000 ft) of floodwall at Eden Isles and 5.9 miles (30,800 ft) of floodwall in the Slidell levee alignment. There are 7 main structural features in 4 areas: the South Slidell Surge reduction on the western side, the W-14 floodgate to the east, the I-10 gates to the southeast, and the Eden Isles sections to the south.

The preliminary design elevation for the South Slidell surge reduction floodwall is 14.5 ft. The two I-10 access gates have a design elevation of 16.5. For Eden Isles, the preliminary design elevation of the floodwall varies from 13.5 ft to 21 ft8 depending on the

location. For the Slidell levee, the floodwall elevation will vary from 13.5 ft to 15 ft. Soil properties such as unit weight, cohesion, and material type were taken from the borings from the previously referenced Eustis Engineering LLC Geotechnical reports. Pile Capacity analyses and Lane's weighted Creep analyses were performed to determine sheet pile lengths and pile lengths for quantity estimates. A 20-foot sheet pile length is recommended for the South Slidell surge reduction floodwall. The W-14 floodgate is recommended to have a 25-foot length. The Eden Isles I-10 walls are recommended to have 30-foot-long sheets. The Eden Isles Marina Gate, South, Southwest, and West Structures are recommended to have 35-foot sheets.

5.8.4 Alternative 7: Pearl River Alignment

There are four floodwall sections along the Pearl River alignment totaling 0.5 miles (2,750 ft) for this alternative. The preliminary design elevation for the structures is 19 ft NAVD88. Soil properties such as unit weight, cohesion, and material type were taken from the boring at Slidell Memorial Hospital and used for analyses. Pile Capacity analyses and Lane's weighted Creep analyses were performed to determine sheet pile lengths and pile lengths for quantity estimates. Pile Capacity curves revealed very limited capacity and were not used to estimate quantities. Potentially the information used underestimated capacity at the site. Site specific information will be needed to get a more accurate assessment of capacity in the area. A sheet pile length of 35 feet is recommended for the Pearl River floodwall. Pile capacities and Lane's weighted Creep results can be found in sections 8 and 9 respectively in Annex D3.

5.9 CONCLUSIONS AND RECOMMENDATIONS

Geotechnical Analysis were performed on the various levee and structural features considered within the scope of this feasibility study. Geotechnical data was limited. For many features, the closest geotechnical investigations obtained from Eustis Engineering, LLC were utilized rather than site specific borings. Also, the number of borings available for a given feature were limited. This resulted in using borings outside the alignment utilized to help inform Final Array decisions. Consequently, geotechnical engineering analyses assumptions were necessary to complete preliminary checks. These assumptions are considered acceptable for informing feasibility level alternative viability, however as scope narrows, site specific geotechnical investigations are required with associated geotechnical testing to better inform soil conditions for design and construction during PED.

SECTION 6

Borrow

6.1 INVESTIGATIONS DURING ALTERNATIVE MILESTONE

The PDT investigated potential borrow sites in the vicinity of the St. Tammany Parish. Through the process 34 sites were identified and screened. See Section 4 of Appendix B for documentation of the identification and evaluation of the borrow sites. ED conducted an evaluation to document the geology and geotechnical information on a subset of the 34 locations in St. Tammany Paris sites to help with the screening process. These five sites are referred to as potential borrow locations STP-1, STP-3, STP-5, STP-6, and STP-9 as discussed in this section. Most of these sites had very little subsurface data to base a geologic description on. For example, confident geologic understanding of the subsurface typically is relied on a combination of geologic profiles, borings, and CPT data. More reliable geologic assumptions could be made with additional data consisting of geologic profiles, borings, and CPTs. Potential borrow sites evaluated with respect to geology and geotechnical information for the western side of the study area are shown in Figure D:6-2.

Potential borrow sites evaluated with respect to geology and geotechnical information for the eastern portion of the study area are shown in Figure D:6-2.

In addition to the sites identified in St. Tammany Parish, potential borrow sites were identified in Hancock County, MS (MS-1 and MS-2); these sites have previous geologic and geotechnical investigation and are described in Individual Environmental Report #19 Pre- Approved Contractor Furnished Borrow Material Jefferson, Orleans, St. Bernard, Iberville, and Plaquemines Parishes, Louisiana, and Hancock County, Mississippi and Individual Environmental Report # 23 Pre-Approved Contractor Furnished Borrow Material # 2 St. Bernard, St. Charles, Plaquemines Parishes, Louisiana, and Hancock County, Mississippi.

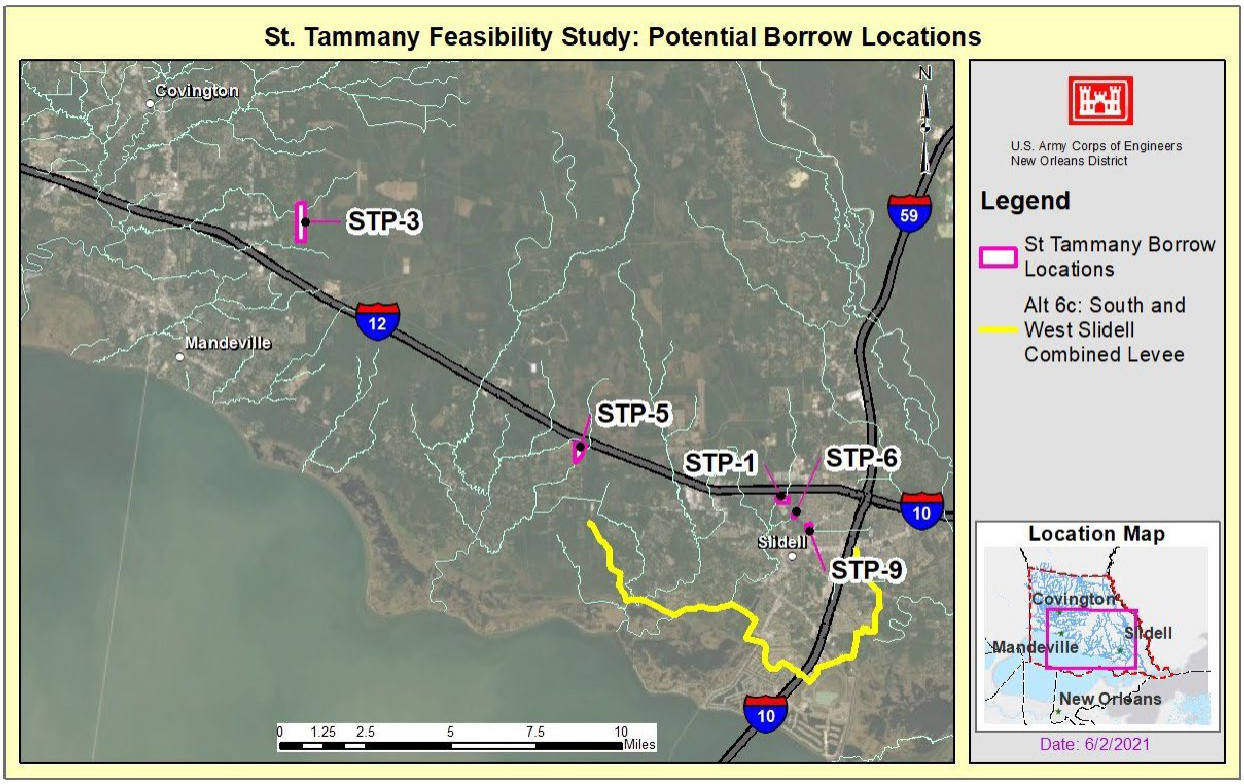


Figure D:6-1. Borrow Locations STP-1, STP-3, STP-5, STP-6, and STP-9

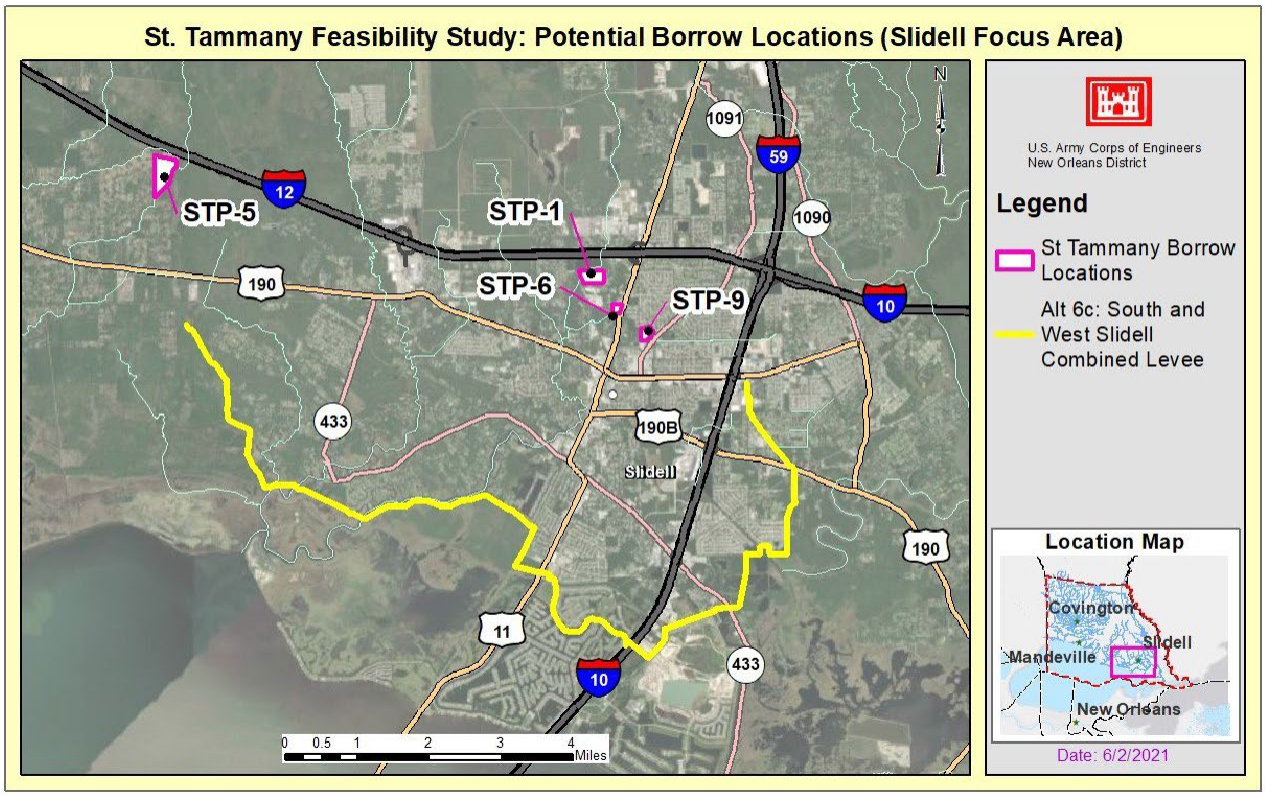


Figure D:6-2. Closer Look at Borrow Locations STP-1, STP-5, STP-6, and STP-9

6.1.1 Potential Borrow Location STP- 1

Geology at location STP-1 cannot be described precisely as there are no boring, CPT, or geologic profile data at the location. A generalization of the location is given based off the closest borings to the location (approximately 0.62 mile away; borings from Location STP- 6). Without any data of the subsurface, accuracy and confidence of subsurface stratigraphy is limited. The top 20 feet of the subsurface are likely to be composed of Pleistocene Prairie Terrace deposits, medium to very stiff lean and fat clay. A 5-foot layer of silt is likely to exist directly below the ground surface. Location STP- 1 is 33.88 acres and the biggest risk to the area is the complete lack of existing data to confirm any of the stratigraphy. However, if the assumed geology is confirmed, the area could potentially serve as a viable embankment clay source. Location STP- 1 is *not actively* being considered due to tree coverage at its surface.

6.1.2 Potential Borrow Location STP- 3

Geology at location STP- 3 cannot be described precisely as there is no boring, CPT, or geologic profile data at the location to base a description after. A generalization of the

location is given based off the closest borings to the location (4 10-ft borings approximately 2.95 miles away). Without any data of the subsurface, accuracy and confidence of subsurface stratigraphy is limited. The top 10 feet of the subsurface is likely to be composed of Pleistocene Prairie Terrace deposits, medium to very stiff lean and fat clay. It is also likely that these Pleistocene Prairie Terrace deposits extend hundreds of feet below the subsurface. Location STP- 3 is 174.91 acres and the biggest risk to the area is the complete lack of data to confirm any of the stratigraphy. However, if the assumed geology is confirmed, the area could potentially serve as a viable embankment clay source.

Additionally, the site appears to be covered in trees. Removal of these trees would add additional costs to the project.

6.1.3 Potential Borrow Location STP- 5

Geology at location STP- 5 was derived from nine borings in the area. These borings show that marsh deposits, depending upon the location, make up the top 2 to 10 feet of the subsurface. These marsh deposits are composed of soft organic clays with some sand layers. Below marsh deposits and in some places at the surface, Pleistocene Prairie Terrace deposits are present. These deposits span at least 40 feet below and are composed of predominantly medium to very stiff clay, however, some borings show an approximately 10- foot-thick sand layer around 5 to 20 feet below the ground surface. Location STP- 5 consists of 72.97 acres and the biggest risk to the area is the 10-feet thick or more sand layer present around 5 to 20 feet below the ground surface in some areas of Location STP- 5.

6.1.4 Potential Borrow Location STP- 6

Geology at location STP- 6 was derived from two 20-foot borings in the location. These borings show that the top 20 feet of the subsurface is composed of Pleistocene Prairie Terrace deposits, medium to very stiff lean and fat clay. A 5-foot layer of silt exists directly below the ground surface. Location STP- 6 consists of 9.83 acres and serves as the best potential source of borrow material. The biggest risk for Location STP- 6 is that the borings show only 20-feet below the ground surface, so it is unknown exactly what is below this layer.

6.1.5 Potential Borrow Location STP- 9

Geology at location STP- 9 was derived from three existing soil borings near the area. Closer to Location STP- 9, borings show that marsh deposits make up the top approximately 10 feet of the subsurface. These marsh deposits are composed of organic lean and fat clay with some silt layers. Below the marsh deposits, beginning around 7 to 10 feet below the ground surface, Pleistocene Prairie Terrace deposits are present. These deposits span up to 60 feet below (in borings closer to location STP- 9) and are composed of predominantly medium to very stiff clay. However, borings indicate that with increasing distance from Location STP- 9, a deep abandoned channel

nears the surface, appearing at a depth of approximately 60 feet below the ground surface just 0.15 miles from the site and only 10 feet below the ground surface just 0.25 miles from the site. This abandoned channel is composed of poorly graded sands and silty sands and is approximately 35 feet thick. There is a possibility the abandoned channel is present at Location STP- 9. Additional boring, CPT, or geologic profile data would confirm exact locations of this abandoned channel layer, but currently, the data is not present. Location STP- 9 consists of 17.44 acres and the biggest risk to the site is the potential of a 35 feet thick sand layer present, although signs indicate that it might be deeper than 60 feet below the ground surface at Location STP- 9.

6.2 FINAL BORROW DETERMINATION

The construction of the Levee and Floodwall System would require approximately 7 million cubic yards of fill or borrow material. Borrow material would come from sites estimated not more than 17 miles of the System alignment. A total of 3,000,000 cubic yards of soil is needed for initial construction and a grand total of 7,239,000 cubic yards is needed over the entire authorized 50-year period to sustain the 1 percent AEP design elevations out to year 2082.

Existing Government borrow sites were not available within the 17 miles distance limitation. Feasibility level borrow site investigations were conducted to confirm that there were available borrow quantities within the vicinity to support the RP decision and evaluate the anticipated impacts associated with the potential borrow sites.

The evaluation of borrow sites led to the identification of three sites in St. Tammany Parish and two sites in Hancock County, Mississippi as potential borrow sources. These sites include land cleared of vegetation and previously investigated HSDRRS borrow sources. Environmental resource assessments were performed on five sites (ST-5, ST-6, ST-9, MS-1 and MS-2) to determine if significant impacts to potentially affected resources in the potential borrow areas. The borrow sites have been previously investigated and partially or fully cleared for Cultural Resources. See IER 19, 23 and 31 for sites MS-1 and MS-2 and SHPO Report number 22-3725, 22-5346 and 22-3151 for the St. Tammany sites. For additional information regarding environmental resource borrow evaluation see Section 5 of the RDIFR-DEIS.

The five (5) potential borrow site options contain approximately 27.3 million cubic yards of borrow where only approximately 7 million cubic yards were estimated to be needed for construction of the RP and follow environmental operating principles to reduce impacts. The potentially affected resources included wetlands, uplands, prime and unique farmland, fisheries, wildlife, threatened and endangered (T&E) species, cultural resources, recreational resources, noise, and aesthetics. The five (5) borrow sites avoid impacts to wetlands and are not expected to require compensatory mitigation. A Phase I ESA will be conducted by the CEMVN on the proposed borrow sites. Any additional

potential borrow sites will require supplemental environmental evaluations in accordance with the NEPA.

The final borrow sources will be selected prior to acquisition and may include borrow material from all sites, from just one of the identified sites or a combination of sites depending on the suitability of the sites. The necessary right of entry and onsite surveys to get the additional information needed for site selection including geologic profiles, borings, and Cone Penetration Test would be obtained.

Transportation routes and mechanisms for the delivery of borrow material have been examined and can be achieved using highways including Interstate-10, Highway 190, Highway 433 and Highway 11. Sensitive areas such as schools and hospital would be avoided. These actions are expected to avoid and minimize transportation, noise and socioeconomic impacts. Staging areas and haul roads would be contained within the borrow site and construction footprints.

The final borrow sites design would include slopes, depths, drainage, and environmental design considerations. Best management practices would be developed and would address the installation of signage, construction fencing and gates, and erosion control. A stormwater pollution prevention plan (SWPPP) would be prepared in accordance with EPA and state regulations. The SWPPP will outline temporary erosion control measures, such as silt fences, retention ponds, and dikes. The construction contract would include permanent erosion control measures, such as turfing and placement of riprap or filter material. For additional information on final borrow determination, refer the Plan Formulation Appendix (Appendix B), See Figure B:4-2 to B:4-6 for locations of final borrow sites.

SECTION 7

Life Safety Risk Assessment

Refer to Annex D4 for the Preliminary Life Safety Risk Assessment.

Refer to Section 10, F-1 Life Safety Annex, in the Economics appendix for the Life Safety Risk Indicator Analysis,"

A potential failure mode analysis was performed and included as Annex-11.

The final life safety risk assessment will be performed via Semi-Quantitative Risk Assessment (SQRA) in PED in accordance with the exemption letter signed by ASA-CW.

SECTION 8
Hydraulics and Hydrology

Refer to Appendix E for Hydraulics and Hydrology analysis.

SECTION 9

Cost Engineering

Refer to Annex D5 for cost analysis for the Final Array of Alternatives.
Refer to appendices D8 to D10 for cost, construction schedule, and Cost Certification of the Recommended Plan.

SECTION 10
RECOMMENDED PLAN

10.1 DESCRIPTION OF THE RECOMMENDED PLAN

Subsequent to the release of the June 2021 Draft Integrated Feasibility Report with Environmental Impact Statement (DIFR-EIS), the PDT conducted additional engineering, economic, and environmental investigations on the separate features of the draft TSP, which included structural measures consisting of the South Slidell and West Slidell levee and floodwall system, the Mile Branch Channel Improvements Measure, and the NS Plan.

Information gathered by the PDT through these additional investigations, together with the consideration of comments received from the public, stakeholders, the U.S. Fish and Wildlife Service, and the National Marine Fisheries Service regarding the levee and floodwall system assisted the PDT in further refining the design of the Draft TSP, which evolved into the Optimized TSP.

Clearing and snagging at Bayou Patassat was included in the Draft TSP but this feature was subsequently screened, so it was not included in the Optimized TSP or the RP. On July 21, 2023, a Revised Draft Integrated Feasibility Report and Environmental Impact Statement (RDIFR-EIS) that contained an Optimized TSP, was released for a concurrent public review. See Section 4.0 of the RDIFR-EIS for additional information.

After review of the comments received on the RDIFR-EIS, the PDT conducted an updated economic analysis on the measures in the Optimized TSP, which included the Mile Branch Channel Improvements (structural FRM) Measure. As a result of this analysis, the cost estimate for this measure was revised. The increases in the cost leading to the screening of Mile Branch was due to two factors: an ATR reviewer's comment and recommendation on the prepared cost estimate for the measure and the incorporation of the cost of the required compensatory mitigation. The ATR reviewer recommended that it is common cost engineering practice to model the third interval in the Cost Risk Analysis (CSRA) for triangular distributions at a lower percentage. The third interval in the risk register for all risk were reduced from 100% to 90%. This increased the contingency on the Mile Branch implementation cost estimate by 11%. The other increase to the cost for the Mile Branch measure was due to the addition of the required compensatory mitigation cost increasing by roughly $4 million.

The new implementation cost for the Mile Branch Channel Improvements Measure far exceeded the amount of damage reduction and was not economically justified.

Consequently, the Mile Branch Channel Improvements Measure was screened and not carried forward into the RP.

The Recommended Plan in the FIFR-EIS includes two measures: the construction (and operation) of approximately 18.5 miles of a levee and floodwall system from West Slidell, LA to South Slidell, LA, and nonstructural (NS) home elevations and floodproofing of non- residential structures in St. Tammany Parish, LA. The NS Plan is further described in Appendix F and Appendix H.

The following sections describe the RP.

Figures D:10-1 and D:10-2 show the RP. The Engineering project descriptions for the levee and floodwall system is included in this Appendix. Costs for the NS Plan portion of the RP are provided in Annex D8 Cost Engineering for the Recommended Plan. Annex D10 contains the costs for the NS and Annex D10 contains the construction schedule for the NS.

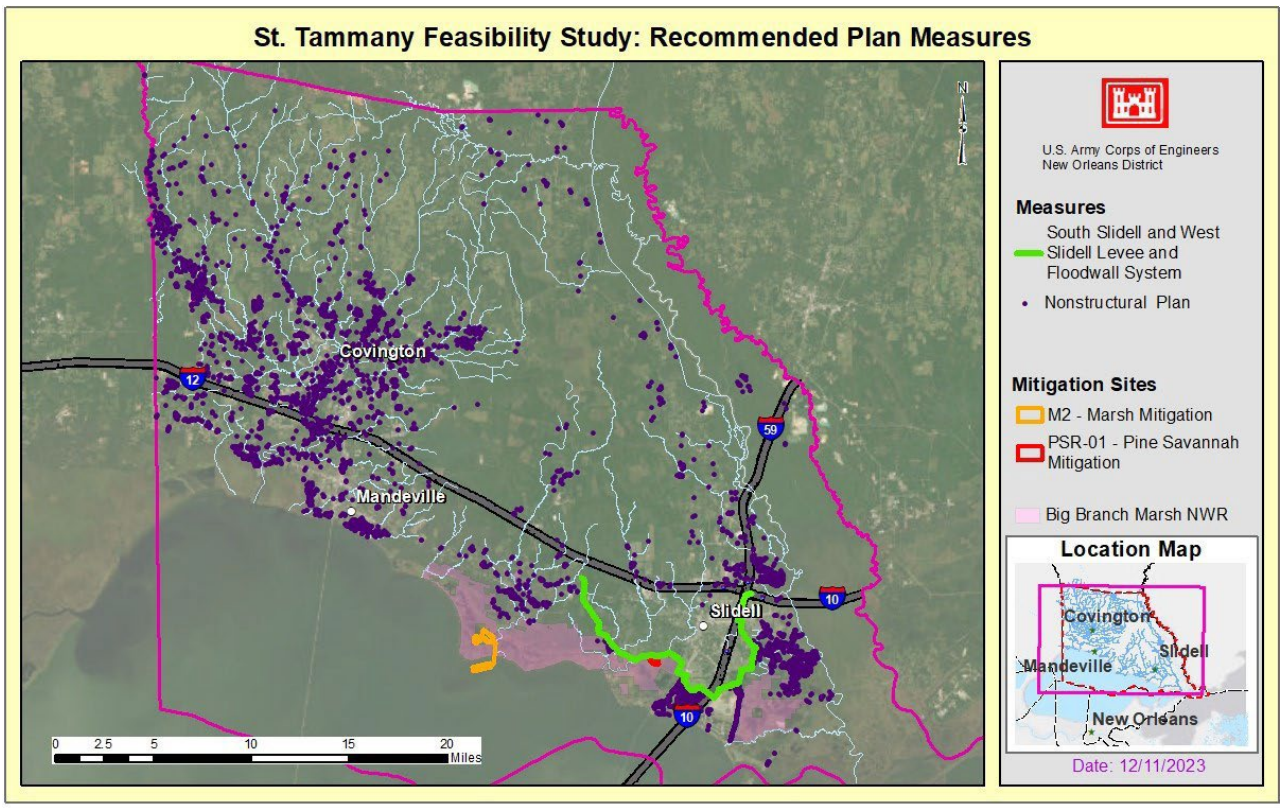


Figure D:10-1. Recommended Plan

10.1.1 RISKS

All geotechnical designs were based on assumptions made about soils in southeast Louisiana as very limited site-specific geotechnical information was available and geotechnical investigations were not included as part of this study. Refer to Section 5 Geotechnical Investigations on Final Array for additional discussion.

There are no surveys available for this area for this study, and no surveys will be conducted during the study phase. The existing elevations used for the hydraulic analysis and design of the RP were obtained from seven sources as described in Appendix E, Section 4.3 Terrain and Land Cover. Designs are based on existing information gathered from reports provided by the non-Federal sponsor (NFS).

Due to the lack of survey and geotechnical data at the time of the proposed design, a 30 percent contingency was added to the levee quantities of the RP.

Field investigations were not funded under study scope, and field investigations would likely have identified additional utilities. Commercial activity continues during the study period and additional utilities may be constructed that may require additional relocations not described in this report.

This section covers the relocations that may be required for implementation of the levee and floodwall system feature of the RP.

10.2 SUMMARY OF LEVEE AND FLOODWALL SYSTEM ALIGNMENT

The RP for the levee and floodwall system consists of a combination of portions of the West Slidell levee alignment proposed in Alternative 5 and the South Slidell levee alignment proposed in Alternative 6. The two alignments would be connected by a new railroad gate across the existing Norfolk Southern Railway Corp. railroad tracks. Additional hydraulic and coastal modeling, analysis by the PDT, and agency and public comments were used to develop the RP.

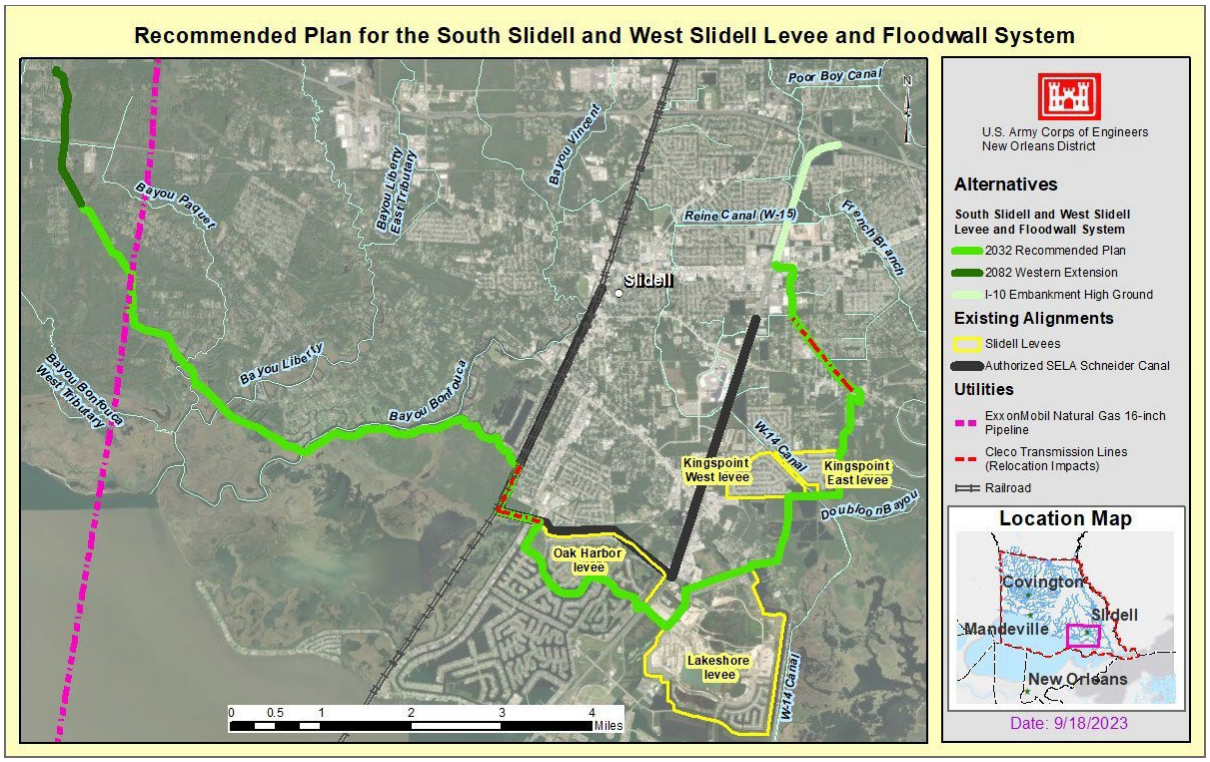


Figure D:10-2. Recommended Plan for the South Slidell and West Slidell Levee and Floodwall System

The RP alignment for the levee and floodwall system consists of a total of approximately 18.5 miles (97,700 feet) of levee and floodwall, which includes approximately 15 miles (79,500 feet) of levees constructed in separate (non-continuous) segments, and approx. 3.5 miles (18,200 feet) of separate (non-continuous) segments of a floodwall. Refer to Figure D:10-2 for levee system alignment. The RP includes pump stations, floodgates, vehicular floodgates, and ramps. Alignment comprises of approx. 7 miles of levee and floodwall in West Slidell and approx. 10 miles of levee and floodwall in South Slidell. The Western High Ground Tie-in for year 2082 accounts for approx. 1.6 miles of the total 18.5 miles of levee and floodwall length. The Western High Ground Tie-in is discussed in Section 10.4.2. Structural components (pump stations, floodgates, vehicular floodgates, and ramps) would be constructed during initial construction.

10.3 GEOTECHNICAL ASSUMPTIONS

All geotechnical designs were based on assumptions made about soils in southeast Louisiana, as no site-specific geotechnical information was available and geotechnical investigations were not funded as part of this study. The closest available geotechnical investigations were used for analyses when available. When not available, analyses were assumed based on general knowledge of soils in the southeast Louisiana area. All estimates for levee sections, levee lifts, and future levee sections are based on engineering judgement with limited data and likely to change in PED when a comprehensive geotechnical exploration plan would be developed and implemented.

The lack of geotechnical information and the number of resulting assumptions required are entered in this study risk register. A full geotechnical exploration program with material testing would be incorporated during PED.

The estimated levee cross sections were determined based on limited subsurface investigations and laboratory test data. Therefore, there is a great deal of uncertainty in the cross sections, and likewise, in the cost estimate. While a 30 percent contingency to levee quantities was applied to the RP project cost to account for such uncertainties, the significant lack of data for a complex project such as this study could result in a final project cost that exceeds the 30 percent contingency to levee quantities.

The magnitude and timing of future levee lifts for the levees in this study includes a great deal of uncertainty given the complete lack of laboratory soils tests that would provide some insight on the amount of predicted future settlement and how long that settlement would take. A better prediction of these long-term settlements and required levee lifts can only be done when additional subsurface investigations and laboratory soils tests are obtained with laboratory analyses conducted and site-specific soil parameters developed.

Refer to Section 10.24 for geotechnical investigations for the RP.

10.4 DESCRIPTION OF LEVEE AND FLOODWALL ALIGNMENT, AND STRUCTURES

All elevations are referenced to NAVD 88 (Geoid 12B), unless otherwise noted.

For the purposes of this study, the following assumptions were made at the beginning of the study:

Authorization - year 2024 PED completed - year 2027

Initial construction completed - year 2032 (base year).

These original assumptions were revised when the construction schedule for the RP was prepared by the Cost team in MVN Engineering. This project description lists the levee hydraulic design elevations for the base year, 2032. Base year plus the period of analysis of 50 years (year 2082) levee design elevations were also used by the PDT for planning purposes. In the final integrated feasibility report and environmental impact statement, year 2082 is the same as “future conditions”. Refer to Section 4 for hydraulic design elevations for year 2082.

The existing elevations used for the analysis of the RP were obtained from the terrain raster dataset. No survey data was obtained at this stage of the study; therefore, a 30 percent contingency was used for the calculation of the borrow quantities for the South Slidell and West Slidell levee alignment.

The RP levee and floodwall system was divided into West Slidell and South Slidell to add clarity to the description.

Table D:10.1 shows a summary of the levee quantities required for the initial construction.

Table D:10.1. Summary: RP Levee Quantities for Initial Construction

Levee Alignment ROW and Levee Quantities Initial Construction (Year 2032)	
WEST SLIDELL	
Permanent ROW	240 acres
Fill Material (includes 30% contingency)	2,007,000 cubic yards
SOUTH SLIDELL	
Permanent ROW	120 acres
Fill Material (includes 30 %contingency)	830 cubic yards**
TOTAL	
Permanent ROW	360 acres
Fill Material (includes 30 % contingency)	3,000,000 cubic yards

**includes quantities for I-10 portion of the alignment and the berm on the north end of the South Slidell alignment

10.4.1 WEST SLIDELL INITIAL CONSTRUCTION

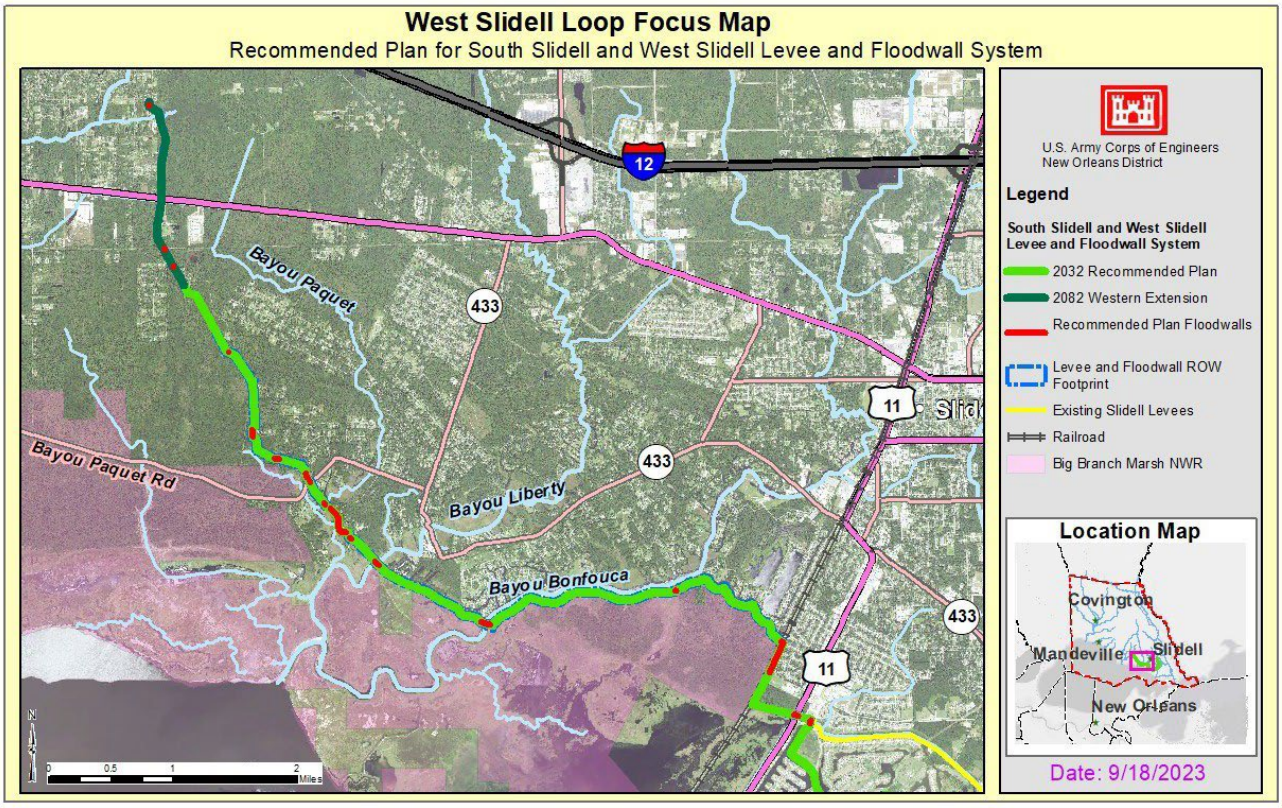


Figure D:10-3. West Slidell Levee and Floodwall System- Recommended Plan Focus with Floodwall Segments

The levee alignment of the RP consists of a levee with several floodwall segments. Refer to Figure D:10-3. The alignment, levee and floodwall segments on the western side, structures, and the design heights of each are described below.

Starting on the western side; the levee construction would commence on the south side of

U.S. Highway 190 and South Tranquility Road, and on the eastern side of Pineridge Road. For the West Slidell portion of the alignment, the levee segments would have a hydraulic design elevation of 13.5 ft (year 2032).

The alignment would run southward and would run on the west side of Tranquility Road (CC Road) and then it would turn in the southeast direction crossing Bayou Paquet Road and would stay on the east side of Bayou Paquet Channel to avoid impact to the Big Branch Marsh NWR. The alignment would cross Bayou Paquet and Bayou Liberty and would continue eastward on the northside of the refuge (BBMNWR). The alignment would cross Bayou Bonfouca and would continue on the south bank of the bayou

(northern side of the refuge) until reaching the Norfolk Southern Railway Corp. railroad tracks west of U.S. Highway 11 in the vicinity of Dellwood Pump Station in Slidell.

10.4.1.1 FLOODWALL SEGMENTS

Along the West Slidell portion of the alignment, there would be floodwall segments constructed due to the presence of nearby residences. The floodwall segments are shown in red in Figure D:10-4.

The floodwall segments would have a hydraulic design elevation of 16 ft (year 2082). Starting from the west, there would be the following floodwall segments:

- 350-ft floodwall going through a group of properties at the end of West Doucette Road.
- 250-ft floodwall located on north side of Bayou Paquet.
- 1400-ft floodwall between east bank of Bayou Paquet and residences along Mayer Drive.

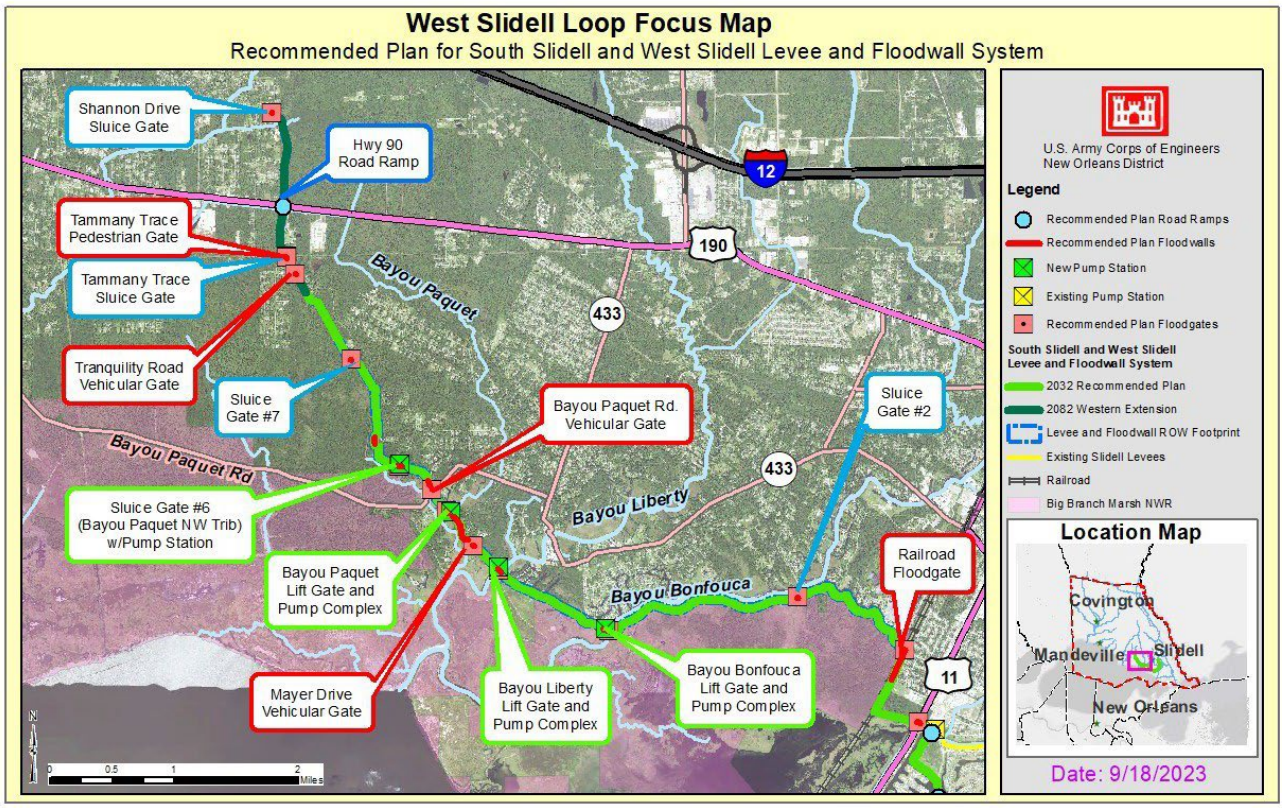


Figure D:10-4. West Slidell Levee and Floodwall System- Recommended Plan Focus with Structures

STRUCTURES

Along the West Slidell portion of the alignment, the structures would have a hydraulic design elevation of 16 ft (Year 2082). Refer to Figures D:10-3 and D:10-4. Starting from the west, there would be the following structures:

- Sluice gate #7 near Tranquility Road/CC Road (control structure). The gate width is 25 ft and structural opening height is 8.9 ft.
- Sluice gate #6 (control structure) at Bayou Paquet North Tributary. The gate width is 75 ft and structural opening height is 15.2 ft. Bayou Paquet North Tributary pump station with a pumping capacity of 300 cfs.
- 60-ft wide Bayou Paquet vehicular gate
- Lift gate at Bayou Paquet (navigable gate). The gate width is 90 ft and structural opening height is 16.5 ft. Bayou Paquet Pump station with a pumping capacity is 500 cfs.
- 20-ft wide Mayer Drive vehicular gate
- Lift gate at Bayou Liberty (navigable gate). The gate width is 80 ft and structural opening height is 22.8 ft. Bayou Liberty pump station with a pumping capacity of 1,800 cfs.
- Lift gate at Bayou Bonfouca (navigable gate). The gate width is 110 ft and structural opening height is 25 ft. Bayou Bonfouca pump station with a pumping capacity of 2,000 cfs.
- Sluice gate # 2 at Bayou Bonfouca (control structure). The gate width is 50 ft and opening height is 17.1ft.

The gates at Bayou Paquet, Bonfouca and Liberty were chosen to be lift gates to minimize impacts to and provide access to recreational navigation and to account for fish-and-larvae- friendly gate openings as requested by the natural resource agencies and USACE-MVN Planning Division. The levee alignment would tie into a 60-ft railroad floodgate. The railroad floodgate would have a hydraulic design elevation of 16.5 ft (year 2082).

10.4.2 WESTERN HIGH GROUND TIE-IN FOR YEAR 2082

After initial construction, the western terminus of the levee and floodwall system would be extended north to account for future conditions (year 2082) using the relative sea level change and subsidence. Updated modeling results, which included the intermediate scenario of sea level rise and subsidence, indicated a higher tie-in elevation would be needed through the period of analysis to continue to provide a 1 percent risk reduction.

To plan for the conditions expected throughout the 50-year period of analysis, the intermediate scenario of relative sea level change between years 2032 and 2082 was used to develop the 2082 hydraulic design elevations. Based on this information, an alignment extension with additional length of levee and additional structures was developed that would adapt the project while maintaining a 1 percent risk reduction. The Western High Ground Tie-in for year 2082 is shown in dark green in Figures D:10-3 and D:10-4.

Using the existing ground elevations available at the time of this study, it is projected that the ground elevation would be higher than the hydraulic design elevation until year 2076 when the risk reduction would be needed. It is projected that this levee would be constructed in year 2076 during the fourth lift of West Slidell.

The alignment would commence north of U.S. Highway 190 in the neighborhood near the intersection of North Tranquility Road and Shannon Drive between two properties. The alignment would be a berm with hydraulic design elevation of 17.5 ft for year 2082. The alignment would switch to levee (hydraulic design elevation of 17.5 ft (Year 2082)) and it would continue south on the edge of the properties and cross U.S. Highway 190, the Tammany Trace Bike Trail, and South Tranquility Road on the eastern side of Pineridge Road. The alignment would run south southeast an additional 890 feet past the intersection with South Tranquility Road and tie into the existing year 2032 alignment for West Slidell.

10.4.2.1 FLOODWALL SEGMENTS

For the Western High Ground Tie-in for year 2082, the are no floodwall segments.

STRUCTURES AND RAMP

For the Western High Ground Tie-in for year 2082, the structures would have a hydraulic design elevation of 17.5 ft. Refer to Figures D:10-3 and D:10-4. Starting from the west, there would be the following structures:

- Sluice gate near Shannon Drive (control structure). The gate width is 4 ft and the structural opening height is 1 ft.
- Highway 190 road ramp.
- Sluice gate at Tammany Trace Bike Trail (control structure). The gate width is 15 ft and the structural opening height is 5.5 ft.
- Pedestrian gate at Tammany Trace Bike Trail (swing gate). The gate width is 10 ft.
- 20-ft wide vehicular gate at Tranquility Road.
- Sluice gate # 7 near Tranquility Road/CC Road (control structure). The gate width is 25 ft and structural opening height is 8.9 ft.

10.4.3 SOUTH SLIDELL INITIAL CONSTRUCTION

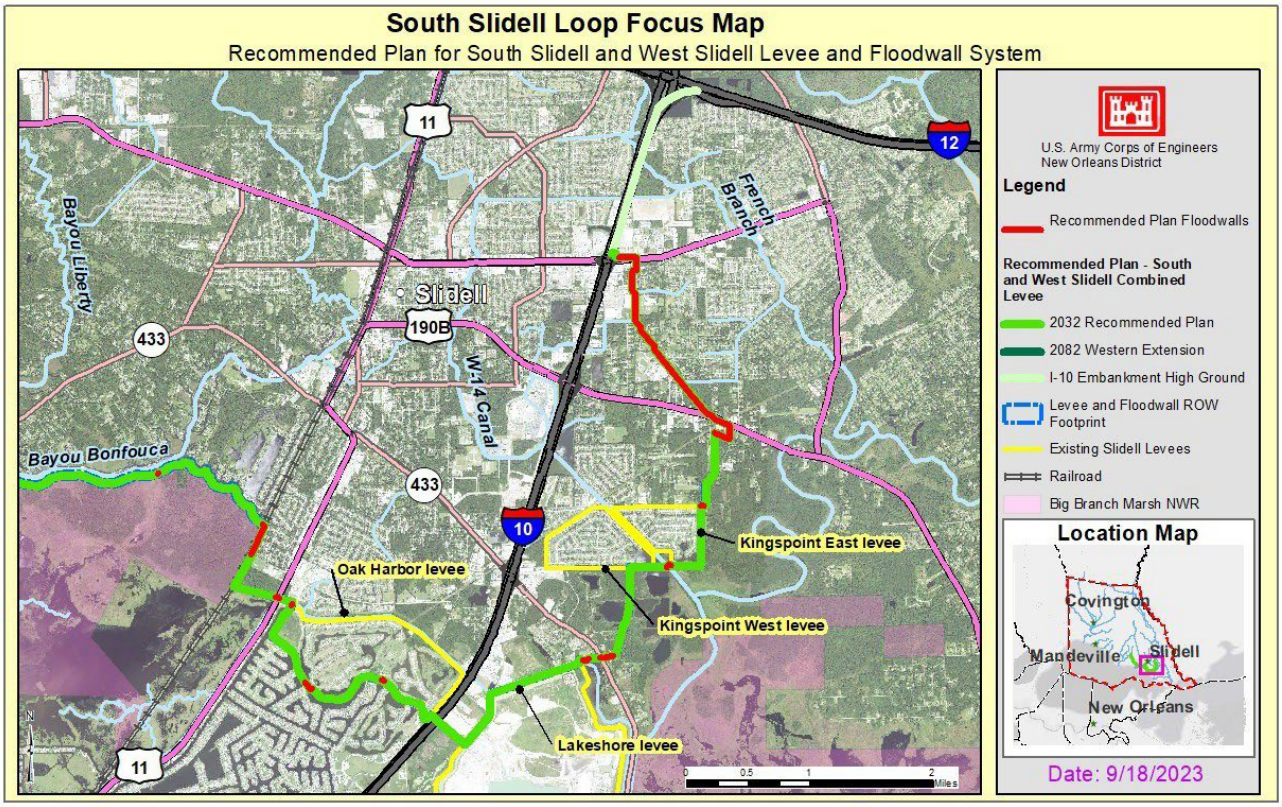


Figure D:10-5. South Slidell Levee and Floodwall System- Recommended Plan Focus

The levee and floodwall system alignment from West Slidell would continue to South Slidell. From the railroad gate connecting West Slidell with South Slidell, the alignment would transition to a floodwall parallel to the east side of the railroad tracks. The floodwall by the railroad tracks would have a hydraulic design elevation of 16.5 ft for year 2082.

The alignment would transition to levee when it turned east toward Highway 11. The alignment would cross Highway 11 and would turn south in the vicinity of the existing Schneider Canal Pump Station and then turn east (on a portion of the existing Oak Harbor ring levee). The alignment would run on the south side of Oak Harbor Boulevard and would cross to the north side immediately past Mariners Cove Boulevard. The levee along the south side of the Oak Harbor would have a hydraulic design elevation of 14 ft for year 2032.

The alignment would coincide with a portion of the existing Oak Harbor ring levee. The alignment would turn north and then east in the vicinity of the I-10. The I-10 would be raised to ramp over the new levee section (hydraulic design elevation of 18.5 ft for year 2082).

The alignment would continue southeast and would tie to an existing portion of the Lakeshore Estates ring levee. The alignment then would turn north and then east and cross Old Spanish Trail/Highway 433. The alignment would continue north and tie to a portion of the existing King's Point west levee. The section of levee would have a hydraulic design elevation of 16 ft for year 2032.

The alignment would cross the W-14 Canal and would tie to a portion of the existing King's Point east levee and would turn north. The levee would have a hydraulic design elevation of 16 ft for year 2032. The levee would turn east and then north. Immediately south of Highway 190 Business the alignment would turn from levee to floodwall to provide risk reduction to the existing Hardin Road power substation. The floodwall would have a hydraulic design elevation of 18.5 ft for year 2082.

The alignment (floodwall) would cross Highway 190 Business and continue northwest on the west side of the existing CLECO Corporate Holdings, LLC utility corridor. The alignment would cross South Holiday Drive and continue north. The alignment would turn east on Manzella Drive and turn north in the middle of the block between Yaupon Drive and Malbrough Drive.

The alignment (floodwall) would cross Gause Boulevard and would turn west (hydraulic design elevation for floodwall of 18.5 ft for year 2082). There would be a vehicular gate across Gause Boulevard, a vehicular gate for access to a private road, and a vehicular gate for the I-10 Service Road. The floodwall would transition to a berm that would tie-in to the I- 10 embankment. There would be a ramp for the on-ramp for the I-10 eastbound at Gause Boulevard.

For the berm, it was assumed a hydraulic design elevation of 16 ft for year 2032 and 19.5 ft for year 2082. The berm was assumed to be 1V:3H. This area of the alignment would be further developed during PED. The drainage on the grass area where the ramp merges to the I-10 would need to be reworked during PED.

The existing highway embankment would serve as the means of risk reduction for the project to form a continuous system up to the elevation required in 2082. There would be floodgates at Reine Canal and French Branch. Refer to light green portion of the alignment in Figure D:10-5.

CLECO Corporate Holdings, LLC has ROW use requirements pertaining to USACE work around their existing utility lines on the northeast corner of the floodwall alignment that would have to be met to provide clearance for construction activities (i.e., pile driving).

10.4.3.1 FLOODWALL SEGMENTS

Along the South Slidell portion of the alignment, floodwall reaches would be constructed due to the presence of nearby residences and other structures. The floodwall segments are shown in red in Figure D:10-5. These floodwall segments would have a hydraulic design elevation of 16.5 ft (year 2082). Starting from the west, there would be the following floodwall reaches:

- 1375 ft of floodwall along the railroad between Dellwood Pump Station and Baptist Church (Front Street).
- Fronting protection would cross downstream of existing Schneider Canal pump station. Minimal number of changes would be required.
- 500 ft of floodwall for narrow section of Oak Harbor levee at Mariners Cove Boulevard.
- 160 ft floodwall for the 20-ft vehicular gate for access to Oak Harbor Country Club.

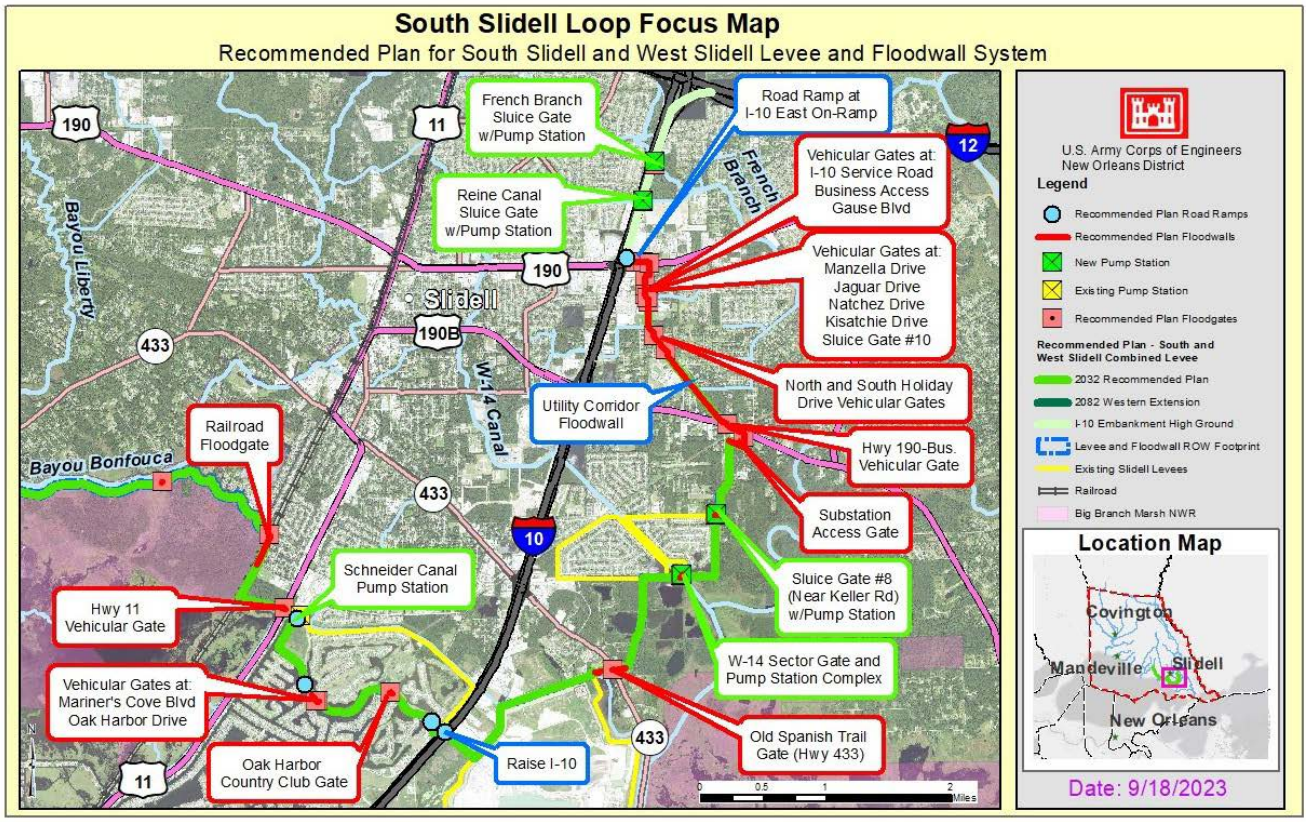


Figure D:10-6. South Slidell Levee and Floodwall System- Recommended Plan Focus with Structures

The following floodwall reaches would have a hydraulic design elevation of 18.5 ft (year 2082 elevation). Refer to Figure D:10-5, which shows the floodwall segments in red for South Slidell.

- 300 ft of floodwall near Old Spanish Trail.
- 450 ft of floodwall behind Esprit du Lac Street.
- 1,950 ft of floodwall to enclose power substation south of Highway 190 Business on east side of alignment.
- 430 ft of floodwall at Highway 190 Business (East Side).
- 3,530 ft of floodwall on western edge of the utility corridor.
- 3,700 ft of floodwall for northeast extension of alignment along the utility corridor and along east side of Yaupon Street.
- 650 ft of floodwall from Manzella Drive to Gause Boulevard
- 635 ft of floodwall north of Gause Boulevard to I-10, on the East Terminus

10.4.3.2 STRUCTURES AND RAMPS

Along the South Slidell portion of the alignment, the structures would have a hydraulic design elevation of 16.5 ft (year 2082). Refer to Figure D:10-6.

- 75-ft wide vehicular (roller) gate at Highway 11 (Pontchartrain Drive)
- Oak Harbor Boulevard ramp
- Islander Drive ramp
- 50-ft wide Mariners Cove Boulevard vehicular gate
- 20-ft wide Oak Harbor vehicular gate (Mariners Cove and Oak Harbor gates would be in proximity of each other)
- 20-ft wide Oak Harbor Country Club vehicular gate
- Grand Champions Lane ramp

After crossing the I-10, the structures for the alignment would have a hydraulic design elevation of 18.5 ft (year 2082 elevation).

- 30-ft wide Old Spanish Trail vehicular gate (Highway 433)
    - Sector gate at W-14 Canal (navigational gate). The gate width is 90 ft and opening height is 18.4 ft. W-14 pump station with pumping capacity of 1,000 cfs.
    - Sluice gate # 8 (control structure) at Kings Point East. The gate width is 90 ft and opening height is 14.1 ft. Kings Point East pump station with a pumping capacity is 200 cfs.
    - 20-ft wide Hardin Road Substation vehicular gate
    - 50-ft wide Hwy 190 Business vehicular gate
    - 20-ft wide South Holiday Drive vehicular gate
    - 20-ft wide North Holiday Drive vehicular gate
    - 20-ft wide Jaguar Drive vehicular gate

- Sluice gate # 10 near eastern terminus (control structure). The gate width is 20 ft and the structural opening height is 8 ft. 20-ft wide Natchez Drive vehicular gate
- 20-ft wide Kisatchie Drive vehicular gate
- 20-ft wide Manzella Drive vehicular gate
- 80-ft wide Gause Boulevard vehicular gate near eastern terminus
- 65-ft wide vehicular gate for businesses on north side of Gause Boulevard
- 85-ft wide vehicular gate on the I-10 Service Road near Gause Boulevard
- Ramp for I-10 on-ramp at Gause Boulevard

On the eastern terminus, the I-10 is consistently at high ground (hydraulic design elevation of 18.5 ft for year 2082). To use the I-10 embankment for risk reduction, any hydraulic openings past the tie-in point would need to be closed so that water does not have a path to flood the protected side of the alignment. There are hydraulic openings where Reine Canal and French Branch cross the I-10. The following structures (hydraulic design elevation of 18.5 ft for year 2082) would be needed north of the eastern terminus:

- Sluice gate at Reine Canal (control structure). The gate width is 30 ft and the structural opening height is 11 ft. Reine Canal pump station (pumping capacity is 200 cfs).
- Sluice gate at French Branch at I-10. The navigable gate width is 25 ft and the structural opening height is 10.2 ft. French Branch pump station (pumping capacity is 450 cfs).

10.4.4 INTERSTATE 10 ELEVATION

The I-10 road surface would be raised to construction elevation 22 ft to ramp over the new levee section to stay above the hydraulic design elevation for year 2082, to ensure the entire pavement section remains above the hydraulic design elevation across the interstate. The hydraulic design elevation at this location for year 2082 is 18.5 ft. The pavement section was assumed to have a thickness of 2.5 ft, and 1 ft settlement was assumed.

The existing elevation of the I-10 at the location of RP implementation is approximately 12.8 ft as per the terrain raster dataset. This location is the highest elevation of the I-10 in the vicinity of the RP alignment. The I-10 elevation is lower (approximately 10 feet) on the adjacent areas.

This feature would be fully designed during PED. USACE will coordinate with LA DOTD and the Federal Highway Administration.

The essential requirements to place a levee across an interstate corridor are listed below. The traffic control would be a complex item to be developed during PED to allow for uninterrupted interstate traffic while working on a total of 6 lanes of traffic.

10.4.4.1 GENERAL DESCRIPTION OF CONSTRUCTION STAGES

The following narrative describes the existing section along I-10, the subsequent construction for four stages of work, and the final finished section. Each section shows the work to be done and the traffic flow for each stage of work. Each stage of work would maintain three lanes of traffic in each direction. The detour section lanes would be 11 ft wide, rather than the normal 12 ft wide. The new travel lanes would remain 12 ft. wide.

Legend:

EB: east bound

WB: west bound

Arrows represent the direction of traffic on the travel lanes.

10.4.4.1.1 STAGE 1

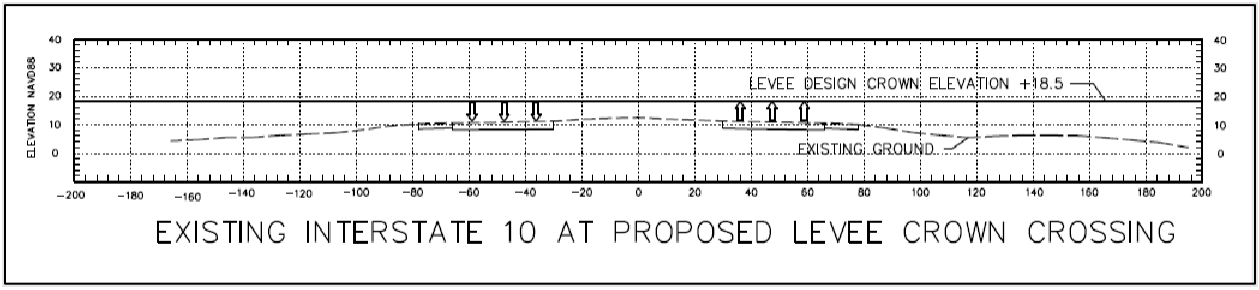


Figure D:10-7. Existing section along I-10

A detour would shift traffic to where the existing right lane in each direction is on the shoulder, the center lane is on the right lane, and the left lane is on the center lane. Construct detour section in existing median of I-10, including tie-in sections from the travel lanes to allow traffic from both sets of lanes to shift to the detour as needed. The detour may be constructed at grade, built to final levee design grade, or a grade in between. However, the top elevation of the detour would need to be so that the new I-10 travel lanes can tie into the shoulders of the detour section and not interfere with the travel lanes. The detour would also have to be low enough so that during initial construction the tie in slopes would not intersect with the existing travel lanes on I-10

(refer to typical detour sections). Other factors in the detour design include driver safety and drainage.

Included in constructing the detour section is clearing and grubbing the median, placing the pavement, installing jersey barriers for vehicle and construction crew protection, construction signage, road striping, and other incidental items. Construction must be built to the standards set forth by the Federal Highway Administration and Louisiana Department of Transportation and Development (La DOTD).

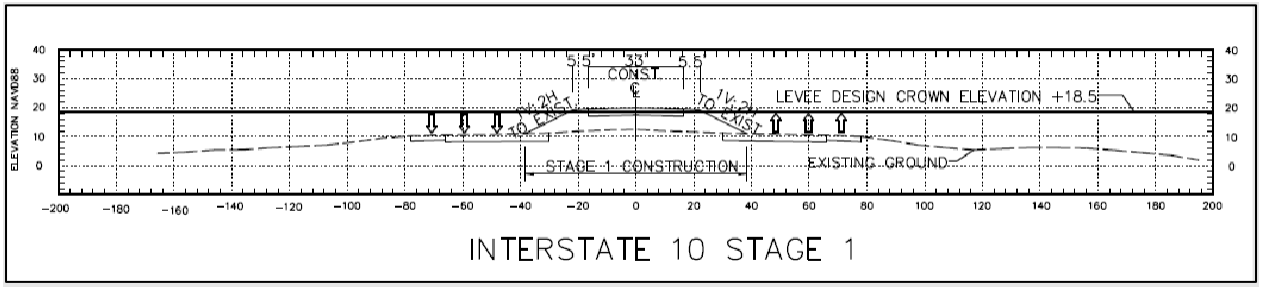


Figure D:10-8. Stage 1

10.4.4.1.2 STAGE 2

Shift traffic from WB lanes to the detour section. Construct WB lanes to design grade. Constructing the WB lanes includes the elements taken to construct the detour section.

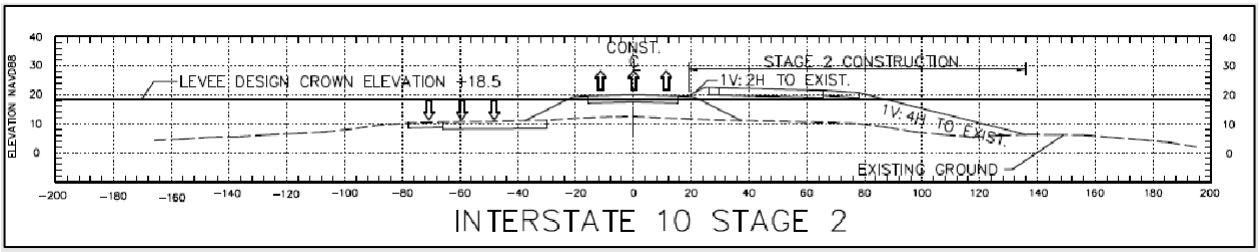


Figure D:10-9. Stage 2

10.4.4.1.3 STAGE 3

Reroute traffic from the detour lanes to the newly constructed WB lanes. Shift traffic on existing EB lanes to the detour section. Construct EB lanes to design grade. Constructing the EB lanes include the elements taken to construct the detour section.

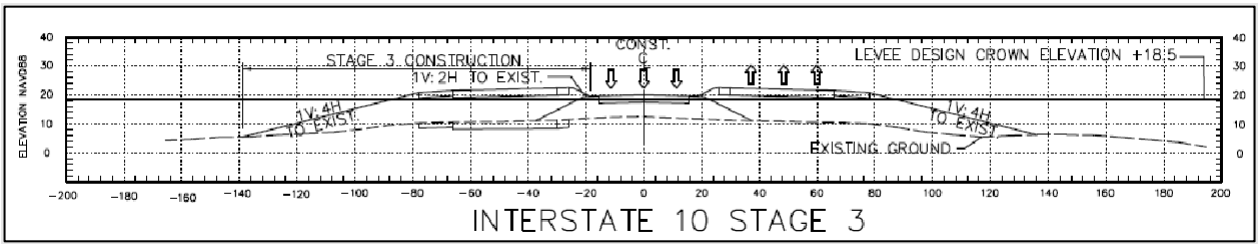


Figure D:10-10. Stage 3

10.4.4.1.4 STAGE 4:

Reroute traffic from the detour lanes to the newly constructed EB lanes. Remove detour pavement. Complete construction of median levee to design grade.

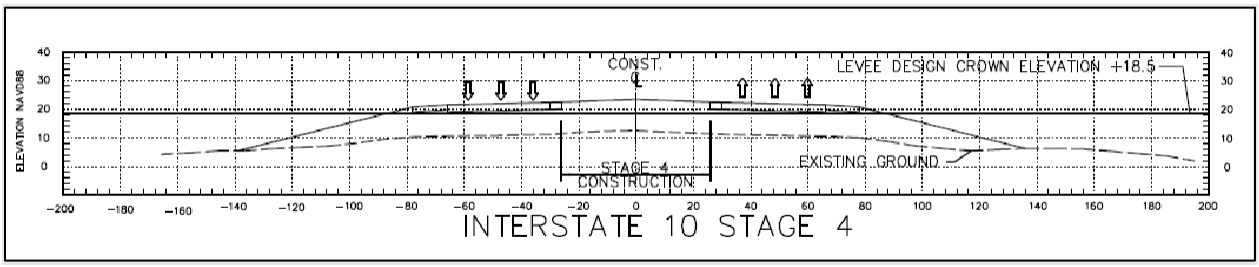


Figure D:10-11. Stage 4

The completed construction of the levee and the Interstate 10 is shown in Figure D:1-12.

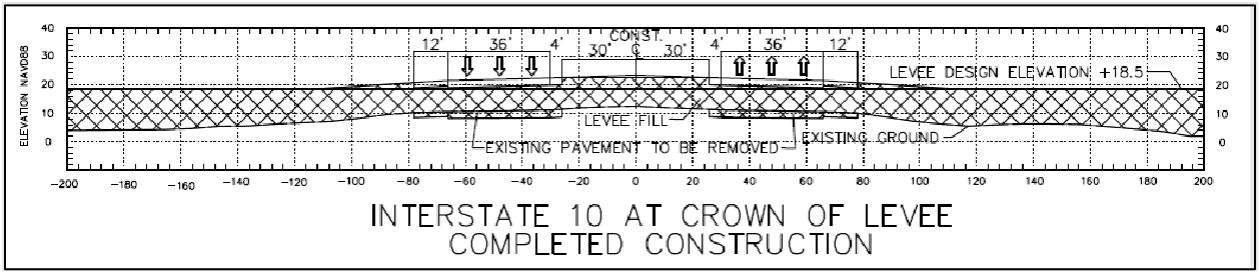


Figure D:10-12. Final Finished Section

10.4.5 Levee Typical Cross-Section and Quantities INITIAL CONSTRUCTION

In the RP, the total levee segments of the alignment would impact a total of approximately 360 acres of permanent ROW. This levee alignment would require approximately 3,000,000 cubic yards of fill (includes 30 percent contingency). Refer to Table D:10-2. Additional levee fill would be needed on the I-10 to raise the travel lanes over the design levee elevation at the required profile grade.

Table D:10-2. Total Levee Quantities for the Recommended Plan

RP Levee Quantities Initial Construction (Year 2032)	
Permanent ROW	360 acres
Fill Material (includes 30 percent contingency)	3,000,000 cubic yards

10.4.6 West Slidell Levee Dimensions and Quantities Initial Construction

The new West Slidell levee dimensions are shown in Table D:10-3 and Figure D:10-13. Berm sections would be needed on both sides of the levee. The hydraulic design elevations of the new West Slidell levee would be 13.5 feet (year 2032) and the 17.5 ft (year 2082). The construction elevation for the initial construction would be 14.5 ft.

Table D:10-3. West Slidell Levee

West Slidell Levee Initial Construction	
Levee Crown Width	10 ft
Side Slopes of Levee	1V:3H
Floodside Berm Slope	1V:42H
Landside Berm Slope	1V:33H
Construction Elevation	14.5 ft
Geotextile	13,200 lbs/ft

Geotextile (13,200 lbs/ft) would be placed under the levee during initial construction of West Slidell alignment. Geotextile would be placed 70 ft from the centerline of the levee on the floodside and 40 ft from the centerline of the levee on the land side for a total of 110 ft.

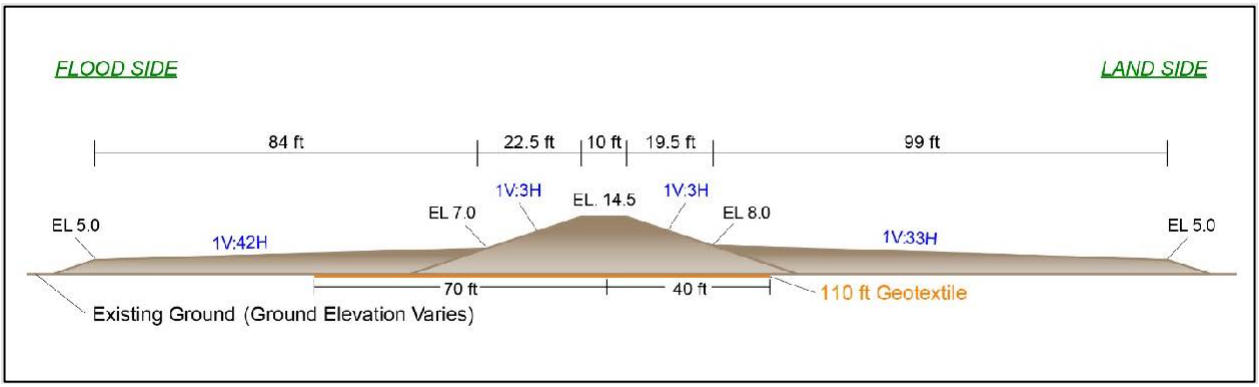


Figure D:10-13. Typical Cross-Section with Berms for West Slidell

ROW for the levee was assumed to be 300 ft wide.

The West Slidell levee alignment for the RP would impact approximately 240 acres of permanent ROW. This levee alignment would require approximately 2,007,000 cubic yards of fill (includes 30 percent contingency) as shown in Table D:10-4.

Table D:10-4. Levee Quantities for the Recommended Plan for West Slidell

West Slidell Only RP Levee Quantities **Initial Construction (Year 2032)**	
Permanent ROW	240 acres
Fill Material (includes 30 percent contingency)	2,007,000 cubic yards

10.4.7 SOUTH SLIDELL DIMENSIONS QUANTITIES INITIAL CONSTRUCTION

The South Slidell levee dimensions for the RP may be found in Table D:10-5 and Figure 10-14. The construction elevation for the first lift would vary depending on location. This portion of the alignment would not have berms or geotextile.

The hydraulic design elevation of the South Slidell levee for the RP would vary between 14 ft and 16 ft (year 2032).

Table D:10-5. South Slidell Levee

South Slidell Levee Initial Construction	
Levee Crown Width	10 ft
Side Slopes of Levee	1V:3H
Construction Elevation	Varies

The construction elevation for the initial construction would vary depending on location.

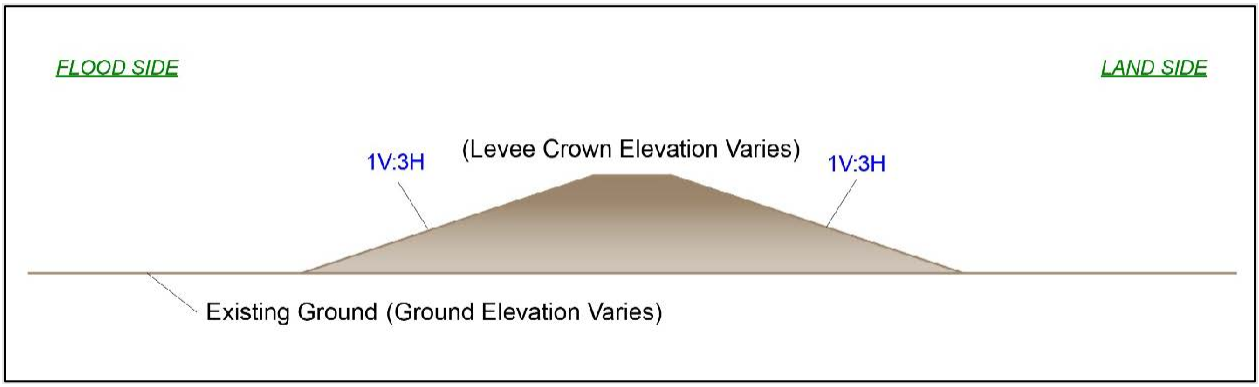


Figure D:10-14. Typical Cross-section for South Slidell

ROW for the levee was assumed to be 130 ft wide.

The South Slidell levee alignment for the RP would impact approximately 120 acres of permanent ROW. This levee alignment would require approximately 821,000 cubic yards of fill (includes 30 percent contingency) as shown in Table D:10-6.

Table D:10-6. Levee Quantities for the Recommended Plan for South Slidell

South Slidell Only Levee Quantities Initial Construction (Year 2032)	
Permanent ROW	120 acres
Fill Material (includes 30 percent contingency)	953,000 cubic yards*

*These quantities include the construction of the levee on the I-10.

The existing elevations used for the analysis were obtained from the terrain raster dataset.

10.4.8 FUTURE LEVEE LIFTS

Levees would be constructed in multiple lifts over the 50-year period of analysis to maintain the levee crown at or above the base year (2032) and future year (2082) design elevations while accounting for levee settlement and relative sea level rise (See Appendix E Hydrologic and Hydraulics for Sea Level Rise Analysis). Both the hydraulic design elevations and construction elevations vary by location. Hydraulic design elevations vary by levee location because of surge and wave differences due to storm path, wind speeds and direction, etc.

Levee lift schedules were developed to provide an estimate for potential future fill quantities needed to stay above a changing hydraulic grade elevation. Settlement durations and years of lifts are likely to change given future condition changes and could be shifted around for constructability purposes. Schedules could change with the acquisition of site surveys and thorough geotechnical explorations. Soil conditions will change in the future after added stress to the soil following future construction activities. Typically, the need for levee lifts is reevaluated several times throughout the lifetime of a project.

The levee lift schedule would follow the hydraulic design elevation requirements and thus were divided into three geotechnical reaches: Oak Harbor South; I-10 Crossing and Slidell East/Northeast as illustrated in Table D:10-7. The fourth lift (final lift for the 50-year period of analysis), projected to occur in year 2076 would elevate the levee to a construction elevation of 19 ft. It is during the scheduled fourth lift that construction of the Western High Ground Tie-in would be necessary for year 2082. The fill quantities listed for the fourth lift include quantities for the construction of the Western High Ground Tie-In.

Table D:10-7. Summary of Future Levee Lifts for the Levee Alignment of the Recommended Plan

	Construction Lift (year)	Construction Elevation (ft)	Permanent ROW (acres)	Fill Material (+30% contingency; cubic yards)
WEST SLIDELL				
First lift	2033	16	N/A	771,000
Second lift	2038	17.5	N/A	901,000
Third lift	2051	19	N/A	685,000
Fourth lift	2076	19	30 *	711,000 *
South Slidell				
Oak Harbor South				
First Lift	2035	17	N/A	106,000
Second Lift	2048	18	N/A	120,000
Third Lift	2064	19	N/A	115,000
I-10 Crossing**				
Slidell East / Northeast				
First Lift	2034	19	N/A	271,000
Second Lift	2047	20.5	N/A	295,000
Third Lift	2064	21.5	N/A	264,000
Total For Future Lifts			**30**	**4,239,000**

***** Includes the levee quantities (192,000 cubic yards) for the Western High Ground Tie-in for Year 2082.

** I-10 Crossing features would be constructed to the 2082 elevation and therefore would not require additional lifts.

Table D:10-8. Summary of Levee Material Quantities for the 50-Year Life of the Project

Levee	Permanent ROW (acres)	Fill Material (+30% contingency; cubic yards)
Initial Construction	360	3,000,000
Future Lifts	30	4,239,000
Total for Life of the Project	390	7,239,000

10.4.9 WEST SLIDELL LEVEE TYPICAL CROSS SECTION FOR FUTURE LIFTS

Four future levee lifts are projected to be needed for West Slidell. The dimensions for the assumed cross-section for these lifts is shown in Table D:10-9. Existing berm sections from initial construction would be in place on both sides of the levee.

Table D:10-9. West Slidell Levee

West Slidell Levee Future Lifts	
Levee Crown Width	10 ft
Side Slopes of Levee	1V:3H
Construction Elevation	Varies

The future levee lift elevations for West Slidell levee would vary as shown in the West Slidell Geotechnical Reach section. Levee ROW requirements are assumed to be 300 ft wide (same ROW as permanent ROW for initial construction).

No survey is data available; 30 percent contingency was used for borrow quantity calculations.

10.4.9.1 FIRST AND SECOND LIFTS

For the first lift (year 2033) and the second lift (year 2038), it was assumed that in addition to elevating the levee, the berm previously built during initial construction would settle 25 percent. Additional material would be placed on the berms during these two lifts.

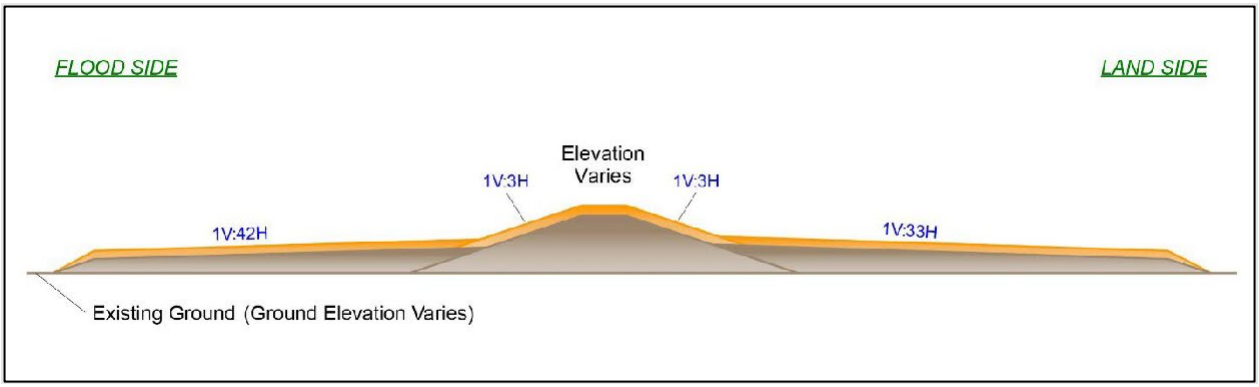


Figure D:10-15. Typical Cross-section with berms for First and Second Lifts for West Slidell

10.4.9.2 THIRD AND FOURTH LIFTS

For the third lift (year 2051) and the fourth lift (year 2076), it was assumed that no additional material would be placed on the berms.

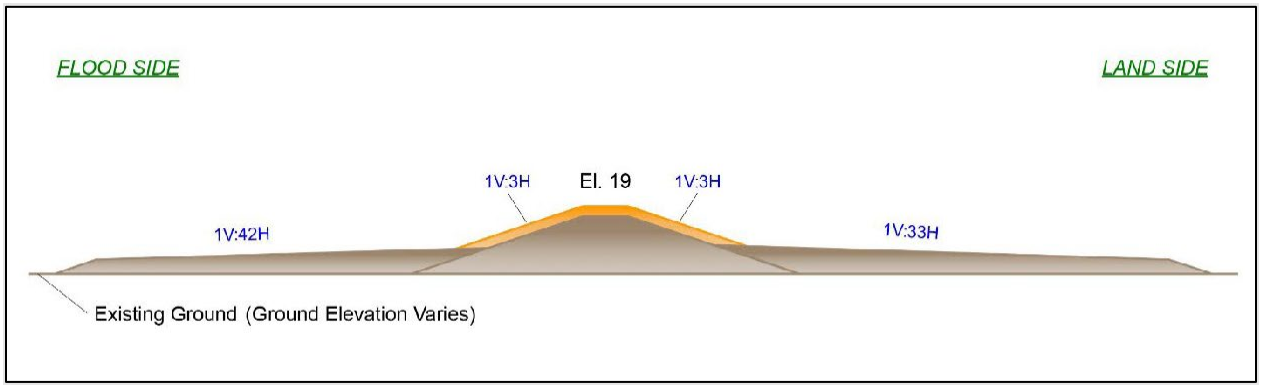


Figure D:10-16. Typical Cross-section with berms for Third and Fourth Lifts for West Slidell

10.4.10 WESTERN HIGH GROUND TIE-IN LEVEE CONSTRUCTION

The construction of the Western High Ground Tie-In would be performed during the fourth lift for West Slidell, which is projected for year 2076. The new levee would have a 10 ft wide levee crown and side slopes of 1V:3H. Levee construction elevation would be 19 ft. This portion of the alignment would not have berms or geotextile.

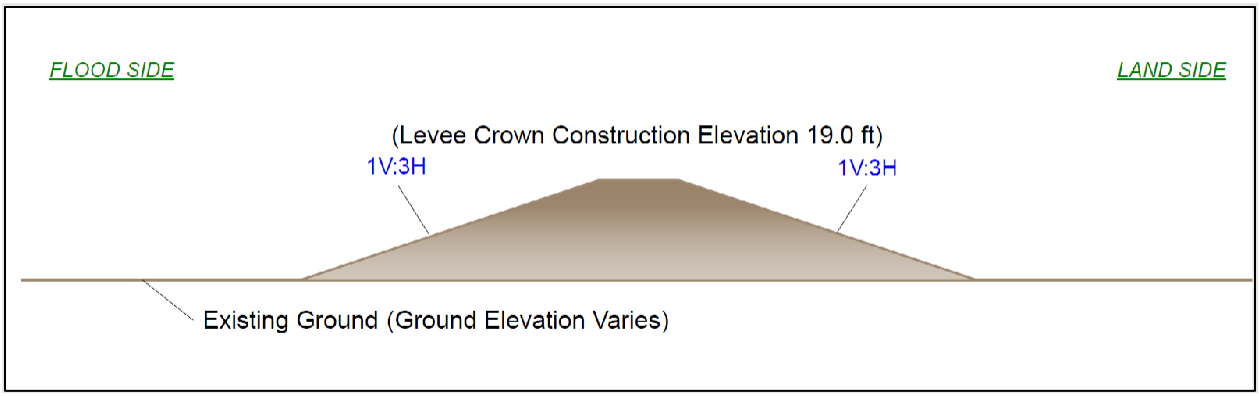


Figure D:10-17. Typical Cross-section for the Western High Ground Tie-in for Year 2082

10.4.11 WEST SLIDELL GEOTECHNICAL REACH

All geotechnical designs were based on assumptions made about soils in southeast Louisiana as no site-specific geotechnical information was available and geotechnical investigations were not conducted during feasibility level design. The West Slidell alignment would have a comprehensive geotechnical design during PED. This comprehensive design would divide the West Slidell alignment into various geotechnical reaches of varying dimensions based on site specific geotechnical information acquired during PED.

The lift schedules for West Slidell consisted of one geotechnical reach as shown in Figure D:10-18. The hydraulic design elevation is 13.5 ft for year 2032 and 17.5 ft for year 2082 are shown in the design line in blue.

The red lines represent the projected lifts. Table D:10-10 and Figure D:10-18 show the year of the construction lift and the construction elevation for each lift for West Slidell.

Table D:10-10. West Slidell Levee Lifts

Lifts for West Slidell Levee	Year of Construction Lift	Construction Elevation for the Levee (ft)
First lift	2033	16
Second lift	2038	17.5
Third lift	2051	19
Fourth lift	2076	19

The fourth lift (final lift for the 50-year period of analysis), projected to occur in year 2076, would elevate the levee to a construction elevation of 19 ft. This final lift would include the construction of the Western High Ground Tie-in for year 2082.

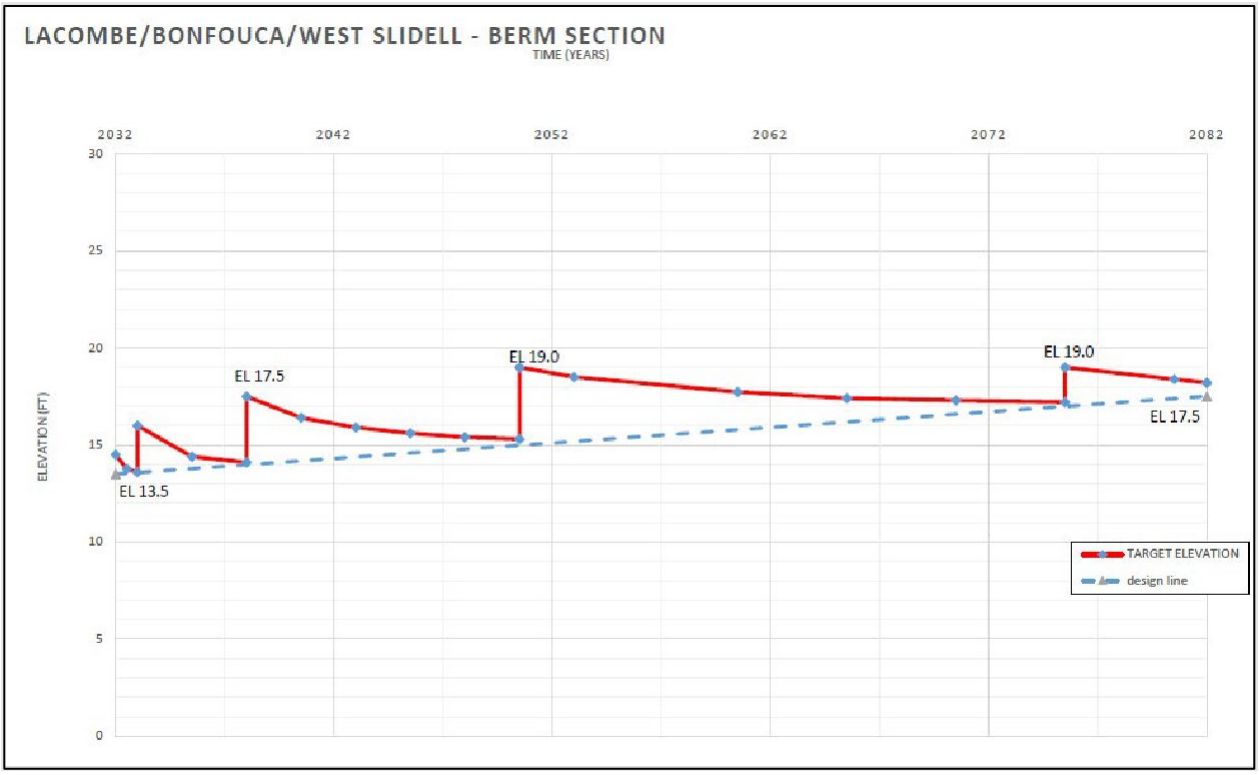


Figure D:10-18. Lift Curve for West Slidell geotechnical reach (includes the Western High Ground Tie-in for Year 2082)

10.4.12 SOUTH SLIDELL LEVEE TYPICAL CROSS SECTION FOR FUTURE LIFTS

The South Slidell future levee lifts would have a 10-feet-wide levee crown and side slopes of 1V:3H. The future levee lift elevations would vary as shown in the South Slidell Geotechnical Reaches section. The levee ROW was assumed to be 160 ft wide.

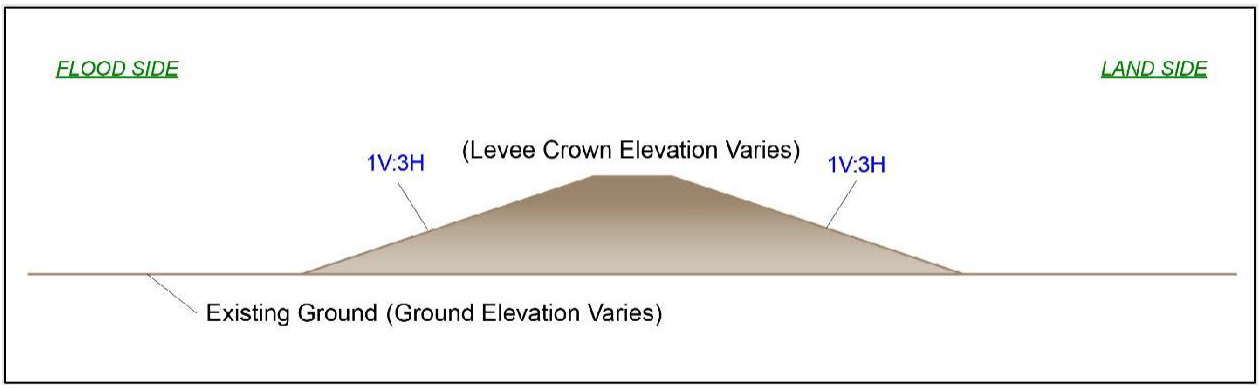


Figure D:10-19. Typical Cross-section for South Slidell Levee for Future Lifts

No survey data is available; 30 percent contingency was used for borrow quantity calculations.

10.4.13 SOUTH SLIDELL GEOTECHNICAL REACHES

The lift schedules for South Slidell follow the hydraulic design elevation requirements and thus were divided into three geotechnical reaches:

- Oak Harbor South
- I-10 Crossing
- Slidell East/Northeast

10.4.13.1 OAK HARBOR SOUTH GEOTECHNICAL REACH

All geotechnical designs were based on limited site-specific geotechnical information that was available and geotechnical investigations were not funded during feasibility level design. A full geotechnical exploration program with testing would be incorporated during PED. The Oak Harbor alignment would have a comprehensive geotechnical design during PED. This comprehensive design would divide the Oak Harbor alignment into various geotechnical reaches of varying dimensions based on site specific geotechnical information acquired during PED.

Figure D:10-20 shows the estimates for lift schedules for the Oak Harbor South levees. The year 2032 hydraulic design elevation of 14 ft and the year 2082 hydraulic design elevation of

17.5 ft are shown in the design line in blue. The red lines represent the projected lifts.

Table D:10-11 shows the construction elevations and Figure D:10-19 shows the typical cross-section for the three levee lifts projected to occur for Oak Harbor South geotechnical reach.

Table D:10-11. Levee Lifts for Oak Harbor South (South Slidell)

Lifts for Oak Harbor South (Slidell Levee)	Year of Construction Lift	Construction Elevation for the Levee (ft)
First lift	2035	17
Second lift	2048	18
Third lift	2064	19

Figure D:10-20. Lift Curve for Oak Harbor South Geotechnical Reach

10.4.13.2 INTERSTATE 10 CROSSING GEOTECHNICAL REACH

All geotechnical designs were based on limited site-specific geotechnical information that was available and geotechnical investigations were not funded during feasibility level of design. A full geotechnical exploration program with consolidation testing would be incorporated during PED. The I-10 crossing would have a comprehensive geotechnical design during PED.

The levee and the I-10 would be lifted during initial construction in year 2032 to construction elevation of 21.5 ft to avoid future disruptions to the traffic on the interstate.

Figure D:10-21 shows the projected lift schedule for the levee at the I-10. The year 2082 hydraulic design elevation of 18.5 ft is shown in the design line in blue. Table D:10-12 shows the construction elevation for the proposed work.

Table D:10-12. Initial Construction for I-10 Crossing (South Slidell)

I-10 Crossing	Year of Construction Lift	Construction Elevation for the Levee (ft)
Initial Construction	2032	21.5

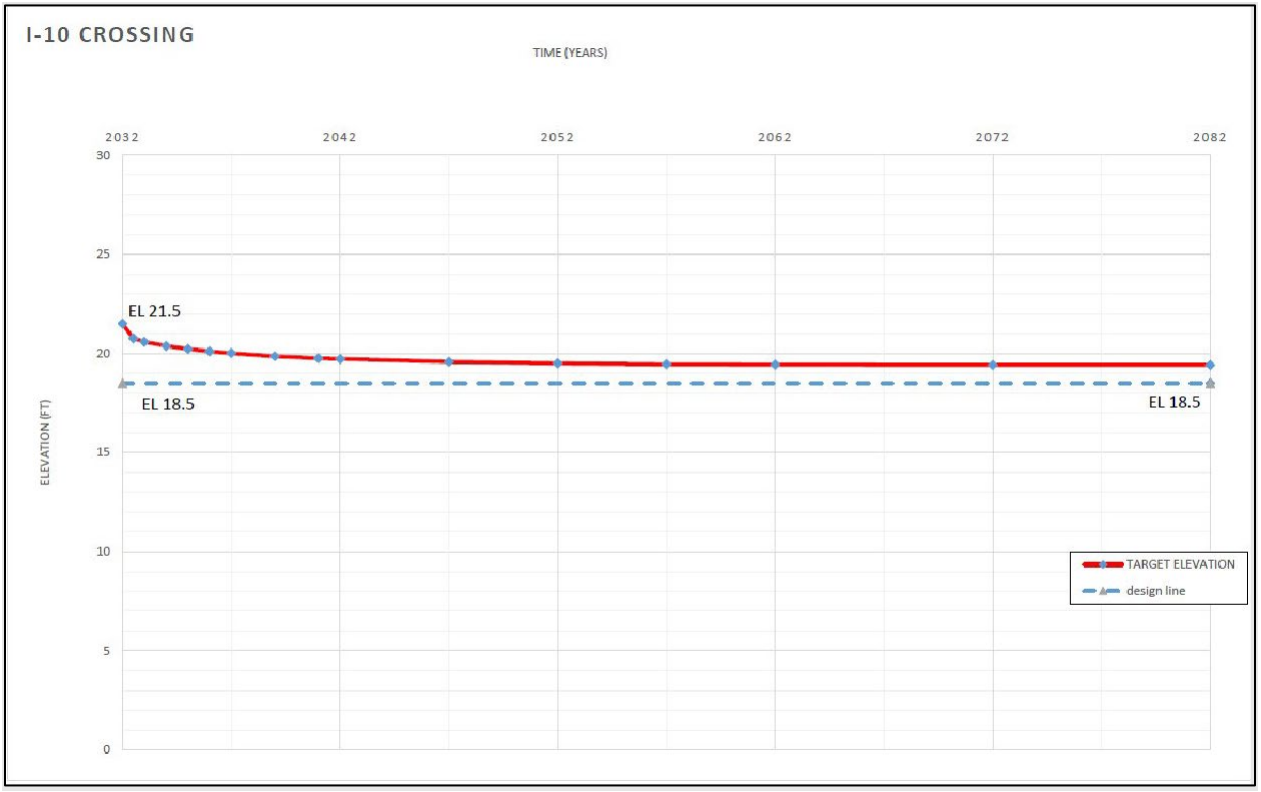


Figure D:10-21. Lift Curve for I-10 Crossing Geotechnical Reach

10.4.13.3 SLIDELL EAST/NORTHEAST GEOTECHNICAL REACH

All geotechnical designs were based on limited site-specific geotechnical information. Geotechnical investigations were not funded during feasibility level of design. A full geotechnical exploration program with testing would be incorporated during PED. The Slidell east/west alignment would have a comprehensive geotechnical design during PED. This comprehensive design would divide the Slidell east/west alignment into various geotechnical reaches of varying dimensions based on site specific geotechnical information acquired during PED.

Figure D:10-22 shows the estimates for lift schedules for the Slidell east/northeast levees. The year 2032 hydraulic design elevation of 16 ft and the year 2082 hydraulic design elevation of 20 ft are shown in the design line in blue. The red lines represent the projected lifts.

Table D:10-13 shows the construction elevations for the three levee lifts projected to occur for Slidell East/ Northeast geotechnical reach.

Table D:10-13. Levee Lifts for Slidell East/Northeast (South Slidell)

Lifts for Slidell East/Northeast (Slidell Levee)	Year of Construction Lift	Construction Elevation for the Levee (ft)
First lift	2034	19
Second lift	2047	20.5
Third lift	2064	21.5

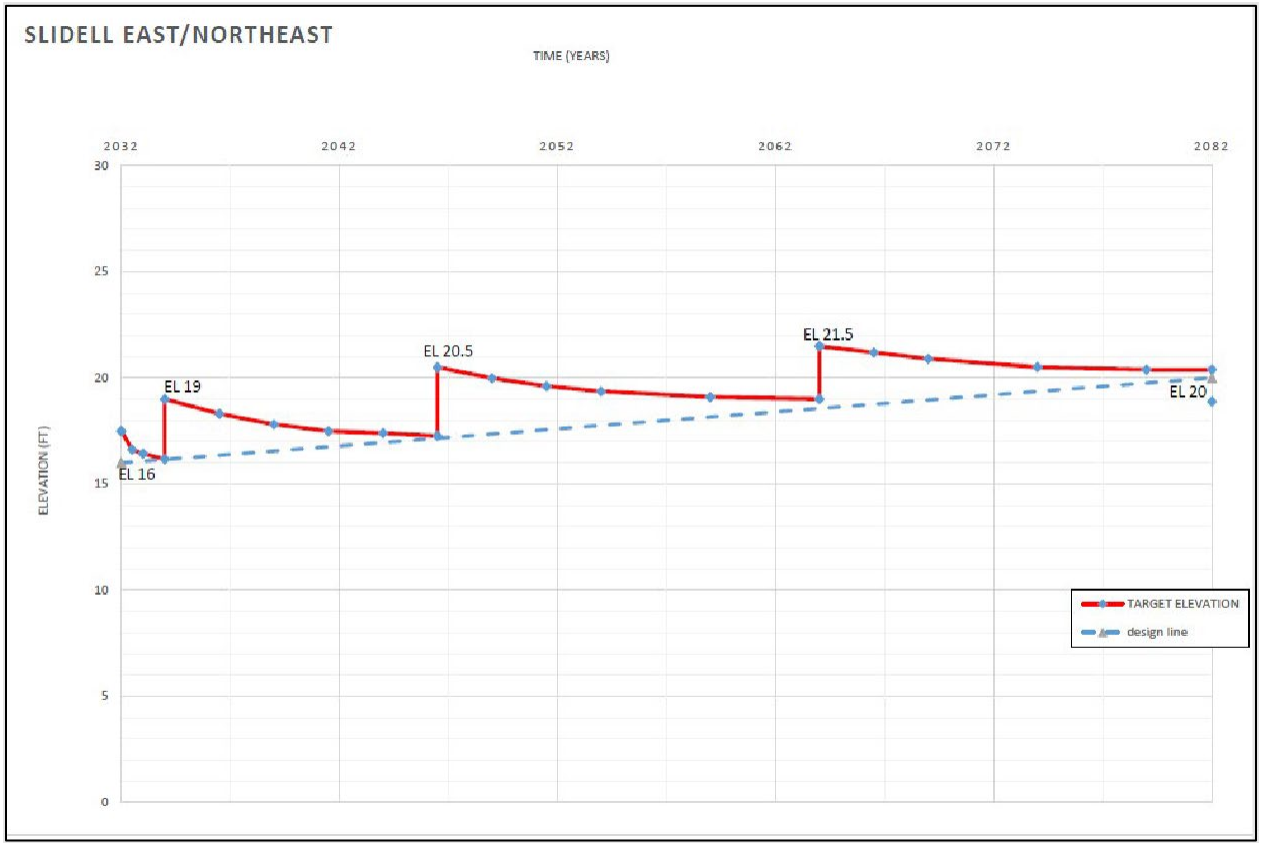


Figure D:10-22. Lift Curve for Slidell East/Northeast Geotechnical Reach

10.4.14 QUANTITIES FOR FUTURE LEVEE LIFTS

Analysis was done for future levee lifts using the lift schedule geotechnical analysis provided above. Tables D:10-14, Table D:10-15 and Table D:10-16 show the fill material needed for the future lifts. The existing elevations used for the analysis were obtained from the terrain raster dataset.

Table D:10-14. Levee Quantities for the Future Levee Lifts for the West Slidell Levee for the Recommended Plan

Levee Quantities Future Lifts West Slidell Only				
	First Lift Year 2033	**Second Lift Year 2038**	**Third Lift Year 2051**	**Fourth Lift Year 2076**
Permanent ROW	N/A	N/A	N/A	30 acres *
Fill Material (includes 30 percent contingency)	771,000 cubic yards	901,000 cubic yards	685,000 cubic yards	709,000 cubic yards *

***** Includes the levee quantities for the Western High Ground Tie-in for Year 2082.

The fill material needed for the levee for the Western High Ground Tie-in for year 2082, which would be constructed during the fourth levee lift (year 2076) for the West Slidell levee, is included in the quantities in Table D:10-14. The quantities for the Western High Ground Tie-in for year 2082 only are provided in Table D:10-15.

Table D:10-15. Quantities for the Western High Ground Tie-in for Year 2082 for the Recommended Plan

Levee Quantities **Western High Ground Tie-in for Year 2082 Only (West Slidell) Fourth Lift Year 2076**	
Permanent ROW	30 acres
Fill Material (includes 30 percent contingency)	192,000 cubic yards

Table D:10-16. Levee Quantities for the Future Levee Lifts for the South Slidell Levee for the Recommended Plan

Levee Quantities **Oak Harbor South (South Slidell)**			
	First Lift Year 2035	**Second Lift Year 2048**	**Third Lift Year 2064**
Permanent ROW	N/A	N/A	N/A
Fill Material (includes 30 percent contingency)	106,000 cubic yards	120,000 cubic yards	115,000 cubic yards
Levee Quantities **Slidell East / Northeast (South Slidell)**			
	First Lift Year 2034	**Second Lift Year 2047**	**Third Lift Year 2064**
Permanent ROW	N/A	N/A	N/A
Fill Material (includes 30 percent contingency)	271,000 cubic yards	295,000 cubic yards	264,000 cubic yards

10.4.15 RAMPS

The RP would include the construction of six ramps, which would include the ramp over the I-10 in the vicinity of Oak Harbor and the ramp in the Western High Ground Tie-In. All ramps would be constructed during initial construction except for the ramp in the Western High Ground Tie-In, which would be constructed during the fourth levee lift of the West Slidell levee in year 2076.

The following table shows the location of the ramps.

Table D:10-17. Ramps

Ramps
Western High Ground Tie-in for 2082
Highway 190
West Slidell
N/A
South Slidell
Oak Harbor Boulevard
Islander Drive
Grand Champions Lane
I-10 would be raised to ramp over the new levee section
I-10 On- Ramp

The following assumptions were made for the quantities for ramp constructions at Highway 190, Oak Harbor Boulevard, Islander Road, and Grand Champions Lane:

- Slope is 1V:12H (8.33 % grade)
- 50-ft vertical curves
- 11-ft lane width (For Oak Harbor Boulevard, 2 ft paved shoulders. For Highway 190, 12-ft lane width and 10 ft paved shoulders. For the other ramps, no paved shoulders.)
- Assumed 1 ft settlement
- Ramps constructed using the 2082 hydraulic elevations (19.5 ft).
- All ramps assumed 2 lanes of traffic. For Oak Harbor Boulevard, it was assumed 4 lanes of traffic.

Assumptions for the pavement for ramps at Highway 190, Oak Harbor Boulevard, Islander Road, and Grand Champions Lane are as follows:

- 2-inch top overlay pavement layer
- 2-inch sub overlay pavement layer
- 4-inch base course layer
- 8-inch aggregate layer
- 50-ft width of crown at ramps, 10-ft levee crown width.

The following assumptions were made for the quantities for ramp construction at the I-10 on- ramp in the north end of the South Slidell alignment:

- Slope is 1V:12H (8.33 % grade)
- 150-ft vertical curves
- 12-ft lane width.
- 14-ft paved shoulders (4-ft inside, 10-ft outside) using same pavement as travel lanes
- Assumed 1 ft settlement
- Ramps constructed using the 2082 hydraulic elevations (19.5 ft).
- Assumed 2 lanes of traffic.
- Assumptions for the pavement for at the I-10 on-ramp in the north end of the South Slidell alignment are as follows:
- 2-inch top overlay pavement layer
- 4-inch sub overlay pavement layer
- 6-inch base course layer
- 18-inch aggregate layer
- 150-ft width of crown at ramps, 10-ft levee crown width

10.5 FLOODWALL TYPICAL SECTION AND ELEVATIONS INITIAL CONSTRUCTION

The T-wall sections for the RP would vary based on location. Refer to Table D:10-18 for sizes for each floodwall segment.

Table D:10-18. Floodwall Segment Dimensions

Description of Floodwall Segment	Length of Floodwall Segment (ft)	Base of Slab BOS (ft)	Base of Wall BOW (ft)	Top of Wall TOW (ft)	Stem Height (ft)	Wall Thick (ft)	Slab Width (ft)	Number of piles per row
Western High Ground Tie-in for Year 2082								
N/A								
West Slidell								
Properties at the end of West Doucette	350	1.5	4.5	17.5	13	2	15	3
North Side Bayou Paquet Dr.	250	-1.5	1.5	16.5	15	2.5	20	4
Bayou Paquet/Mayer Dr.	1400	-1.5	1.5	16	14.5	2.5	20	4
South Slidell								
Front Street/ Railroad	1375	-0.5	2.5	16.5	14	2.5	20	4
Mariner's Cove Boulevard	500	7.5	10.5	16.5	6	1.5	10	2
Oak Harbor Country Club	160	8.5	11.5	16.5	5	1.5	10	2
Old Spanish Trail	300	-2.5	0.5	18.5	18	2.5	20	4
Esprit du Lac Street	450	1	4	18.5	14.5	2.5	20	4
Substation Floodwall	1950	4.5	7.5	18.5	11	2	15	3
Highway 190 Business	430	5	8	18.5	10.5	2	15	3
Utility Corridor	3530	5	8	18.5	10.5	2	15	3
Hollywood Dr. to Yaupon	3700	9	12	18.5	6.5	1.5	10	2
Manzella Dr. to Gause Boulevard	650	10.5	13.5	18.5	5	1.5	10	2
Gause Boulevard to I-10	635	13	16	18.5	2.5	1.5	10	2

10.5.1 FLOODWALL SEGMENTS WITH STEM HEIGHTS LESS THAN 8 FT

The typical cross-section for floodwall segments with stem heights less than 8 ft (Hollywood Drive to Yaupon Drive, and Gause Boulevard to I-10), is shown in Figure D:10-23.

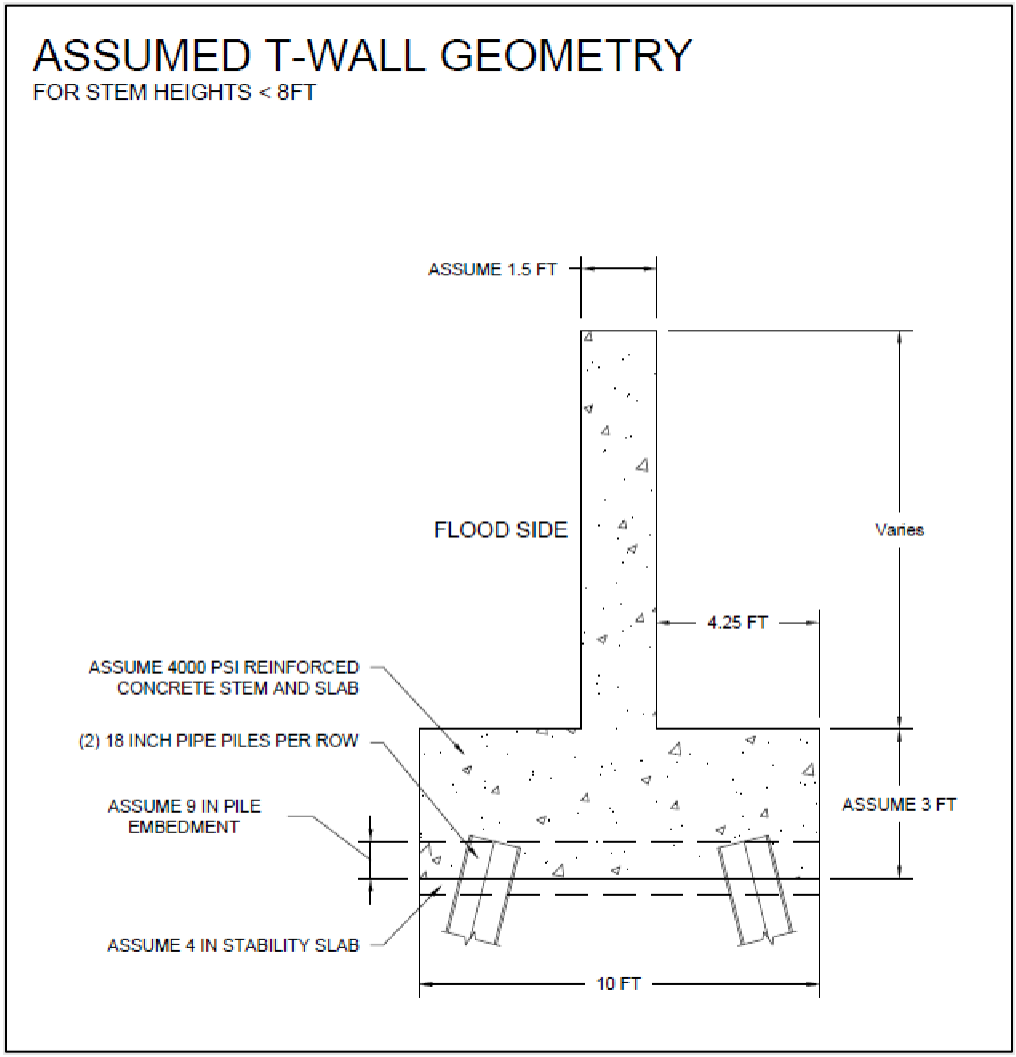


Figure D:10-23. Typical Cross-section for Floodwall with Stem Heights Less than 8 ft

10.5.2 FLOODWALL SEGMENTS WITH STEM HEIGHTS BETWEEN 8 FT AND 12 FT

The typical cross-section for two floodwall segments with stem heights with dimensions between 8 ft and 12 ft (Hollywood Drive to Yaupon Drive and Manzella Drive to Gause Boulevard), is shown in Figure D:10-24.

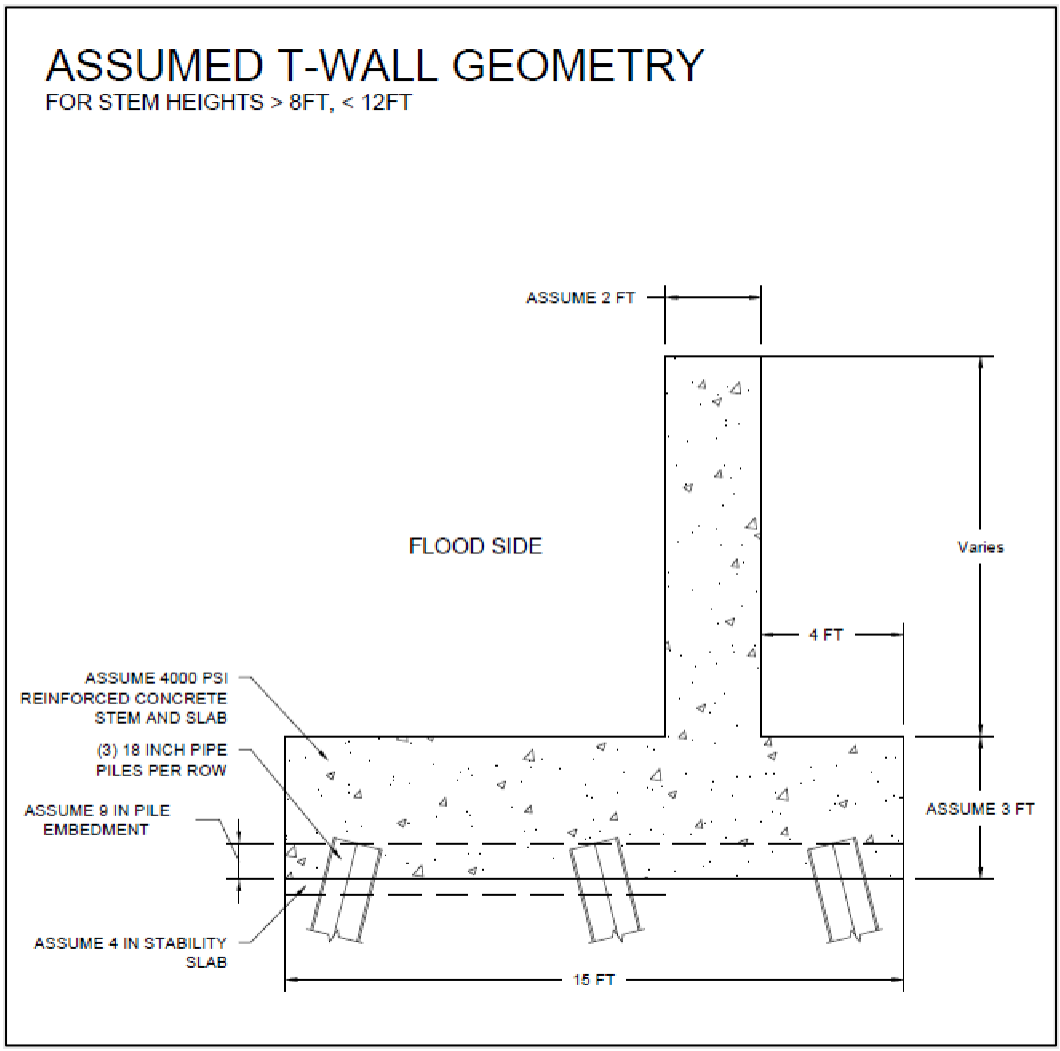


Figure D:10-24. Typical Cross-section for Floodwall with Stem Heights Between 8 ft and 12 ft

10.6 CONCRETE AND PILE QUANTITIES FOR FLOODWALL SEGMENTS FOR INITIAL CONSTRUCTION

The floodwall segments would require the following concrete quantities as shown on Table D:10-19.

Table D:10-19. Concrete Quantities for Floodwall Segments for the Recommended Plan

CONCRETE FLOODWALL SEGMENTS	
Total Concrete Quantities	37,100 cubic yards
Total Sheetpile Quantities	470,400 square ft
Total Slope Paving for floodwall/levees tie-ins	7,300 square ft

The design of the new T-wall including the foundation is subject to change once detailed geotechnical investigations are conducted during PED.

Table D:10-20. Pile Quantities for Floodwall Segments for the Recommended Plan

PILES FOR FLOODWALL SEGMENTS	
Type of pile	18-inch pipe
Configuration	1H:2V battered
Length of each pile	101 ft
Total Length of Piles	912,500 linear ft

10.7 FLOODGATES DESIGN INFORMATION

The RP includes a total of 13 gates. Three gates would be lift gates and one gate would be a sector gate. These gates would allow navigation of recreational vessels. There are nine sluice gates that would be control structures (non-navigable).

During construction of the gated structures, temporary bypass channels would be constructed for recreational vessels in Bayous Paquet, Bonfouca, and Liberty.

Table D:10-21. Floodgate Dimensions

Description of the Floodgate	Type of Gate	Width of Opening of the Gate (ft)	Ground/ Sill Elevation (ft)	Structural Height of Drainage Gate (ft)
Western High Ground Tie-in for Year 2082				
Sluice gate near Shannon Drive	Sluice	4	15.5	2.0
Tammany Trace Sluice Gate	Sluice	15	12	5.5
West Slidell				
Sluice Gate # 7 (Near CC Road)	Sluice	25	8.6	8.9
Sluice Gate # 6 (Bayou Paquet North Tributary)	Sluice	75	0.8	15.2
Bayou Paquet Gate Nav. Gate	Lift	90	-0.5	16.5
Bayou Liberty Nav. Gate	Lift	80	-6.8	22.8
Bayou Bonfouca Nav. Gate	Lift	110	-9	25.0
Sluice Gate # 2 (Bayou Bonfouca Sluice Gate)	Sluice	50	0.4	15.6
South Slidell				
W-14 Canal Nav. Gate	Sector	90	0.1	18.4
Sluice Gate # 8 (Kings Point East)	Sluice	90	4.4	14.1
Sluice Gate # 10 (Near Eastern Terminus)	Sluice	20	10.5	8.0
Reine Canal	Sluice	30	7.5	11.0
French Branch at I-10	Sluice	25	8.3	10.2

The floodgate locations and minimum sizes above are an estimate. A detailed interior drainage design would be provided during PED.

Limited information and estimates of channel depths and widths has been considered in estimates of the minimum gated opening dimensions. An increase in the size of the gated openings would likely benefit environmental conditions and would provide additional flood flow conveyance. Any channel constriction, such as a gate, has the potential to locally increase velocities, which could erode natural channels.

It is assumed that most of these floodgate locations would need to retain some flood conveyance capacity during construction. During PED, temporary bypass channels would be considered as part of the design.

10.7.1 Temporary Bypass Channel

Temporary bypass channels would be constructed at locations where a pump station or floodgate is proposed within the limits of a channel. The temporary bypass channel would route water around the structure for the construction to be completed in dewatered conditions.

To maintain pre-construction flow conditions and minimize environmental impacts during construction, the temporary bypass channels would be similarly sized to the channels being impacted. After construction, the bypass channel is assumed to be included in the footprint of the structure site and the channel flow would be rerouted through the new structure feature. Navigation of common local vessels would be considered for the bypass channels, and design features of a navigable bypass channel would be developed during PED.

10.7.2 Temporary Retaining Structures (TRS)

TRSs (cofferdams) are temporary features that facilitate the construction of major structures. Cofferdams allow water or other materials to be removed inside the TRS to work in an excavated and/or dewatered condition.

Cofferdams would be required during the construction of the pump stations and floodgates. Qualified designers employed or sub-contracted by the construction contractors would design the TRS for this project.

10.8 TYPES OF FLOODGATES

10.8.1 FISH-FRIENDLY LIFT GATE

For Bayou Paquet, Bayou Bonfouca and Bayou Liberty, the proposed navigable gates would be designed to have a small amount of restriction and a gradual slope so that fish and larvae may traverse the structures. The navigable gates would consist of a lift gate, which would be raised during open mode to let water and recreational vessels traverse. This design would include smaller sluice gates on both sides of the lift gate to simulate the natural opening of the bayous.

During PED, additional fish-friendly design and input provided by the NFS, U.S. Fish and Wildlife Service and National Marine Fisheries Service criteria, including the rock arch and rock ramp designs would be incorporated if necessary and applicable.

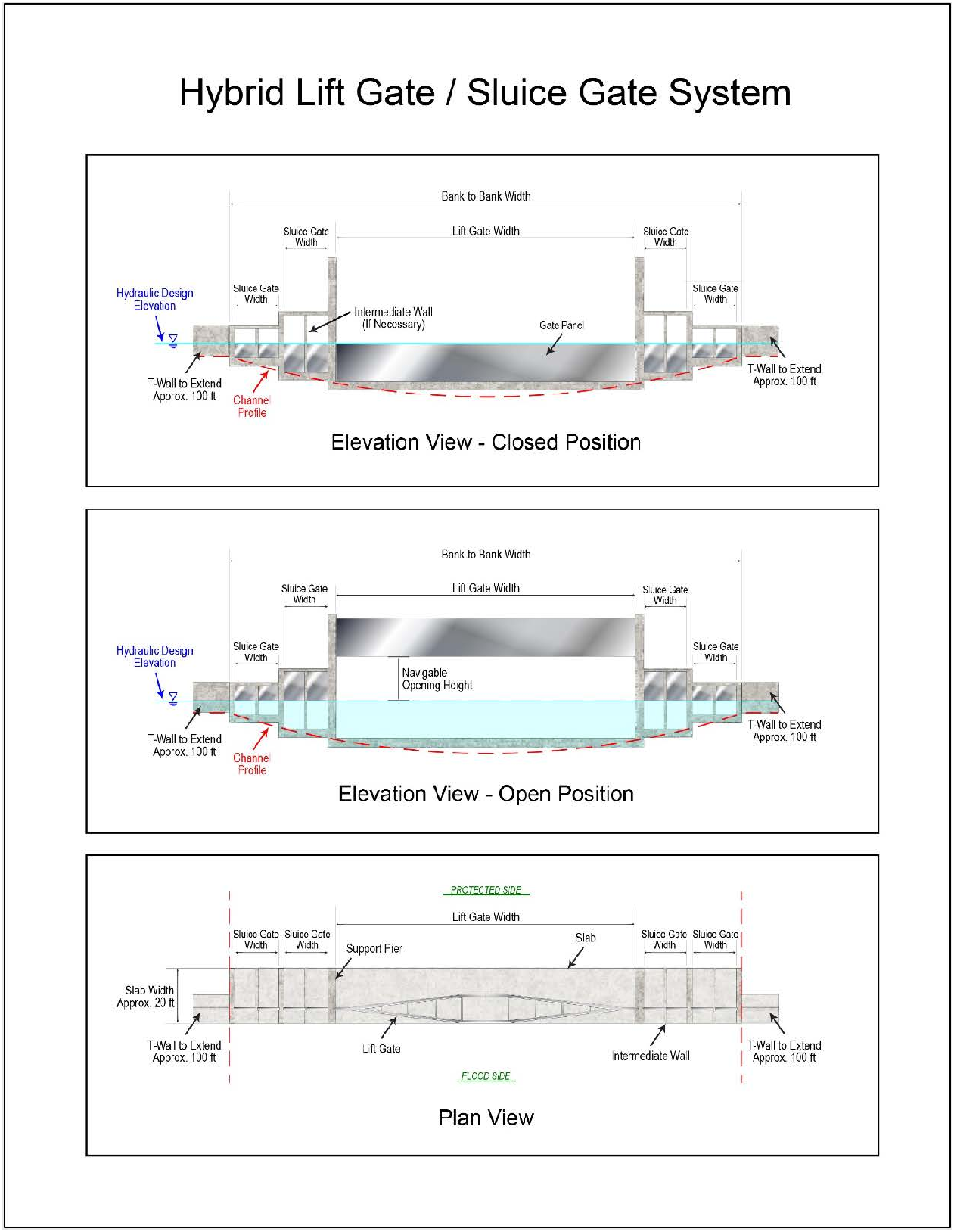


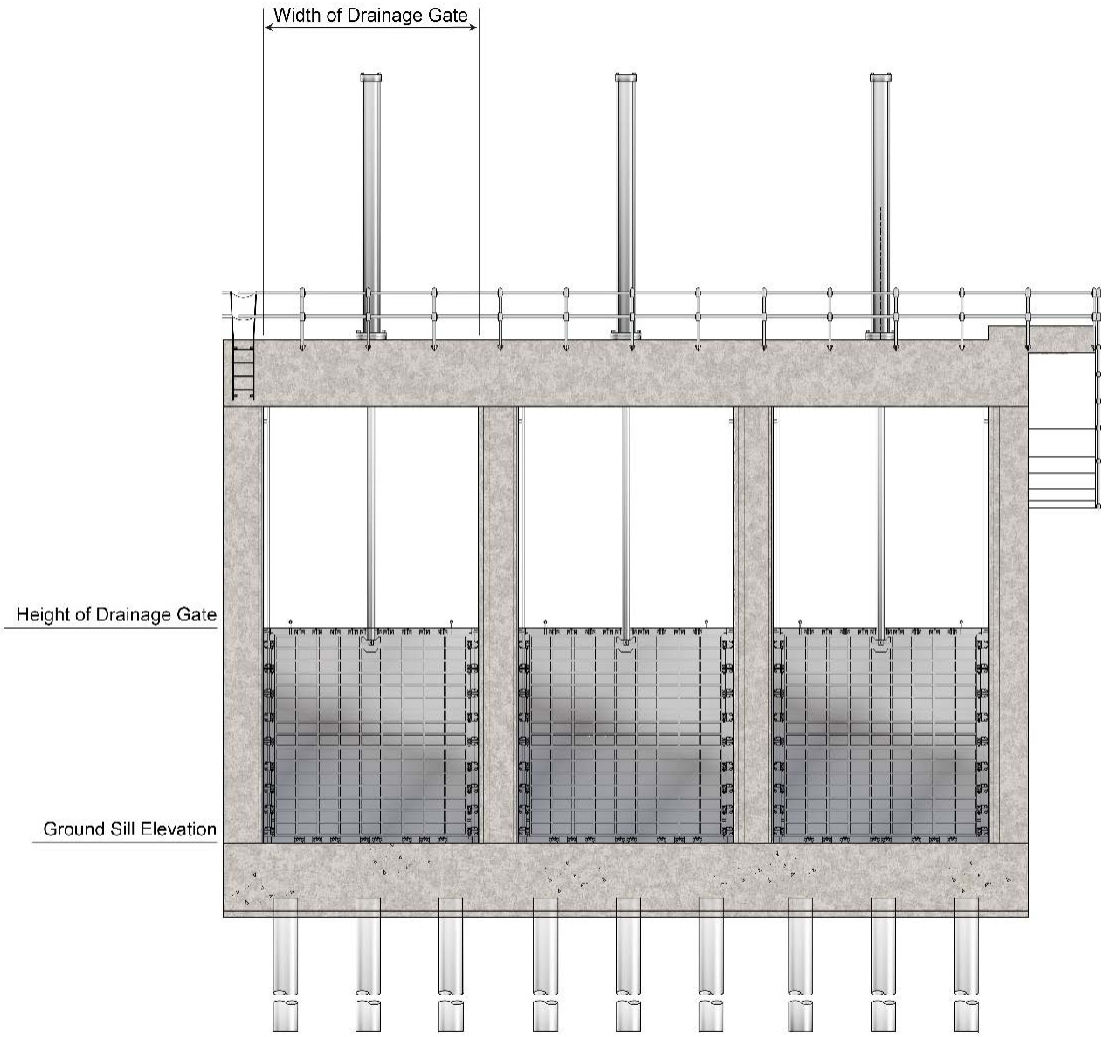
Figure D:10-26. Typical Fish-Friendly Gate - Elevation and Plan Views

10.8.2 SLUICE GATE

A sluice gate is a structure that contains a movable gate or series of movable gates that, when lifted, allow material and water to flow under it. Generally, sluice gates are not navigable as they do not raise high enough, or they have fixed components that do not allow vessels to pass through."

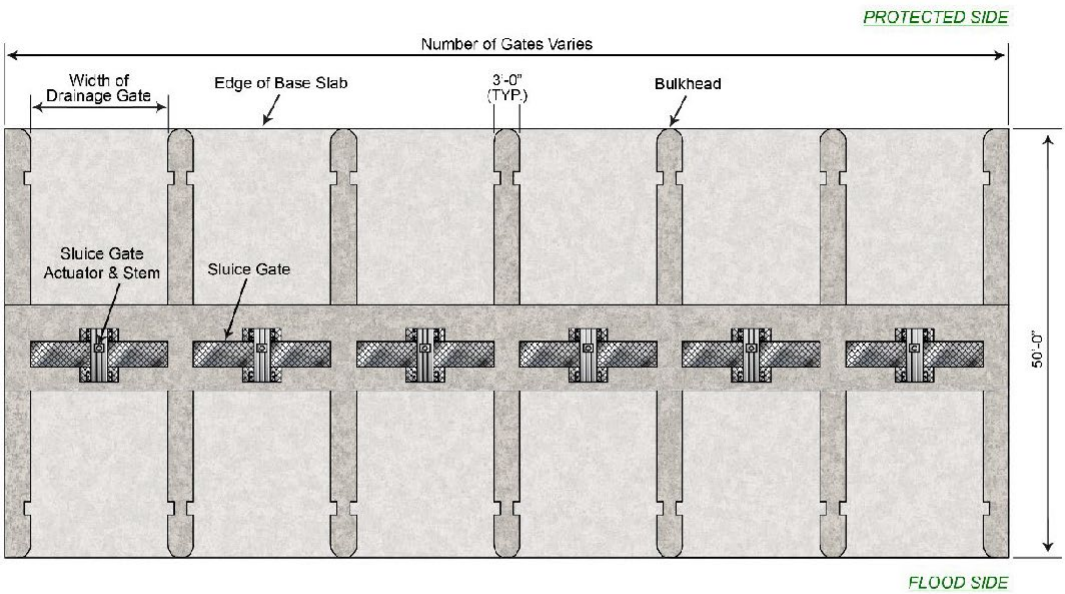
In West Slidell, there would be five sluice gates: a sluice gate near Shannon Drive, Tammany Trace sluice gate, sluice gate # 7 (near CC Road), sluice gate # 6 (Bayou Paquet North Tributary and sluice gate # 2 (Bayou Bonfouca Sluice Gate). The sluice gates near Shannon Drive and Tammany Trace would be constructed as part of the Western High Ground Tie-In for year 2082.

In South Slidell, there would four sluice gates: sluice gate # 8 (Kings Point East), sluice gate # 10 (near Eastern Terminus), at Reine Canal and at French Branch at I-10.



Typical Sluice Gate - Elevation View

Figure D:10-27. Sluice Gate - Elevation View

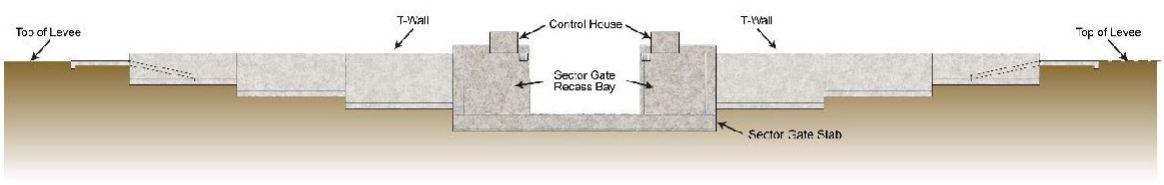


Typical Sluice Gate - Plan View

Figure D:10-28. Sluice Gate - Plan View

10.8.3 SECTOR GATE

A sector gate is a pie-slice structure that allows navigation to get through when in the open position. A sector gate is proposed for the W-14 Canal in the South Slidell portion of the alignment for the RP.



Sector Gate - Elevation View - Open Position

Figure D:10-29. Sector Gate - Elevation View with Gates in Open Position

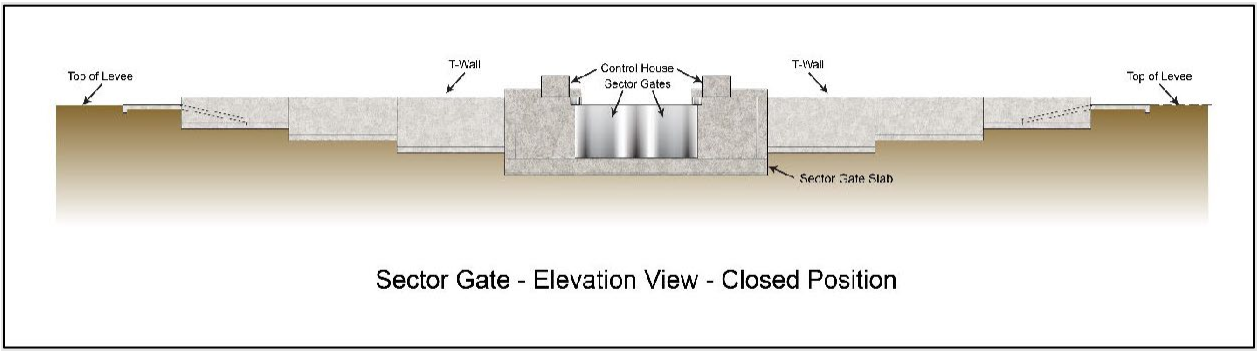


Figure D:10-30. Sector Gate - Elevation View with Gates in Closed Position

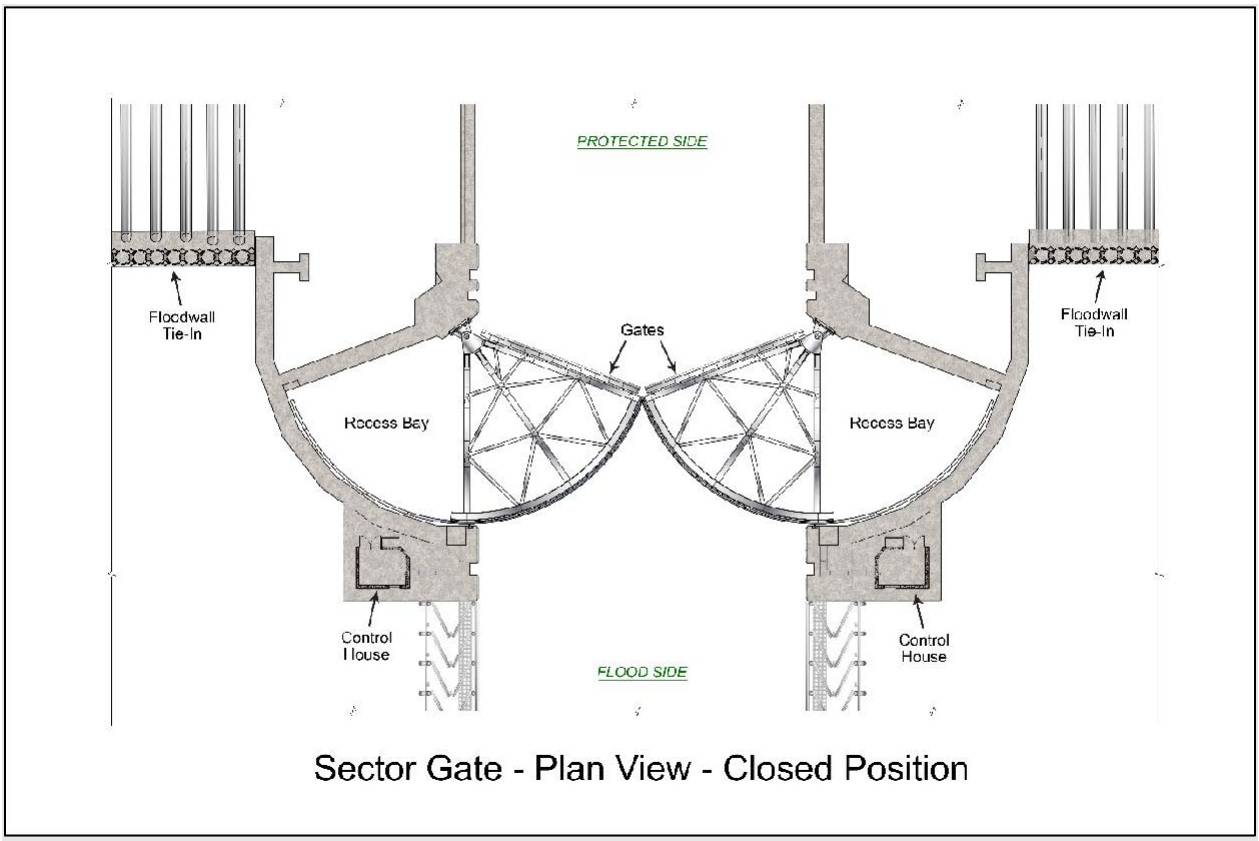


Figure D:10-31. Sector Gate - Plan View

10.8.4 ROLLER GATE

A roller gate is a structure that uses rollers for the gate to open and close. The operating motion of the gate is typically parallel to the skin plate face of the gate.

Refer to Table D:10-22 for location and dimensions of the roller gates.

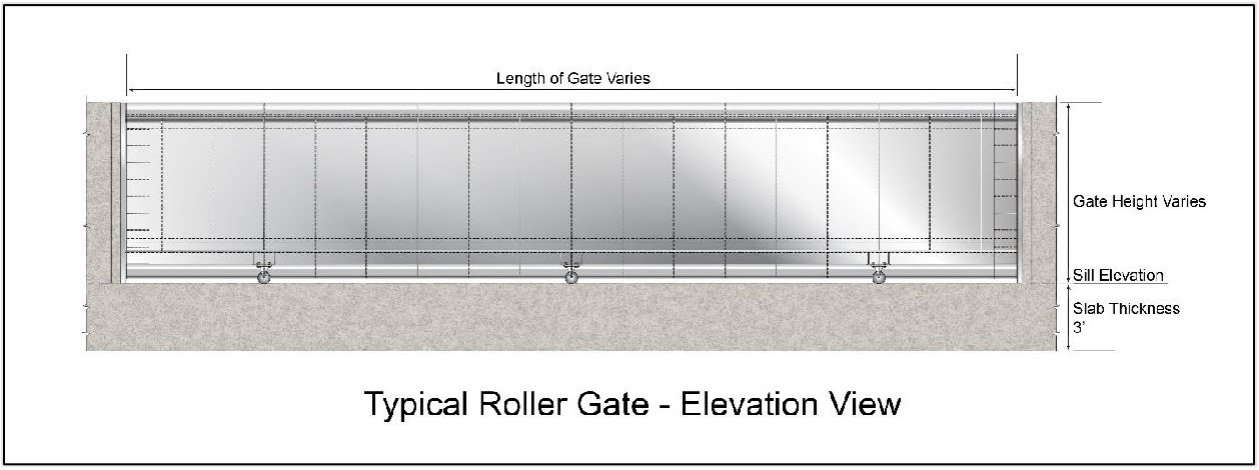


Figure D:10-32. Roller Gate - Elevation View

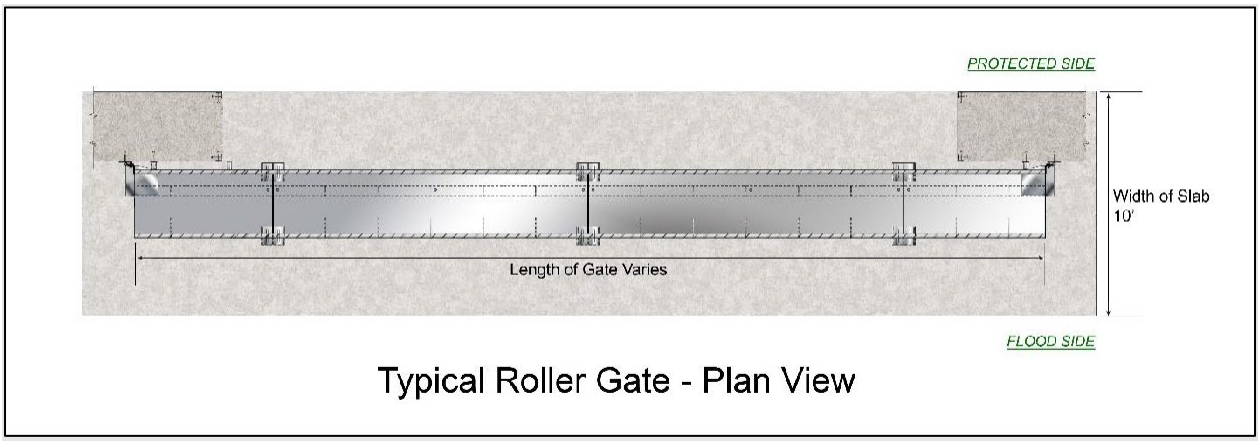


Figure D:10-33. Roller Gate - Plan View

10.8.1 SWING GATE

A swing gate is a structure that uses a hinge system to open horizontally. The gate can be actuated through automated mechanical means such as hydraulic arm or manually.

It was assumed that a swing gate would be constructed where the alignment crosses the Southern Railway Corp. railroad tracks. (The analysis for this gate was based on MRL Carrollton Railroad Gate.)

Refer to Table D:10-21 for location and dimensions of the swing gate.

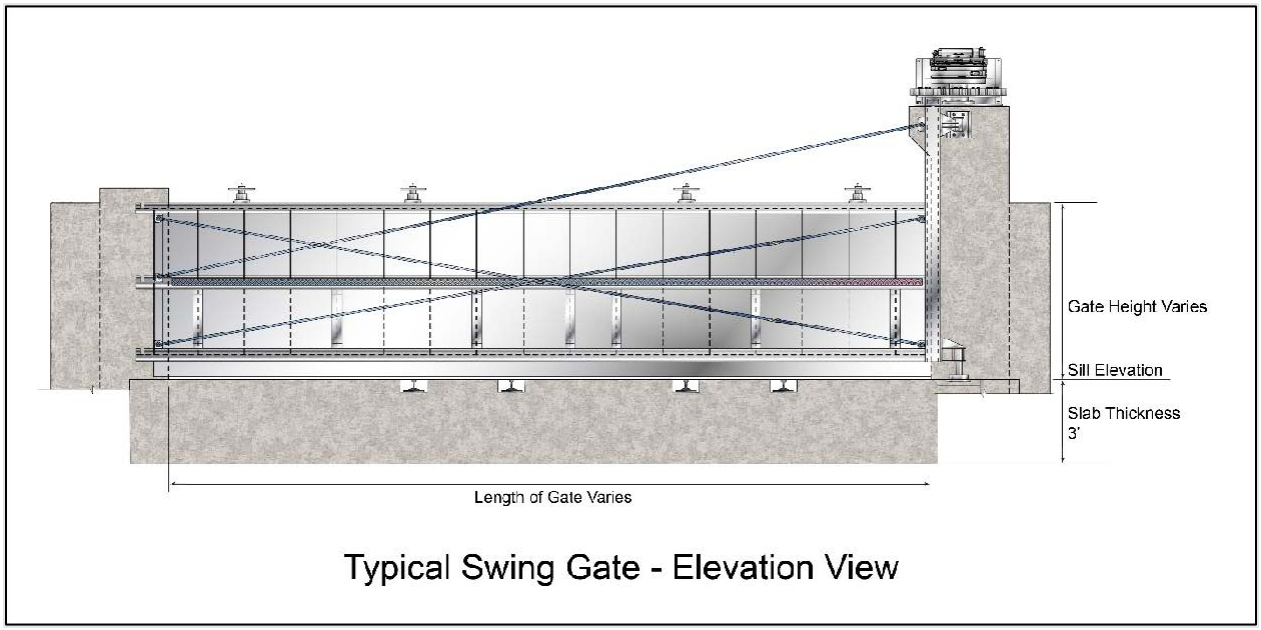


Figure D:10-34. Swing Gate - Elevation View

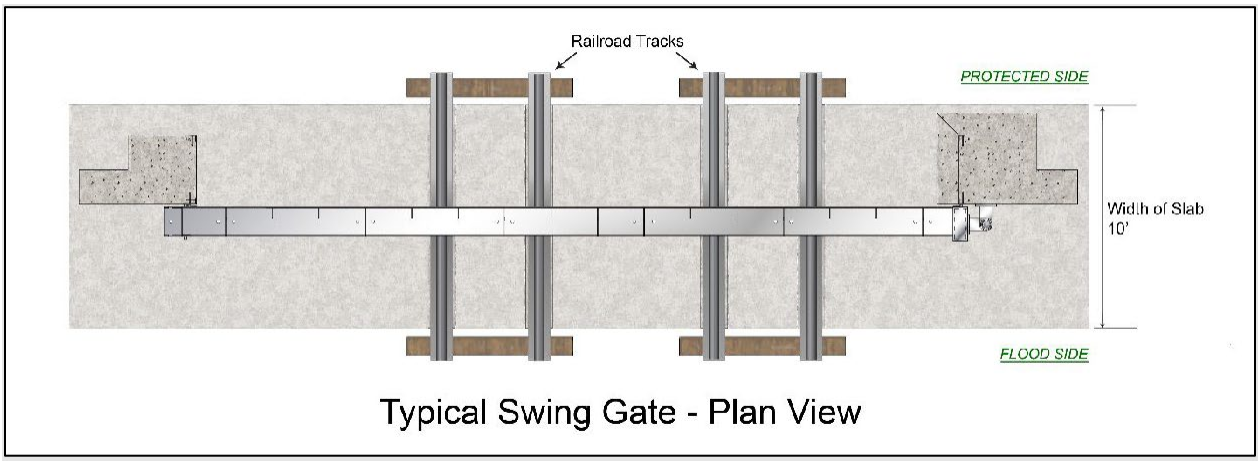


Figure D:10-35. Typical Swing Gate - Plan View

10.9 VEHICULAR, PEDESTRIAN AND RAILROAD GATES DESIGN ASSUMPTIONS AND INFORMATION

Access gate (roller gates and swing gates) concrete monolith slabs and piles were assumed to have similar geometry as the T-Wall designs for this study. Gate monoliths assume end columns for connection to steel access gates. Steel access gate quantities

are scaled from LPV-145 Chalmette Loop Levee - Bayou Bienvenue to Bayou Dupre, Roller Gate 100 percent design.

Table D:10-22 contains the design information for the eighteen vehicular, one pedestrian and one railroad gate for the RP.

The Western High Ground Tie-in for 2082 would contain one pedestrian and vehicular gate.

The West Slidell portion of the alignment would have two vehicular gates and one railroad gate.

The South Slidell portion of the alignment would have fifteen vehicular gates.

Table D:10-22. Vehicular, Pedestrian and Railroad Gates

Name	Description	Type	Mode	Width	Ground/ Sill Elevation (ft)	Design Height (ft	Height of Gate (ft)
Western High Ground Tie-in for 2082							
Tammany Trace Pedestrian Gate and Culvert	10-ft Pedestrian Gate at Tammany Trace with Lift Gate for Culvert on south side	Swing	Pedestrian	10	13	17.5	4.5
Tranquility Road Vehicular Gate	20-ft Vehicular Gate at Tranquility Road	Roller	Vehicle	20	12	17.5	5.5
West Slidell							
Bayou Paquet Road Floodgate # 2	60-ft Floodgate at Bayou Paquet Road	Roller	Vehicle	60	3	16	13
Mayer Drive Vehicular Gate	20-ft Vehicular Gate at Mayer Road	Roller	Vehicle	20	2.5	16	13.5
Railroad Floodgate	60-ft floodgate for Railroad	Swing	Railroad	60	0.5	16.5	16

South Slidell							
Hwy 11 Vehicular Gate	75-ft Roller Gate at Hwy 11 (Pontchartrain Drive)	Roller	Vehicle	75	4	16.5	12.5
Mariners Cove Floodwall and Vehicular Gate	500 linear ft of floodwall for narrow section of Oak Harbor levee at Mariners Cove Blvd	Roller	Vehicle	50	10.5	16.5	6
Oak Harbor Vehicular Gate	Floodwall and 20-ft Vehicular Gate for Oak Harbor	Roller	Vehicle	20	11.5	16.5	5
Oak Harbor Country Club Vehicular Gate	Floodwall and 20-ft Vehicular Gate for access to Oak Harbor Country Club	Roller	Vehicle	20	11.5	16.5	5
Old Spanish Trail Floodgate (Hwy 433)	30-ft roller gate at Hwy 433 east crossing (Old Spanish Trail)	Roller	Vehicle	30	3.5	18.5	15
Hardin Road Substation Gate	20-ft roller gate for access from Hardin Road to power substation	Roller	Vehicle	20	8	18.5	10.5
Hwy 190-B Floodgate (East Floodwall)	50-ft roller gate at Hwy 190-B east crossing (Fremaux Road)	Roller	Vehicle	50	9	18.5	9.5
South Holiday Drive Vehicular Gate	20-ft roller gate at South Holiday Drive	Roller	Vehicle	20	14	18.5	4.5
North Holiday Drive Vehicular Gate	20-ft roller gate at North Holiday Drive	Roller	Vehicle	20	14	18.5	4.5

Jaguar Drive Vehicular Gate	20-ft roller gate at Jaguar Avenue	Roller	Vehicle	20	12	18.5	6.5
Natchez Drive Vehicular Gate	20-ft roller gate at Natchez Avenue	Roller	Vehicle	20	12	18.5	6.5
Kisatchie Drive Vehicular Gate	20-ft roller gate at Kisatchie Avenue	Roller	Vehicle	20	14	18.5	4.5
Manzella Drive Vehicular Gate	20-ft roller gate at Manzella Drive (Added to extend floodwall to 18.5 ft ground elevation south of Hwy 190)	Roller	Vehicle	20	15	18.5	3.5
Gause Boulevard Vehicular Gate	80-ft roller gate crossing Gause Boulevard	Roller	Vehicle	80	16	18.5	2.5
Private Road Vehicular Gate	65-ft roller gate crossing private road north of Gause Boulevard	Roller	Vehicle	65	16	18.5	2.5
I-10 Service Road Vehicular Gate	85-ft roller gate crossing the I-10 service road	Roller	Vehicle	85	16	18.5	2.5

10.10 PUMP STATIONS DESIGN INFORMATION

The RP would include a total of eight (8) new pump stations. These pump stations are divided into two groups: large pumping capacity and small pumping capacity.

10.10.1.1 PUMP STATIONS WITH LARGE PUMPING CAPACITY

In West Slidell there would be two pump stations with large pumping capacity and two pump stations with small pumping capacity. In South Slidell there would be four pump stations with small pumping capacity.

The RP would include two pump stations with large pumping capacity at Bayou Liberty (1,800 cfs) and Bayou Bonfouca (2,000 cfs). These pump stations were assumed to have similar components and configuration as the USACE Westshore Lake Pontchartrain Reserve Relief Canal Pump Station (WSLP Pump Station). The structural quantities from the Reserve Relief Canal Pump Station were scaled accordingly to reflect the size of the pump stations for this study.

Figure D:10-36 and Figure D:10-37 show a typical site plan and a typical layout of a pump station with large pumping capacity, respectively.

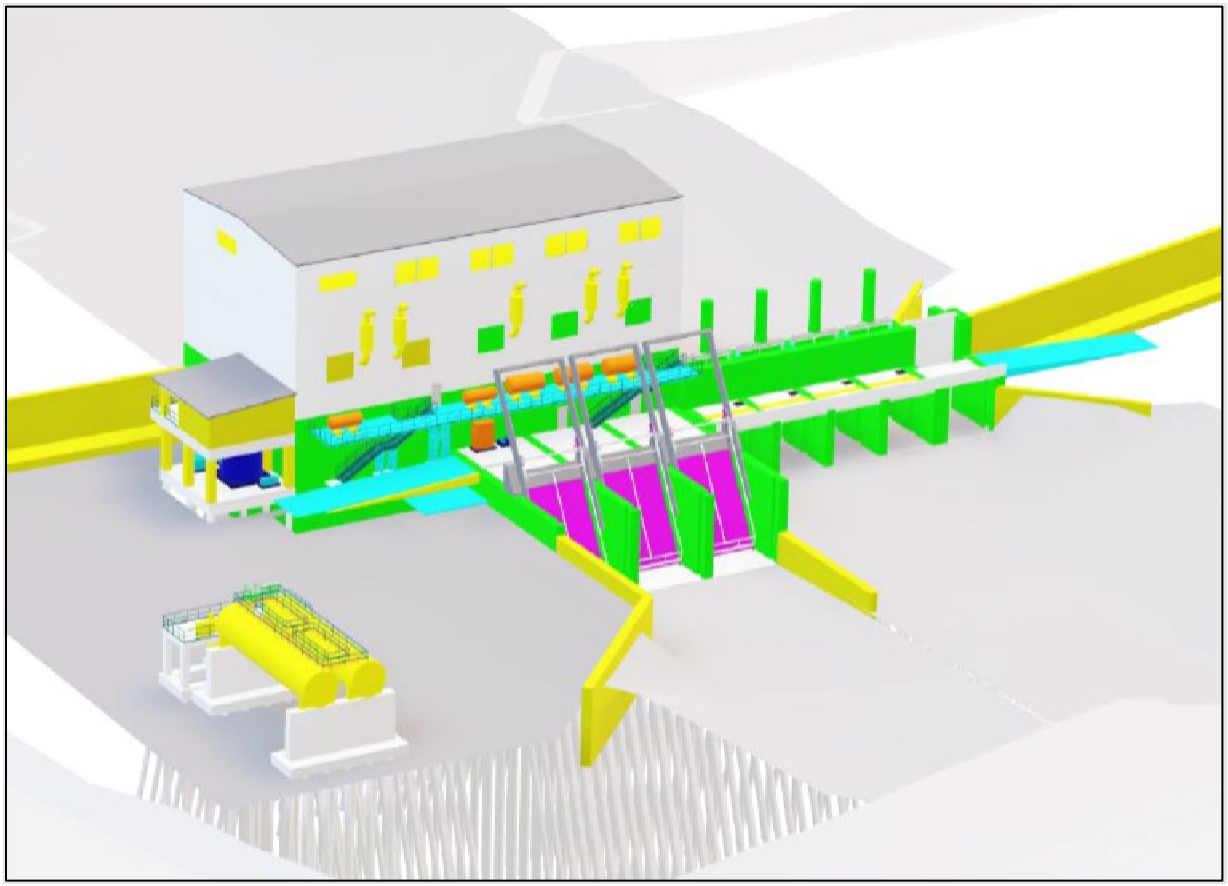


Figure D:10-36. Typical Site Plan of a Pump Station with Large Pumping Capacity

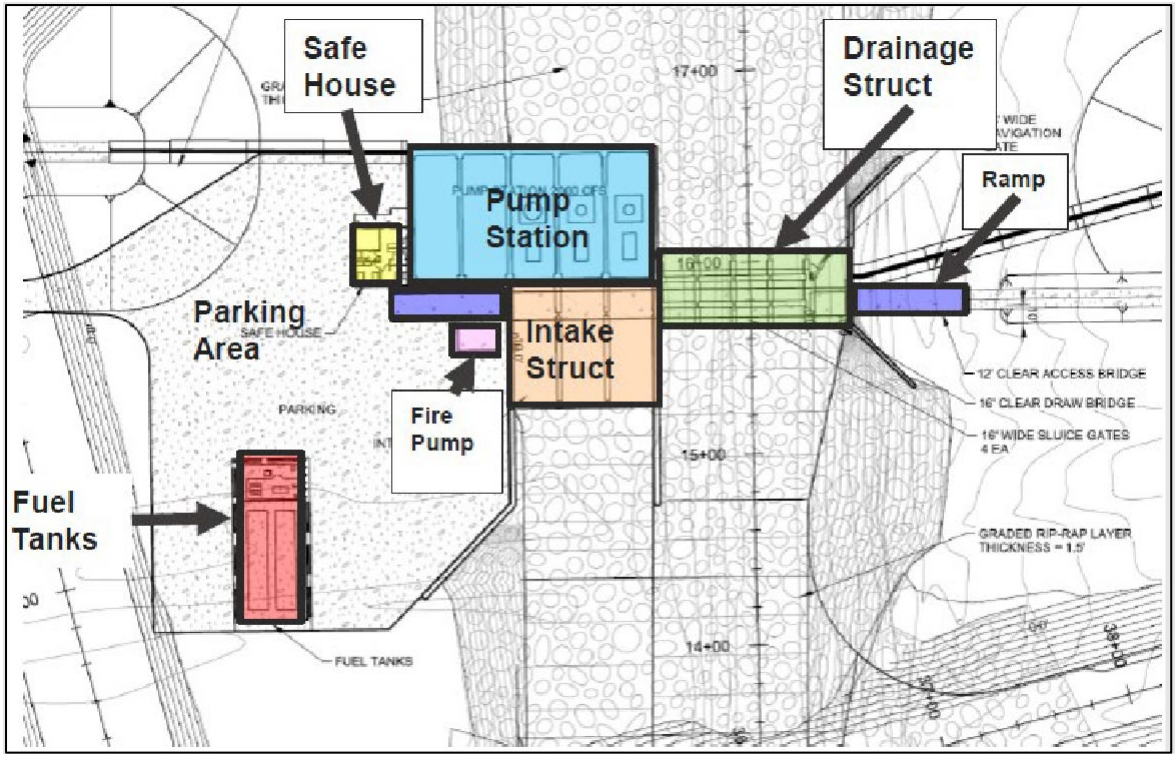


Figure D:10-37. Typical Layout of a Pump Station with Large Pumping Capacity

10.10.1.2 PUMP STATIONS WITH SMALL PUMPING CAPACITY

The RP would include six pump stations with small pumping capacity at sluice gate # 6 on the Bayou Paquet North Tributary (300 cfs), Bayou Paquet lift gate (500 cfs), W-14 Canal (1,000 cfs), sluice gate # 8 at Kings Point (200 cfs), Reine Canal (200 cfs) and at French Branch at the I-10 (450 cfs).

These pump stations would have similar pumping capacities to the Prescott Road Pump Station for the Westshore Lake Pontchartrain Study. The structural quantities from the Prescott Road Pump Station were scaled accordingly to reflect the size of the pump stations for this study.

Figure D:10-38 and Figure D:10-39 show a typical site plan and a typical layout of a pump station with small pumping capacity, respectively.

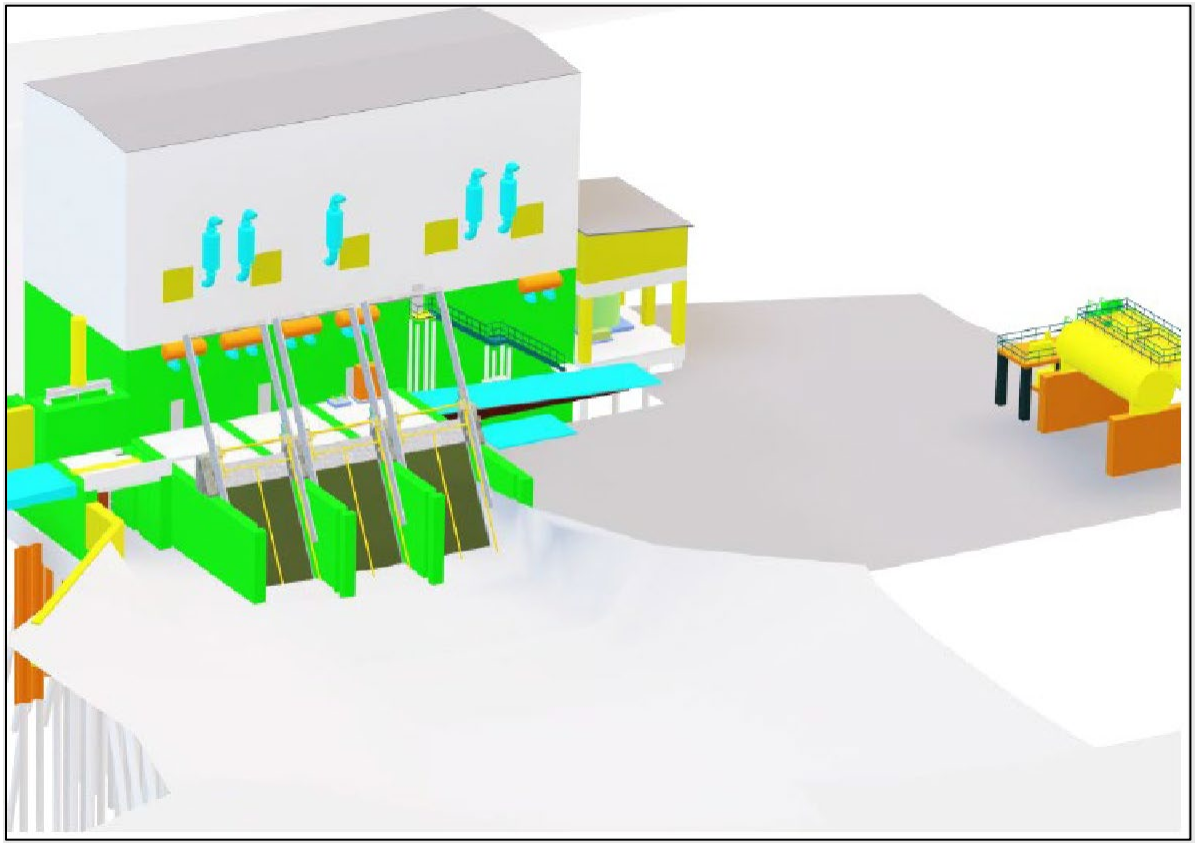


Figure D:10-38. Typical Site Plan of a Pump Station with Small Pumping Capacity

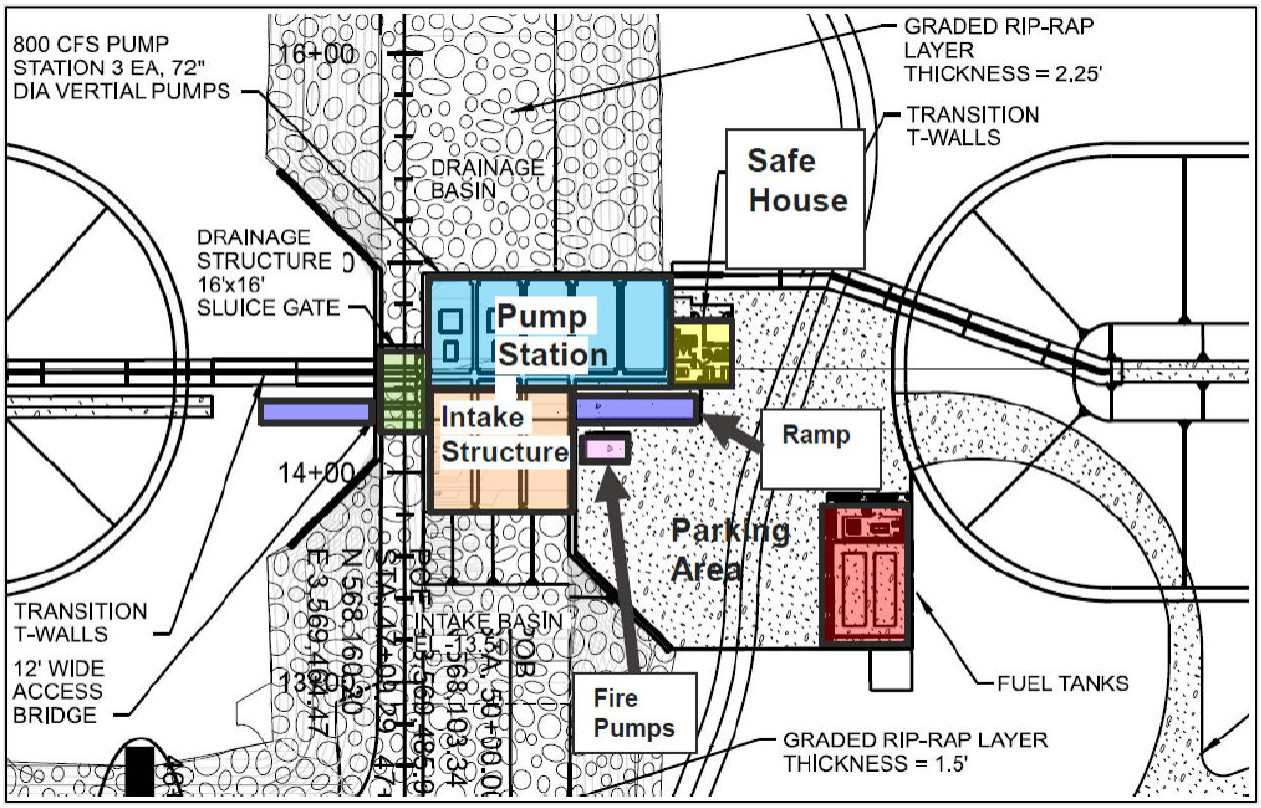


Figure D:10-39. Typical Layout of a Pump Station with Small Pumping Capacity

Note: The schematics for the pump stations with large and small pumping capacities were obtained from a presentation prepared by Stantec.

10.11 STRUCTURAL ASSUMPTIONS

The assumptions for the design of the structural features of the levee and floodwall system of the RP are below.

10.11.1 General Structural Assumptions

10.11.1.1 Structural Geotechnical Assumptions

- All structural design estimates are based on soil characteristics from previous boring investigations.
- No pile load test was assumed for the T-walls, access gates, or control gates; unless component is part of a referenced project.
- Pile load tests were assumed in referenced pump station project components, referenced roller gate project components, and referenced lift gate project

- components. These referenced projects were used to generate scalable quantity estimates.

10.11.1.2 Structural Hydraulic Assumptions

- All structural design estimates are based on hydraulic information for the 50-year design elevations provided by HH&C Branch.
- Unit weight of water for estimates is 62.4 pcf. Unit weight of water will be refined during PED.

10.11.1.3 Structural Superiority

- Structural superiority was not considered for the design elevations of structures (except pump stations). Design estimates are considered conservative and structural superiority will be added during PED.
- Structural superiority was only considered for design and quantity take-offs for pump stations. These designs and quantities were based off of WSLP pump station designs.

10.11.1.4 Seismic Design

- Based on ER 1110-2-1806, Appendix C, Page C1, this project is located in a low- risk zone, so no earthquake load was considered. During PED, the earthquake load will be investigated.

10.11.1.5 Coastal Design

- All structures are in coastal region and HSDRRS design criteria was applied. Concrete Design
- All concrete is assumed to be 4,000 pounds per square foot (psf) normal reinforced concrete with 60 kilo-pound per square inch (ksi) deformed rebar.

10.11.1.6 Steel Design

- Steel strength is assumed to be 50 ksi carbon steel for gates components, piles, and misc. pump station components.

10.11.1.7 Foundation Design

- Pile depth, size, and batter were designed based on largest T-wall stem height per reach.
- Piles are assumed to be 50 ksi 18-inch pipe piles with ½ inch member thickness for all structures except pump stations. Piles spacing in both directions is assumed 5 ft.
- Pile batter is assumed to be 1:2 for all piles, for all structures.
- Pile batter, length, spacing, and size will all be refined at next design level.

10.11.1.8 Slope Pavement

Slope pavement is assumed at transition between T-walls and levees. Slope pavement between gates and levees or other structures is included in gate quantities.

Slope pavement design is based off MRL Carrollton Phase III District Floodwall project.

10.11.1.9 Sheet Piles

All structures are assumed to have PZ 22 steel sheet piles that extend 30 feet deep from base of slab. Where T-walls or other structures transition to levees, sheet piles extend 20 feet into the levee section.

10.11.1.10 Excavation

Area of excavation is assumed to be the footprint of the structure. Depth of the excavation is assumed to be the depth of the slab of the structure.

10.11.1.11 Construction Access

Where access is required, access roads are assumed to be 25 ft with gravel surface.

10.11.1.12 Temporary Retaining Structures

- TRS lengths are assumed to be equal to the perimeter of the footprint of the structure. Depth of sheet pile is assumed to extend 30 feet from base of slab.
- No pile interference check between piles and sheet pile was performed and will be considered during PED.
- Preliminary TRS quantities for large structures were estimated for this study. A revised cost estimate and layout of TRS will be incorporated in PED phase. MVN will comply with all requirements of the Memorandum for Record Design Responsibility for Temporary Retaining Structures per ER 1110-2-8152 (Annex D7).

10.11.1.13 Environmental Design Considerations

- All structures are assumed to be designed to minimize adverse environmental impact.

10.11.1.14 Bypass Channels

- All control structures that are in navigable waterways are assumed to have a bypass channel that matches the original cross-section of the original channel.

10.11.1.15 New Utilities

- Investigations of new utility routing for pump stations and floodgates were not performed during the study. Utility routing and easements for pump stations and

- floodgates will be developed during PED once the exact locations of these structures are finalized.

10.11.2 Vertical Lift Gates

- During PED, ER 1110-2-1150 will be incorporated to vertical lift gate design.

10.11.2.1 T-Walls

10.11.2.1.1 Ground Surface Elevations

- World Imagery information was used to determine ground surface elevations. Estimated design ground surface elevations for each reach were taken as the average elevation for each reach.

10.11.2.1.2 Configuration

- All T-walls are assumed to be pile founded with sheet pile seepage cutoff and a 3 ft thick slab. Slab widths range from 10 ft to 20 ft wide. Stem thicknesses range from 1.5 to 2.5 ft thick. No finish type has been specified.
- Top of slab elevation is assumed to be even with ground surface.

10.11.2.1.3 Design

- T-wall quantities for each derived from governing load condition on largest stem elevation within that reach (18 ft).
- Standard HSDRRS load cases were used in T-wall pile design.
- Governing loading condition is Freeboard (TOW) + Impervious Uplift + Debris.
- No barge impact is assumed and barge activity is not common in the project area.

10.11.2.1.4 Go-bye

- Monoliths are based off similar design of NOV-NF-W-06B.5, Magnolia Pump Station.

10.11.2.1.5 Right of Way Offset

- ROW offset for T-walls is assumed to be 25 ft from the center line of the wall alignment in both directions.

10.11.3 Access Gates

10.11.3.1 Configuration

- Access gates are assumed to be roller gates type unless site conditions require otherwise.
- All gate monoliths are assumed pile founded with sheet pile seepage cutoff and a 3 ft thick slab by 10 feet wide slab. No finish type has been specified.
- Top of slab elevation is assumed to be even with grade elevation.
- Gate monoliths include 3 feet wide x 3 feet deep concrete end columns for attachment of gate hinge and closure hardware. Column heights match the design height of the adjacent T-wall, structure, or levee.

10.11.3.2 Design

- All access gate designs are based on Avondale Shipyard Floodwall roller gate design (2015). The Avondale gate was scaled based on the areas of the gate skin plates.

10.11.3.3 Swing Gates

- Gates at St. Tammany Trace and railroad crossing were assumed to be swing gates due to site constrictions. Swing gate quantities were scaled from referenced roller gate project cited above.

10.11.3.4 Sluice Gate Design

- Sluice gates concrete and steel quantities were scaled from comparable gate structures. Refinement to sluice gate design will be performed during PED.
- All gate sluice gate monoliths are assumed pile founded with sheet pile seepage cutoff and a 3 feet thick slab by 10 feet wide slab, similar to T-wall monolith. No finish type has been specified.
- Top of slab elevation is assumed to be at elevation necessary for hydraulic requirement through the line of protection.

10.11.3.5 Lift Gate Design

- All lift gate concrete, steel, riprap, and excavation quantities are based off (and scaled from) estimated amounts from Texas Coastal lift gate design estimates.
- All lift gates are assumed to be in navigable waterways.

10.11.3.6 Sector Gate Design

- All sector gate concrete, steel, riprap, and excavation quantities are based off (and scaled from) estimated amounts from the Morganza to the Gulf, GIWW West Sector Gate.
- Sector gates are assumed to be in navigable waterways.

10.11.3.7 Pump Stations

10.11.3.7.1 Configuration

- Pump stations were located as determined by hydraulic design of the risk reduction system.

10.11.3.7.2 Design

- Bayou Liberty and Bayou Bonfouca Pump Stations quantities were based off (and scaled from) the 95 percent design of Westshore Lake Pontchartrain Reserve Relief Canal Pump Station.
- All pump stations other than Bayou Liberty and Bayou Bonfouca were based off (and scaled from) the 65 percent design of Westshore Lake Pontchartrain Prescott Road Pump Station.
- All pump stations were scaled based on their capacity in cubic feet per second (cfs)

10.12 RELOCATIONS FOR THE LEVEE AND FLOODWALL SYSTEM

10.12.1 General

Based on the research and investigations conducted as part of the study effort, multiple facilities or utilities are located within the study area of the RP. Refer to the disposition table for assumptions of how these utilities/facilities may be impacted.

10.12.2 Methodology

A review of multiple pipeline databases was used to investigate the facilities located within the project area. A field investigation was not performed for the project area, which would have been more accurate in identifying impacted utilities. However, assumptions of impacted utilities were made based on Google Earth's imagery and street view, City of Slidell GIS data, and historical data of utilities within residential areas. The utilities that could be potentially impacted by the RP are water lines, sewer lines, drainage lines, overhead electrical lines, overhead communication lines, underground conduits, transmission lines and power poles. All utilities are labeled in the Disposition Table D:10-23.

The impacts on the facilities/utilities were based on the assumptions for construction of the subject project. Power poles found to be within the proposed 25 ft utility corridor were assumed to be relocated outside of the corridor during construction. Utilities at proposed vehicular flood gates would be sleeved through the sheet pile cut off wall underneath the flood gate. Other utilities traversing proposed earthen levee must be raised up and over the levee section.

Overhead Electric and Communication Lines were assumed to meet the National Electric Safety Code minimum clearance height criteria over existing ground, which is 18.5 feet at 0 to 22 kV lines and 15.5 feet for communication lines.

10.12.2.1 Railroad Crossing

The RP alignment crosses the Norfolk Southern Railroad. A proposed floodgate in this area would affect the railroad itself and a transmission corridor running parallel to the eastern side of the railroad tracks. USACE would have to meet criteria around these transmission lines to provide necessary clearance for pile driving activity associated with construction of the floodgate and adjacent floodwall. Possible underground utilities servicing the railroad (i.e., communication lines) would be impacted as well.

10.12.2.2 Considerations for the Utility Corridor

Entergy Louisiana, LLC has ROW use requirements pertaining to USACE work around their existing transmission lines, electrical distribution lines and power poles within the project area, that would have to be met to provide clearance for construction activities (i.e., pile driving).

10.12.3 IMPACT TO EXISTING UTILITIES

The pipeline that would impact the RP is shown in Figure D:10-40 and listed in Table D:10- 23.

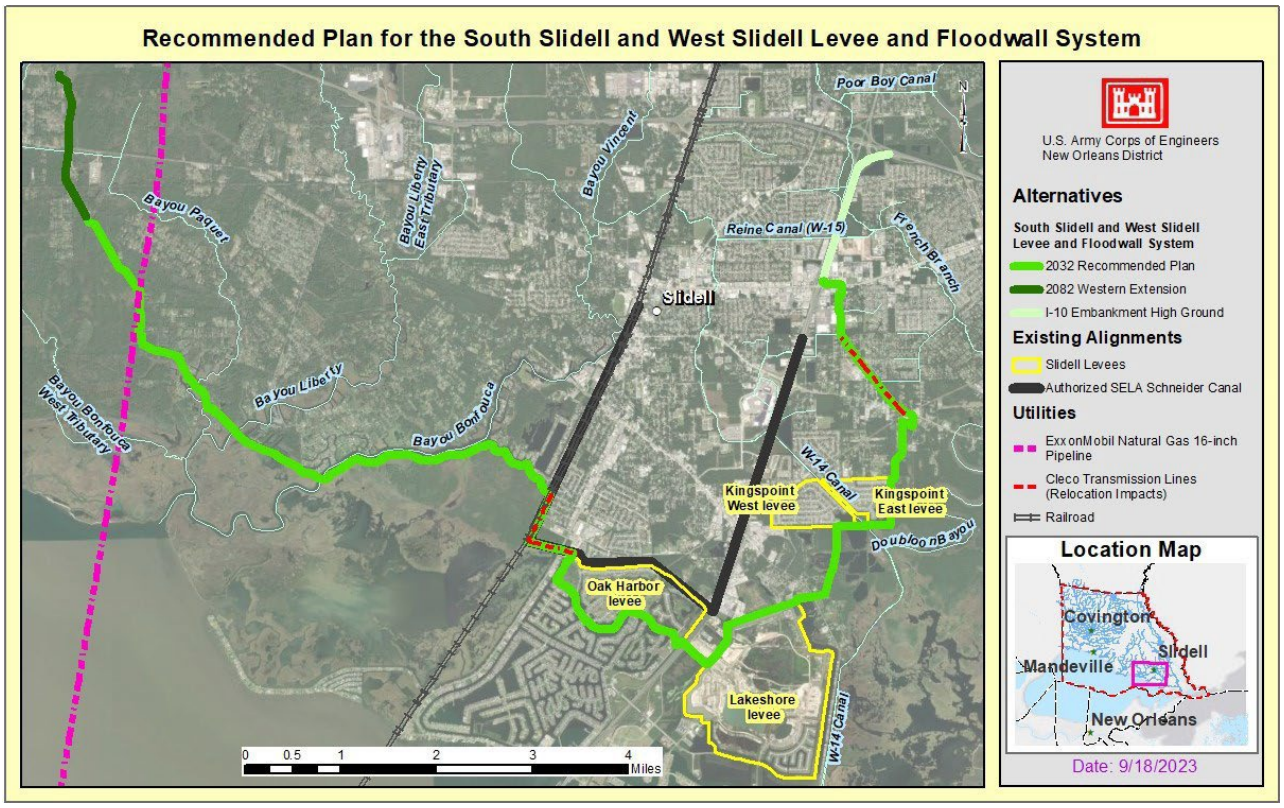


Figure D:10-40. Location of Collins Pipeline (formerly ExxonMobil) and Cleco Transmission Lines

The RP includes a railroad floodgate at Norfolk Southern Railway Corp. railroad tracks to connect the West Slidell portion and the South Slidell portion of the alignment. Preliminary assumptions when crossing a railroad, are that in some instances there are communications lines and/or electrical lines that service the railroad within the railroad ROW. These lines can be used for the railroad signal lights or track switches. There are also cases where utilities run under the railroad to service nearby buildings, such as underground water or sewer lines. The preliminary assumption was that all utilities servicing the Dellwood Pump Station do not run under the railroad, so there could be communication lines or electric lines near the Norfolk Southern Railway Corp. railroad tracks. Further refinement would be needed during PED.

Regarding the construction activities for the alignment in the utility corridor, there are several requirements from CLECO Corporate Holdings, LLC that would have to be met to provide clearance between the construction activities associated with pile driving and the existing utility line on the northeast corner of the new alignment. The PDT used a similar approach to the NOV-NF-W-06b.1 Woodpark Floodwalls in Plaquemines Parish, for the utility corridor relocation assumptions.

Risk reduction would be constructed surrounding a CLECO electrical substation near Hardin Road. Permanent access would be imperative during construction as to not impede maintenance and/or emergency repairs to the substation. Gaps in the risk reduction to maintain access to the substation would require flood fight measures (i.e., sandbags) during a major storm or flood event.

10.12.3.1 Considerations for Assumed Utilities

The following table lists assumed utilities impacted by the proposed levee and floodwall alignment. Google Earth and Google Street view were used to make the most comprehensive assumptions of what utilities exist in the area along with past knowledge of what utilities are common in residential and nonresidential areas.

The City of Slidell provided a GIS file of their utilities in the area. However, the levee and floodwall alignment fall outside of the Slidell municipal limits. Therefore, the city of Slidell’s utilities are not shown to cross the alignment.

There are utilities that serve residents and businesses at and near the alignment. Information from the city of Slidell’s utilities was used to assume the size and materials of utilities traversing the levee and floodwall alignment. These assumptions are the following:

- 10-inch cast iron water lines
- 8-inch galvanized steel (GS) sewer lines
- 30-inch reinforced concrete pipe (RCP) drainage
- 4-inch steel conduits for underground power
- 4inch steel conduits for underground communication lines
- Reinforced concrete box (RCB) for culverts

Utilities at Gause Boulevard were provided by the City of Slidell and those sizes and materials are accurate.

Dispositions for the utilities were determined based on the proposed risk reduction at each location with most utilities being sleeve through sheet pile at flood gates or being relocated up and over at ramps. Determination of how many linear feet of the utilities is affected at ramps can be assumed by the width of ROW needed at a location plus 50 ft.

The study identified numerous utility relocations, potential pipe crossings (either at the levees or floodwalls) and appurtenant structures. During PED, requirements listed in EM 1110-2-2902 would be incorporated into the design of the levee and floodwall system. Depending on the type of crossing or appurtenant structure, the EM requirements could impact the cost of these features, i.e., additional levee fill over a crossing, valving, etc. and these costs would be accounted for during PED.

Table D:10-23. Existing Utilities on the Levee and Floodwall Alignment for the Recommended Plan

Utility	Diameter	Material	Disposition
Hwy 190 road ramp			
Overhead power line			to be raised
overhead communication line			to be raised
underground communication line	4"	steel	up and over
underground sewer line	8"	GS	up and over
underground water line	10"	Cast Iron	up and over
At St. Tammany Bike Trail swing gate			
overhead power line (2)			to be raised
overhead communication line			to be raised
power pole			to be relocated
At Tranquility Road vehicular gate			
no utilities			
At Collins Pipeline (formerly ExxonMobil)			
Collins Liquid Gas pipeline	16"	steel	to be relocated up and over
At Bayou Paquet Road vehicular gate			
overhead power line			to be raised
overhead communication line			to be raised
At Mayer Drive vehicular gate			
overhead power line			to be raised

overhead communication line			to be raised
underground water line	10"	Cast Iron	sleeved thru sheet pile
At railroad flood gate			
underground communication line	4"	steel	sleeved thru sheet pile
underground power line	4"	steel	sleeved thru sheet pile
overhead transmission line			do not disturb
overhead power line			do not disturb
culvert	62"	RCB	do not disturb
Railroad gate to Hwy 11			
transmission poles (13)			do not disturb (3) to be relocated (10)
power pole			to be relocated
underground communication line	4"	steel	sleeved thru sheet pile
At Hwy 11 vehicular gate			
overhead transmission line (2)			do not disturb
overhead power line (2)			to be raised
transmission pole (1)			do not disturb
Hwy 11 to Oak Harbor Blvd			
transmission pole (1)			to be relocated
overhead transmission line (2)			to be relocated
overhead power line (2)			to be raised
power poles (2)			to be relocated
At Oak Harbor Blvd vehicular ramp			
overhead power line			to be raised
underground power line	4"	steel	up and over
light pole (2)			to be relocated
underground drainage	30"	RCP	
power poles (2)			to be relocated
At Island Drive vehicular ramp			
underground drainage	30"	RCP	sleeve thru sheet pile
underground communication line	4"	steel	sleeve thru sheet pile
underground power line	4"	steel	sleeve thru sheet pile
light pole			to be relocated
underground water line	10"	cast iron	sleeve thru sheet pile
At Mariner Cove Blvd vehicular gate			

overhead transmission line			do not disturb
underground sewer line	8"	GS	sleeve thru sheet pile
underground power line	4"	steel	sleeve thru sheet pile
underground water line	10"	cast iron	sleeve thru sheet pile
underground communication line	4"	steel	sleeve thru sheet pile
underground drainage	30"	RCP	sleeve thru sheet pile
At Oak Harbor Blvd vehicular gate			
underground drainage	30"	RCP	sleeve thru sheet pile
underground water line	10"	cast iron	sleeve thru sheet pile
underground power line	4"	steel	sleeve thru sheet pile
underground communication line	4"	steel	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
At Oak Harbor Country Club vehicular gate			
underground water line	10"	cast iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
underground drainage	30"	RCP	sleeve thru sheet pile
underground power line	4"	steel	sleeve thru sheet pile
At Grand Champions Lane ramp			
underground communication line	4"	steel	up and over
underground power line	4"	steel	up and over
underground drainage	30"	RCP	
underground water line	10"	cast iron	up and over
underground sewer line	8"	GS	up and over
At I-10 ramp			
underground drainage	30"	RCP	
overhead power lines (2)			to be raised
overhead communication line			to be raised
Power poles (2)			to be relocated
At Lakeshore Village ramp			
underground power line	4"	steel	up and over
underground water line	10"	cast iron	up and over
underground sewer line	8"	GS	up and over
underground communication line	4"	steel	up and over
At Old Spanish Trail vehicular gate			
overhead power line			to be raised

overhead communication line			to be raised
At Nunez Road ramp			
overhead power line			to be raised
Nunez Road to W-14 gate			
Power poles (10)			to be relocated
At W-14 gate			
overhead power line			to be raised
power pole			to be relocated
At Hardin Road electrical substation (gate)			
overhead power line (3) (both sides)			to be raised (on substation side only)
overhead communication line (3)			to be raised (on substation side only)
Overhead Transmission line			do not disturb
power poles (3)			to be relocated
At US Hwy 190E (substation) (gate)			
overhead power line (3)			to be raised
underground communication line	4"	steel	sleeve thru sheet pile
overhead communication line			to be raised
Overhead Transmission line			do not disturb
underground water line	10"	Cast Iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
At Transmission Line Corridor			
overhead transmission line			do not disturb
Transmission poles			do not disturb
At South Holiday Drive vehicular gate			
overhead transmission line			do not disturb
overhead power line			to be relocated
overhead communication line			to be relocated
Holiday Drive to Yaupon Floodwall			
power poles (4)			to be relocated
overhead transmission line			do not disturb
At North Holiday Drive vehicular gate			
overhead transmission line			do not disturb
underground communication line	4"	steel	sleeve thru sheet pile

underground water line	4"	Cast Iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
along Carol Drive			
overhead power line			do not disturb
underground drainage	30"	RCP	do not disturb
underground communication line	4"	steel	do not disturb
overhead communication line			do not disturb
underground water line	10"	cast iron	do not disturb
At Natchez Drive gate			
overhead power line			to be raised
overhead communication line			to be raised
underground communication line	4"	steel	sleeve thru sheet pile
underground water line	10"	Cast Iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
At Kisatchie Drive vehicular gate			
overhead power line			to be raised
overhead communication line			to be raised
underground communication line	4"	steel	sleeve thru sheet pile
underground water line	10"	cast iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
At Manzella Drive vehicular gate			
overhead communication line			to be raised
overhead power line			to be raised
underground communication line	4"	steel	sleeve thru sheet pile
underground water line	10"	cast iron	sleeve thru sheet pile
underground sewer line	8"	GS	sleeve thru sheet pile
At Gause ramp			
water main	10"	C900	up and over
water main	12"	PVC	up and over
underground sewer line 8"	8"	PVC	up and over
underground power line	4"	steel	up and over
underground communication line	4"	steel	up and over
At ramp for business on N side of Gause			
underground water line	6"	PVC	up and over

underground power line	4"	steel	up and over
underground communication line	4"	steel	up and over
At ramp for I-10 service road near Gause			
underground water line	6"	PVC	up and over
underground power line	4"	steel	up and over
underground communication	4"	steel	up and over
At I-10 on ramp			
no utilities			
At Reine Canal sluice gate			
guy poles (2)			to be relocated
underground communication line	4"	steel	to be relocated
At French Branch sluice gate			
underground communication line	4"	steel	to be relocated

10.13 SUMMARY FOR 50-YEAR PERIOD OF ANALYSIS

Construction of the levee and floodwall system would impact a total of approximately 450 acres of permanent ROW and 101 acres of temporary ROW, and it would require approximately 7,239,000 cubic yards of fill, including fill material required for future levee lifts (estimates include a 30 percent contingency) for the 50-year period of analysis. Table D:10- 24 provides a summary of the attributes of the South and West Slidell Levee and Floodwall System.

Table D:10-24. Summary of South Slidell and West Slidell Levee and Floodwall System for the 50-year Period of Analysis

Attribute	South Slidell and West Slidell Levee and Floodwall System
Total Length of Alignment	18.5 miles (97,700 ft)
Length of Floodwall	3.5 miles (18,200 ft)
Length of Earthen Levee	15 miles (79,500 ft)
Temporary Acres of Construction for Levee and Floodwall system[1]	101 acres
Permanent Acres for Levee and Floodwall system	450 acres

Hydraulic Design Elevation Range (Dependent on location)	13.5 to 16 (year 2032) 17.5 to 20 (year 2082)
Pump Stations	8
Sluice Gates/Lift Gates	13
Number of Vehicular Floodgates	18
Number of Pedestrian Floodgates	1
Number of Railroad Gates	1
Number of Road Ramps	6 (includes the I-10 near Oak Harbor)
Fill (Borrow Material) Required	7,239,000 cubic yards

[1] The staging areas required during initial construction of the levee alignment would be the same staging areas required for construction of future levee lifts. For Real Estate purposes, the staging areas were included in the permanent ROW.

Tables D:10-25 to D:10-27 provide a summary of the necessary staging areas and permanent ROW required for construction of the levee, floodwall segments, pump stations, gates, road ramps, and access roads.

Table D:10-25. Summary of Staging Areas and Permanent ROW for the Levee and Floodwall System of the Recommended Plan

SUMMARY of STAGING AREAS AND PERMANENT ROW		
Levees	**Staging Areas (Acres)**	**Permanent ROW (Acres)**
Western High Ground Tie In	2	30
West Slidell	8.5	240
South Slidell (includes 23 acres for I-10)	30	120
Sub-Total for Levees	**40.5[1]**	**390**
Floodwall Segments		
Western High Ground Tie In	NA	NA
West Slidell	0	4
South Slidell	0.5	23
Sub-Total for Floodwall Segments	**0.5[2]**	**27**
Floodgates and Pump Stations		
Western High Ground Tie In	1.5	2.5
West Slidell	11	21
South Slidell	3.75	6.25
Sub-Total for Floodgates and Pump	**16.25**	**29.75**

Stations		
Vehicular, Pedestrian, and Railroad Gates		
Western High Ground Tie In	1.5	1.25
West Slidell	2.25	0
South Slidell	11.25	0
Sub-Total for Vehicular, Pedestrian, and Railroad Gates	15	1.25
Road Ramps		
Western High Ground Tie In	0.5	0
West Slidell	0	0
South Slidell	2	0
Sub-Total for Road Ramps	2.5	0
Access Roads - New3		
Western High Ground Tie In	0	0
West Slidell	0	0.84
South Slidell	0	1.75
Sub-Total for New Access Roads	0	2.59
Access Roads - Existing		
Western High Ground Tie In	0	0
West Slidell	0	0
South Slidell	15.8	0
Sub-Total for Existing Access Roads	25.7	0
Sub-Total for Access Roads	25.7	2.59
Total for Levee and Floodwall System for 50- year Period of Analysis	101	450

[1] The staging areas required during initial construction of the levee alignment would be the same staging areas required for construction of future levee lifts. For Real Estate purposes, the staging areas were included in the permanent ROW.

For floodwall segments, staging areas would be included in the 80-ft wide permanent ROW. Except for the utility corridor on South Slidell, in the vicinity of Northshore Drive, there would be a 0.5 acre staging area outside of the 80-ft wide corridor.

New access roads (acres) do not include areas where the access is within the permanent ROW.

10.14 TOTAL PROJECT COSTS FOR LEVEE AND FLOODWALL SYSTEM

Costs for the levee and floodwall system, which is the Coastal Storm Risk Management (CSRM) portion of the RP are provided in the Total Project Cost Summary in Annex D8

Cost Engineering for the Recommended Plan. The relocations costs are included in the Summary. Annex D10 contains the construction schedule for the CSRM.

10.15 ACCESS FOR THE RECOMMENDED PLAN

Construction access and staging areas would be needed along the alignment for all elements of the RP. Project access post-construction for future maintenance would be needed for all elements except the nonstructural home raisings. Permanent access would include access to the levee alignment. Further development of access would be prepared during PED.

Existing public roads would be used for access to the maximum extent as possible. In locations where access cannot be achieved via existing roadways, a new road would be constructed. Construction of new roads would require permanent ROW.

10.16 ROW CRITERIA AND ACCESS ROUTES FOR INITIAL CONSTRUCTION AND FUTURE LIFTS

Table D:10-26 lists the ROW width required per levee or floodwall segment. The width includes a 15 ft of vegetation free zone (VFZ) on each side of the levee and floodwall segments.

Table D:10-26. Typical Widths of Permanent ROW for Levee and Floodwalls Segments

Levee and Floodwall Segments	Width of Permanent ROW (ft)*
Western High Ground Tie-in	160
West Slidell	300
South Slidell	160
Floodwall Segments	80
Access Roads	NA
Floodwall on the eastern end of the South Slidell alignment (north of Gause Boulevard). No VFZ.	50 ft

*(Includes 15-ft VFZ on both sides)

10.16.1 LEVEES

The following criteria applies to initial construction and future levee lifts.

10.16.1.1 STAGING AREAS FOR LEVEES

The staging areas for levee construction would be included in the temporary ROW. The staging areas for levee initial construction would be the same staging areas used for future lifts of the levee. No additional ROW would be needed for future lifts.

This Temporary ROW is considered “permanent easement” for Real Estate purposes due to future lifts.

For staging areas, crushed stone would be placed (assuming crushed stone for vehicle parking/staging and for path from road to area).

Surveys would be taken prior to disturbing the staging area. Any trees would be removed and hauled away to an approved facility. Material would be processed on-site. Areas would be restored to pre-construction elevation after construction activities are complete.

10.16.1.2 ACCESS FOR LEVEES

There are locations where an existing road would be used for access. In other locations, a new road would be built. New access roads would be a 40-ft wide footprint (consisting of a 25 ft ROW for the access road itself and a 7.5-ft width for VFZ on both sides of the road).

10.16.1.3 MATERIALS FOR STAGING AREAS AND ACCESS ROADS

For staging areas and new access roads for levee construction, not including area for material processing during levee construction, a 7-inch depth of stone, and 115 lbs/cubic ft stone weight was assumed. This assumption does not apply to the access road on the railroad tracks.

10.16.1.3.1 STAGING AREAS LEVEE CONSTRUCTION ON THE INTERSTATE 10

Staging areas for the construction of the I-10 crossing would be in the median and within the La DOTD ROW. No additional staging areas would be needed.

10.16.1.4 LEVEE CONSTRUCTION ON THE BBMNWR

10.16.1.4.1 STAGING AREA FOR LEVEE CONSTRUCTION

There would be one 2-acre staging area on the reach on the BBMNWR land that would be considered a temporary easement. The staging area would be located off the BBMNWR and would be used to process the material prior to building the levee. Staging areas would be required to be continuously accessible. Any trees would be removed and hauled away to an approved facility. The area would be restored to pre-construction elevation that existed prior to impacting the site due to construction activities.

10.16.1.4.2 ACCESS FOR LEVEE CONSTRUCTION

For levee construction on the BBMNWR land (from Bayou Bonfouca to the railroad tracks), the ingress and egress would be at Norfolk Southern railroad tracks on the east side of Bayou Bonfouca and existing roads on the west side. Access to the BBMNWR is one way in, one way out at the Norfolk Southern railroad tracks. There would be no two-way traffic, so coordination of vehicles entering the site would be required. USACE would need to obtain permission from the railroad owner (Norfolk Southern Railway Corp.) prior to construction.

An access road would be constructed on the protected side of the ROW between the proposed crown of the levee and Bayou Bonfouca. The access road would be a temporary road. Once construction is complete, the area would be cleared of vegetation within the ROW and graded to drain away from the levee. Access for future inspections would be done by driving on the crown of the levee.

10.16.2 RAMPS

Ramps would be constructed to the 2082 hydraulic elevations during the initial construction of the levee alignment. The temporary ROW (during construction) for the construction of ramps would be as follows:

- For ramps adjacent to levees or floodwall segments:
    - Temporary ROW- use 0.5 acres for staging area

10.16.3 FLOODWALLS

The width for permanent ROW for initial construction, which includes 15 ft of VFZ is as follows:

- Floodwall segments - 80 ft (includes staging areas)
- North of Gause Boulevard (South Slidell) – floodwall segment – 50 ft (excludes 30 ft VFZ which is not applicable at this location).

10.16.3.1 STAGING AREAS FOR CONSTRUCTION OF FLOODWALLS

It was assumed that during construction of the floodwall segments, the staging areas would be within the 80-ft-wide ROW. One exception is at the utility corridor in South Slidell. In the vicinity of Northshore Lane, there would be a 0.5 acre staging area outside of the 80-ft wide corridor.

Table D:10-28 shows the temporary and the permanent ROW for the floodwall segments.

10.16.3.2 ACCESS ROADS FOR CONSTRUCTION OF FLOODWALLS

Existing public roads would be used for access to the maximum extent possible. In locations where access cannot be achieved via existing roadways, a new road would be constructed. Construction of new roads would require permanent ROW.

All listed access routes to access structures would each have a 40-ft wide footprint (consisting of a 25 ft ROW for the access road itself and a 7.5-ft width for VFZ).

10.16.4 STRUCTURES

The temporary ROW during initial construction of structures, and permanent ROW that would be needed for the pump stations and floodgates would be as follows:

- For two larger pump stations and floodgate complexes (Bayou Liberty and Bayou Bonfouca):
    - Temporary ROW during initial construction - staging area- 4 acres
    - Permanent ROW - 8 acres
- For the rest of the pump stations (small) and floodgate complexes:
    - Temporary ROW during initial construction - staging area - 0.75 acres
    - Permanent ROW - 1.25 acres
- For smaller floodgates, lift gates, and control gates:
    - Temporary ROW during initial construction - staging area - 0.75 acres
    - Permanent ROW - 1.25 acres
- For pedestrian gate at Tammany Trace in West Slidell:
    - Temporary ROW during initial construction - staging area - 0.75 acres
    - Permanent ROW- 1.5 acres
    - Note that there are separate temporary and permanent ROW for the sluice gate and the pedestrian gate at Tammany Trace Bike Trail in West Slidell.
- For vehicular and railroad gates:
    - Temporary ROW during initial construction - staging area- 0.75 acres
    - Permanent ROW - none
    - This is a change from the original assumptions during the Draft TSP analysis, of using the levee ROW as the staging area.

10.16.4.1 ACCESS ROADS FOR STRUCTURES

Existing public roads would be used for access to the maximum extent as possible. In locations where access cannot be achieved via existing roadways, a new road would be constructed. Construction of new roads would require permanent ROW.

New access roads would be a 40-ft wide footprint (consisting of a 25 ft right-of-way for the access road itself and a 7.5-ft width for VFZ on both sides of the road. Access roads would be constructed using crushed stone for the road surface.

10.16.4.2 PERMANENT ROW FOR THE EASTERN TERMINUS BY THE INTERSTATE-10

For Reine Canal and for French Branch, the permanent ROW would be within the existing highway ROW.

10.17 SUMMARY OF ROW FOR RAMPS

For the six ramps, the temporary ROW (during construction) would be as shown in Table D:10- 27.

Table D:10-27. ROW for Ramps

Ramps	Temporary ROW (Staging Area (Acres))
Western High Ground Tie-in for 2082	
Highway 190	0.5
West Slidell	
N/A	N/A
South Slidell	
Oak Harbor Boulevard	0.5
Islander Drive	0.5
Grand Champions Lane	0.5
I-10 would be raised to ramp over the new levee section	0
I-10 On-Ramp	0.5
Total for Ramps	2.5

10.18 SUMMARY OF ROW FOR FLOODWALL SEGMENTS

For the fourteen floodwall segments, the temporary ROW (during construction) would be as shown in Table D:10-28.

Table D:10-28. ROW for Floodwall Segments

Floodwall Segments	**Temporary ROW (Staging Area (Acres)**
Western High Ground Tie-in for 2082	
N/A	
West Slidell	
Properties west of Doucette Road	0
North Side Bayou Paquet Drive	0
Bayou Paquet/Mayer Drive	0
South Slidell	
Front Street/Railroad	0
Mariners Cove Boulevard	0
Oak Harbor Country Club	0
Old Spanish Trail	0
Esprit du Lac Street	0
Substation Floodwall	0
Highway 190 Business	0
Utility Corridor	0.5
Hollywood Drive to Yaupon	0
Manzella Drive to Gause Boulevard	0
Gause Boulevard to I-10	0
Total	0.5

10.19 SUMMARY OF ROW FOR FLOODGATES AND PUMP STATIONS

For the floodgates and pump stations, the temporary ROW (during construction) and the permanent ROW would be as shown in Table D:10-29.

Table D:10-29. ROW for Floodgates and Pump Stations

Floodgates and Pump Stations	Pump Station	Pumping Capacity (cfs)	Temporary ROW (Staging Area (Acres))	Permanent Area (Acres)
Western High Ground Tie-in for 2082				
Sluice gate near Shannon Drive	No	N/A	0.75	1.25
Sluice gate at Tammany Trace	No	N/A	0.75	1.25
West Slidell				
Sluice Gate # 7 (Near CC Road)	No		0.75	1.25
Sluice Gate # 6 (Bayou Paquet North Tributary)	Yes	300	0.75	1.25
Bayou Paquet Navigable Gate	Yes	500	0.75	1.25
Bayou Liberty Navigable Gate	Yes	1800	4	8
Bayou Bonfouca Navigation Gate	Yes	2000	4	8
Sluice Gate # 2 (Bayou Bonfouca Sluice Gate)	No		0.75	1.25
South Slidell				
W-14 Canal Navigable Gate	Yes	1000	0.75	1.25
Sluice Gate # 8 (Kings Point East)	Yes	200	0.75	1.25
Sluice Gate # 10 (Near East Terminus)	No		0.75	1.25
Reine Canal	Yes	200	0.75	1.25
French Branch at I-10	Yes	450	0.75	1.25
Total			**16.25**	**29.75**

10.20 SUMMARY OF ROW FOR VEHICULAR, PEDESTRIAN, AND RAILROAD GATES

For the vehicular, pedestrian and railroad gates, the temporary ROW (during construction) and the permanent ROW would be as shown in Table D:10-30.

Table D:10-30. ROW for Vehicular, Pedestrian and Railroad Gates

Gate Location	Temporary ROW (Staging Area (Acres))	Permanent ROW (Acres)
Western High Ground Tie-in for 2082		
Tammany Trace Pedestrian Gate	0.75	1.5
Tranquility Road Vehicular Gate	0.75	0
West Slidell		
Bayou Paquet Road Vehicular Gate	0.75	0
Mayer Drive Vehicular Gate	0.75	0
Railroad Floodgate	0.75	0
South Slidell		
Hwy 11 Vehicular Gate	0.75	0
Mariners Cove Floodwall and Vehicular Gate	0.75	0
Oak Harbor Vehicular Gate	0.75	0
Oak Harbor Country Club Vehicular Gate	0.75	0
Old Spanish Trail Floodgate (Hwy 433)	0.75	0
Hardin Road Substation Gate	0.75	0
Hwy 190-B Floodgate (East Floodwall)	0.75	0
South Holiday Drive Vehicular Gate	0.75	0
North Holiday Drive Vehicular Gate	0.75	0
Jaguar Drive Vehicular Gate	0.75	0
Natchez Drive Vehicular Gate	0.75	0
Kisatchie Drive Vehicular Gate	0.75	0
Manzella Drive Vehicular Gate	0.75	0

Gause Boulevard Vehicular Gate	0.5	0
Private Road Vehicular Gate	0.5	0
I-10 Service Road Vehicular Gate	0.5	0
Total	**15**	**1.5**

10.21 ROW, ACCESS AND STAGING AREA MAPS

Refer to Annex D6 for maps that provide the ROW for the features of the alignment from the RP starting from the northwest.

10.22 HYDRAULIC DESIGN ELEVATIONS

The following criteria was used for the hydraulic design elevations of the features of the RP. Refer to Table D:10-31 and Figure D:10-41. Additional information may be found in Appendix E Hydrologic and Hydraulics.

10.23 GEOTECHNICAL INVESTIGATIONS FOR THE RECOMMENDED PLAN

10.23.1 GEOLOGY

The amount of geologic information available for use in the study was limited. Neither geologic profiles nor geologic maps could be obtained for the Final Array of Alternatives. Geological conditions are inferred from the limited boring information obtained from geotechnical reports prepared by Eustis Engineering, LLC for various projects in St.

Tammany Parish.

A geologic profile should be developed for the project alignment during PED, when a thorough geotechnical investigation is planned to be conducted along the alignment. Refer to Section 5.2 of the Final Array for additional information.

LEGEND:

Yellow represents present (existing) condition levee elevations **Orange** represents future-year levee elevations, for levees **Blue-gray** represents future year elevations for wall/gate

Table D:10-31. Hydraulic Design Elevations

Levee	Design Year	Return Period	Levee/ Flood wall	Levee Elevation (ft. NAVD 88)	Longitude	Latitude	FID (Figure Below)
Lacombe	2032	100.0	1.0	**13.5**	-89.8692	30.2825	0
Bayou Bonfouca	2032	100.0	1.0	**13.5**	-89.8359	30.2568	1
Slidell West Levee	2032	100.0	1.0	**13.5**	-89.7997	30.2566	2
I-10 Crossing	2032	100.0	1.0	**15.0**	-89.7729	30.2280	3
Slidell East Levee - Nunez Rd	2032	100.0	1.0	**16.0**	-89.7476	30.2409	4
Oak Harbor South	2032	100.0	1.0	**14.0**	-89.7887	30.2333	5
Slidell Northeast Levee	2032	100.0	1.0	**16.0**	-89.7346	30.2472	6
Lacombe	2082	100.0	1.0	**17.5**	-89.8692	30.2825	0
Bayou Bonfouca	2082	100.0	1.0	**17.5**	-89.8359	30.2568	1
Slidell West Levee	2082	100.0	1.0	**17.5**	-89.7997	30.2566	2
I-10 Crossing	2082	100.0	1.0	**18.5**	-89.7729	30.2280	3
Slidell East Levee - Nunez Road	2082	100.0	1.0	**20.0**	-89.7476	30.2409	4
Oak Harbor South	2082	100.0	1.0	**17.5**	-89.7887	30.2333	5
Slidell Northeast Levee	2082	100.0	1.0	**20.0**	-89.7346	30.2472	6
Northeast Tie-in	2082	100.0	0.0	**18.5**	-89.7289	30.2610	7
W Doucette Road Gate	2082	100.0	0.0	**16.0**	-89.8678	30.2795	8
West Bayou Paquet Gate	2082	100.0	0.0	**16.0**	-89.8652	30.2745	9
Bayou Paquet Gate	2082	100.0	0.0	**16.0**	-89.8574	30.2639	10
Bayou Liberty Gate	2082	100.0	0.0	**16.0**	-89.8521	30.2637	11
Bayou Bonfouca Gate	2082	100.0	0.0	**16.0**	-89.8365	30.2573	12
Slidell Railroad	2082	100.0	0.0	**16.5**	-89.7982	30.2511	13
Lakeshore Estates Northeast Gate	2082	100.0	0.0	**18.5**	-89.7533	30.2374	14
Kings Point W14 Canal Gate	2082	100.0	0.0	**18.5**	-89.7434	30.2498	15

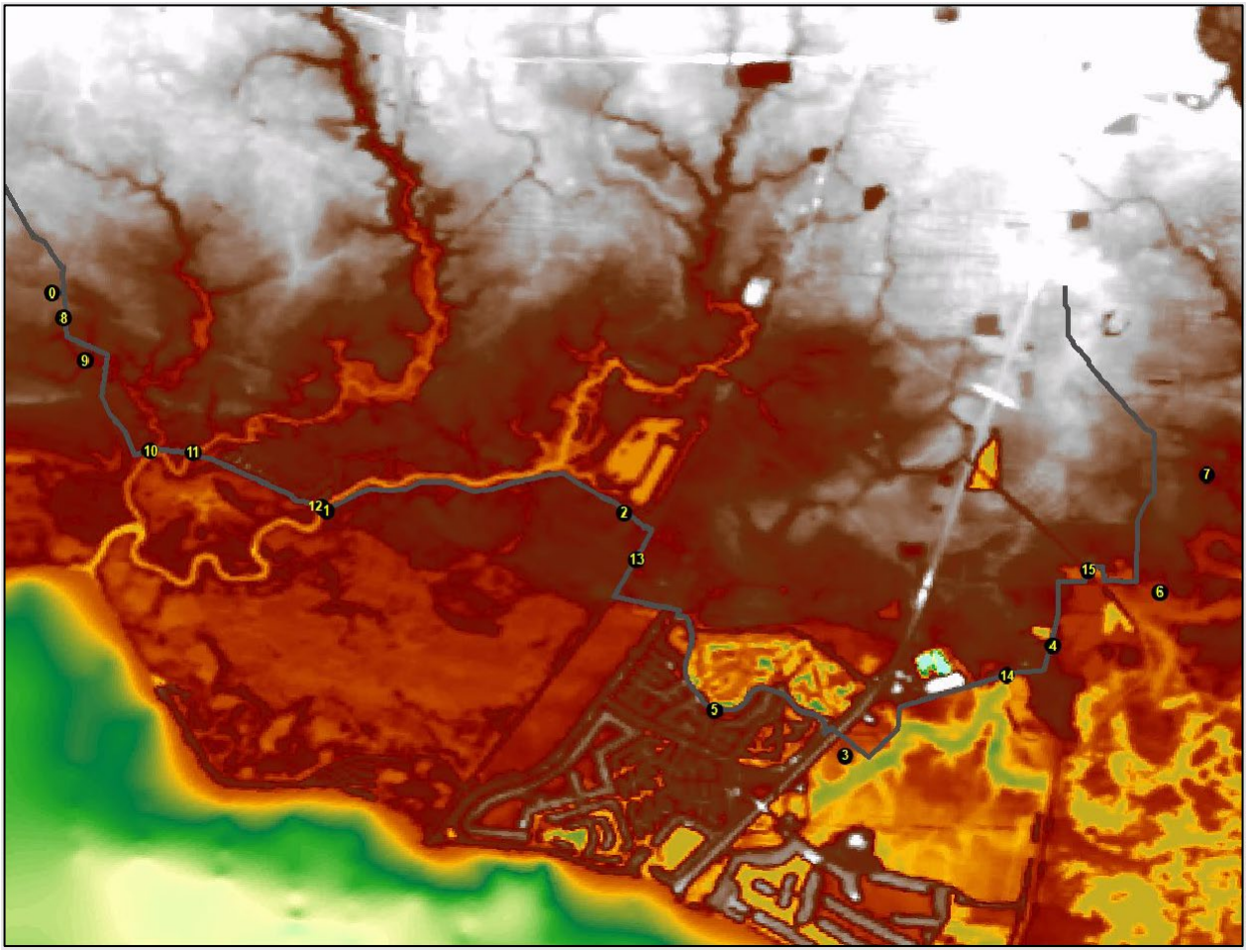


Figure D:10-41. Map of Hydraulic Reaches

10.23.2 BACKGROUND

This section summarizes the preliminary geotechnical design results for the RP. The RP consists of a combination of portions of the Slidell levee alignment proposed in Alternative 5 (excluding the western portion of that alignment) and the West Slidell levee alignment proposed in Alternative 6a (excluding the northwestern portion of that alignment). The two alignments are connected by a new railroad gate across the existing Norfolk Southern Railway Corp. railroad tracks.

The two alignments or segments combined (both South Slidell and West Slidell) make up the RP. The RP is divided into four hydraulic reaches. Geotechnical analyses also use the same four reaches for analyses. The west Slidell section is comprised only of the West Slidell west levee section (Reach 1), which stretches west from Dellwood pump station until it ties into high ground. The West Slidell high ground (Reach 1A) is already above 2032 design grade elevation. The floodwall tracks north to south parallel to the Norfolk Southern Railway Corporation railroad tracks. The South Slidell section has the other three reaches: Oak Harbor, I-10 Crossing, and the Slidell East and Northeast reach. Tracking eastward from the floodwall is the Oak Harbor levee section (Reach 2), which tracks along part of the existing Oak Harbor levee section to I-10. The part of the alignment that crosses underneath I-10 is called I-10 crossing (Reach 3). The fourth part of the alignment ties to an existing portion of the Lakeshore Estates ring levee and turns northward and ties to the King’s Point ring levees. This is the Slidell East and Northeast (Reach 4) part of the alignment. Reach 4 ties into the Northeast Tie-in floodwall, which completes the alignment in an area with less ROW. These reaches can be found in Figure D:10-42.

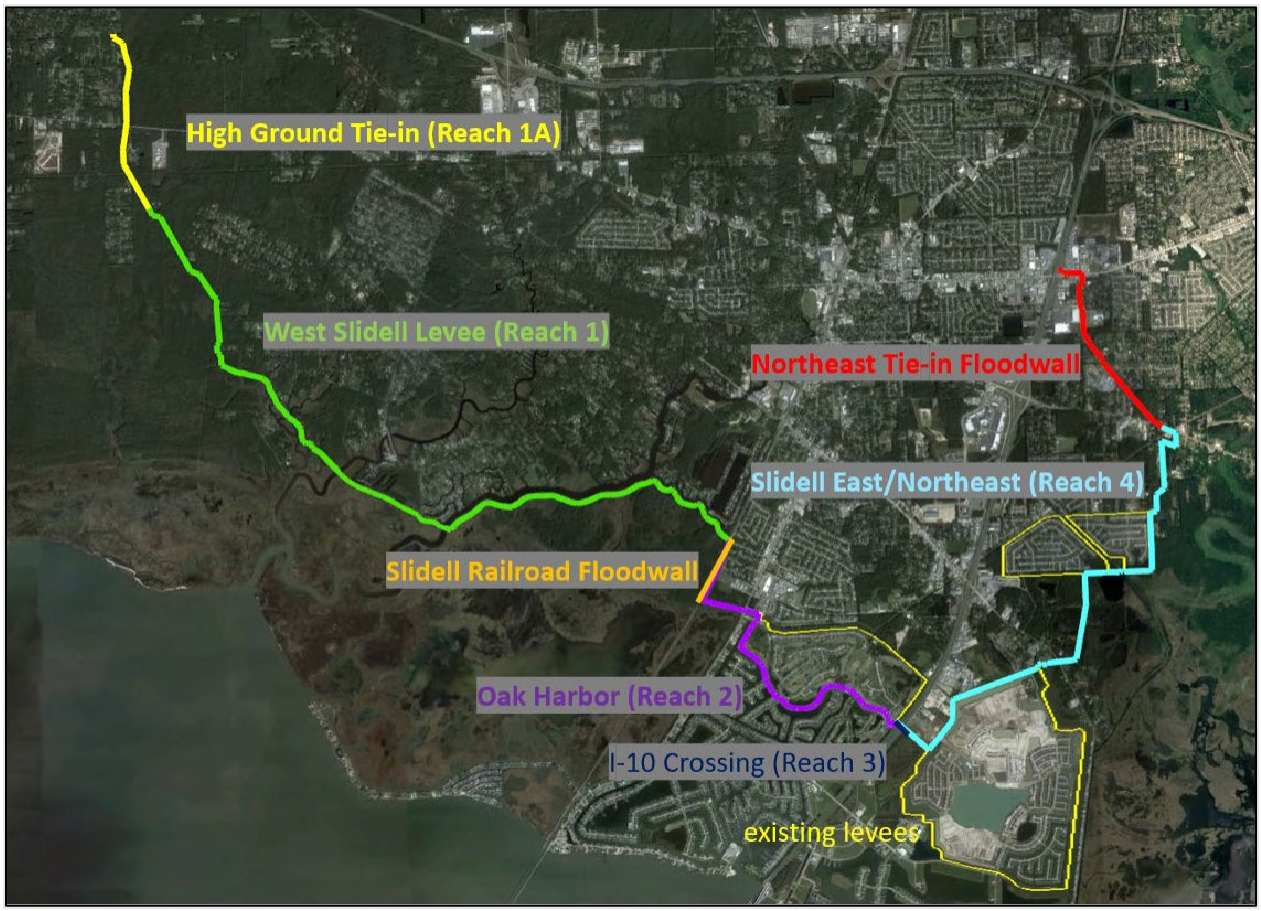


Figure D:10-42. Reach Breakdown for the Recommended Plan

The amount of information available for geotechnical investigations, associated testing, and geologic profiles was limited. Soil borings were not taken, and soil testing was not performed for this study. Soil unit weights and shear strengths were assigned based on USACE geotechnical experience in the region and limited boring information in the vicinity of the RP alignment.

10.23.3 FURNISHED INFORMATION AND LOCATIONS

USACE has very limited boring information in St. Tammany Parish. Eustis Engineering LLC provided all the job reports for previously completed projects in St. Tammany Parish.

Projects that had boring information were plotted based on the project location. Borings of appropriate depth were considered for use for this study. The closest available projects to levee or structural features that had geotechnical investigations were used to develop parameters. Levees were not broken into individual geotechnical reaches. Sections analyzed were based on hydraulic reaches. Boring data located within the

vicinity to study features with enough data of sufficient depth was used to get soil information for development of parameters. Parameters were used throughout an entire levee or structural feature. Future analyses should take site specific information and a more refined parameter selection process should be developed. This could result in a more refined geotechnical design involving more geotechnical reaches. The RP comprises of a levee alignment that is approx. 18.5 miles long, so a levee feature may have multiple reaches requiring the full scope of geotechnical analyses instead of one per feature as done for the analysis. The lack of subsurface information and the number of assumptions required as a result are noted in the risk register.

The RP alignment includes seven hydraulic reaches and are captured in four different geotechnical areas of study. These are the West Slidell area, the Oak Harbor area, the I-10 crossing, and Slidell east and northeast. When applicable, the closest Eustis Engineering LLC report that had viable boring information was used. The Eustis project number, boring information including number and depth, proximity to feature and assumptions are contained within.

10.23.3.1 Slidell West Levee (Reach 1)

The levees would commence on the south side of U.S. Highway 190 and South Tranquility Road, and on the eastern side of Pineridge Road. The alignment would track southward and would run on the west side of Tranquility Road (CC Road) and then it would turn in the southeast direction crossing Bayou Paquet Road and would stay on the east side of Bayou Paquet Channel to avoid impact to the BBMNWR. The alignment would cross Bayou Paquet and Bayou Liberty and would continue eastward on the northside of the BBMNWR. The alignment would cross Bayou Bonfouca and would continue on the south bank of the bayou (northern side of the BBMNWR) until reaching the Norfolk Southern Railway Corp railroad tracks west of U.S. Highway 11 in the vicinity of Dellwood Pump Station in Slidell.

The closest boring is at Dellwood Pump Station at the far eastern end of the hydraulic reach taken from Eustis Engineering LLC Job No. 13965. Dellwood Pump Station is at the corner of the eastern end of the West Slidell levee alignment and the northern end of the Slidell floodwall segment by the railroad tracks. This boring information was used for development of the soil parameters for settlement of the levee and for pile capacities for structures within the first hydraulic reach. The stability analysis for the levee section used generalized values assumed from soils in southeast Louisiana.

The parts of the alignment that are west of Dellwood Pump Station are in a non-developed area with limited geotechnical information. It is probable that the borings used at the Dellwood Pump Station are overestimating the strengths occurring in the undeveloped parts of the alignment where marsh deposits are more likely near the surface. An alternative section, which utilized cohesion to effective overburden pressure (c/p) values multiplied by a factor of 0.22, was used to estimate in-situ material strengths. This proved to also overestimate the strengths to estimate the need for berms, so strata was taken in 10-foot increments and starting with a typical weak value for marsh of 200 psf. Every 10 feet, 50 psf was added to the cohesion values until the strength line intersected the c/p value. From this point the c/p value was used. The parameters can be found in section 3 of Annex D3. The current berm section was created to help give a more accurate fill quantity count for estimation purposes for berms, which would likely be used in actual construction.

There are five structures in the West Slidell section of the alignment. These include the W Doucette Road Gate, West Bayou Pacquet Gate, Bayou Pacquet Gate, Bayou Liberty Gate, and Bayou Bonfouca Gate. No boring data was available at these locations. The closest boring is at Dellwood Pump Station from Eustis Engineering LLC Job No. 13965. This boring was used to establish soil properties such as unit weight, cohesion, and material type and used for structural analysis and levee settlement. The section was used to analyze levee stability.

10.23.3.2 South Slidell Section: Oak Harbor, I-10 Crossing, Slidell East and Northeast Levee

10.23.3.2.1 Oak Harbor Levee (Reach 2)

From the railroad gate connecting West Slidell with South Slidell, the alignment would transition to a floodwall running parallel along the east side of the railroad tracks. The alignment would transition to levee when it turned east toward Highway 11. The alignment would cross Highway 11 and would turn south in the vicinity of the existing Schneider Canal Pump Station and then turn east (on a portion of the existing Oak Harbor ring levee). The alignment would then turn north and then east in the vicinity of the I-10.

Boring information from Eustis Engineering LLC Job No. 10120 was used for development of geotechnical parameters for this section. The report totals 20 borings that make up the Oak Harbor ring ranging from 20 to 50 feet deep. Thirteen of those borings are along the proposed Oak Harbor alignment. This stretch of the alignment gives the greatest confidence against risk of changing in PED; however, additional explorations and testing are still required to meet design requirements.

The floodwall along the Norfolk Southern Railway Corporation railroad tracks divides the West Slidell levee section from the Oak Harbor levee section. Boring information was also taken from borings taken for Dellwood Pump Station from Eustis Engineering LLC Job No. 13965 to establish soil properties such as unit weight, cohesion, and material

type. Dellwood Pump Station is at the corner of the eastern end of the West Slidell levee alignment and the northern end of the floodwall by the Norfolk Southern Railway Corporation railroad tracks.

10.23.3.2.2 I-10 Crossing (Reach 3)

The alignment at I-10 would be raised to ramp over the new levee section. This lift will be applied to clear the 2082 elevation with settlement to avoid future work for maintenance at the I-10 crossing.

Borings from Eustis Engineering LLC Job No. 10120 was used for development of geotechnical parameters for this section. There are two borings taken directly below I-10.

10.23.3.2.3 Slidell East and Northeast (Reach 4)

The alignment would continue southeast and would tie to an existing portion of the Lakeshore Estates ring levee. The alignment then would turn north and then east and cross Old Spanish Trail/Highway 433. The alignment would continue north and tie to a portion of the existing King's Point levees and would turn north. Immediately south of Highway 190 Business the alignment would turn from levee to floodwall to provide risk reduction to the existing Hardin Road power substation. The alignment would wind northerly before tying into high ground in the vicinity of I-10.

There was no geotechnical information from King's Point west levee. Eustis Engineering LLC Job No. 10802 was developed to address seepage around a bend of the northern end of the Lakeshore Estates levee (approximately 100 feet south of the RP alignment), but very limited strength data was within, and none at a greater depth than 15 feet. Eustis Engineering LLC Job No. 10120 has geotechnical information at the western end of this reach (at I-10) and approximately 1,700 feet from the Lakeshore Estate levee on its western side. The geotechnical information goes down to depths of 50 feet. No other geotechnical information was provided this far east. Eustis Engineering LLC Job No. 10120 was used for parameter development and analyses for the Slidell east and northeast levees.

Three structures are within the Slidell east and northeast sections of the alignment. These are the northeast tie-in walls, the Lakeshore Estates Northeast Gate, and Kingspoint W-14 canal gate. The closest boring to the Northeast tie-in wall is at Slidell Memorial Hospital from Eustis Engineering LLC Job No. 13418 which is approximately 2.5 miles away from the project location. Borings taken from Oak Harbor Pump Station from Eustis Engineering LLC Job No. 10463 were used for the Lakeshore Estates Northeast gate and Kingspoint W-14 canal gate. The boring is 1.5 miles from Lakeshore Estates Northeast gate and 2.1 miles from Kingspoint W-14 canal gate. Soil properties such as unit weight, cohesion, and material type were taken from these borings.

10.23.4 METHODOLOGY AND ASSUMPTIONS

The HSDRRS design criteria was used as a reference to direct design criteria. During PED, a full geotechnical exploration plan should be developed with investigations taken at every 500 feet off center with borings alternating between 5-inch continuous Shelby tube borings (undisturbed) and 3-inch Shelby tube borings (general type) or CPT. These explorations should include comprehensive testing and full development of site-specific parameters. The project areas across the alignment would be further refined, which would divide into numerous reaches based on subsurface profiles, topography, and design elevations.

Stability and seepage analysis follows HSDRRS guidelines for factor of safety requirements. Levees were analyzed for 2032 and 2082 project grade elevation with some overbuild to account for settlement. Water levels were checked for the overbuild elevation (1-2 feet above project grade) and the project grade elevation, as well as low water condition in the evaluation of the Q-case (i.e., undrained). S-case (i.e. drained) analysis were also performed for low water cases for floodside and land side analyses. Q case analyses use unit weight and cohesion for stability parameters, whereas S-case use unit weight and friction angle for stability parameters. In the case of the levee sections without berms, the factor of safety for S-case low water was the same for only a centerline vertical was assumed for analysis with no variation at the toes. Block Searches and Circular were done for Spencer's analysis.

Janbu was used as a design check in lieu of Method of Planes. The HSDRRS guidelines for factor of safety requirements for Method of Planes was followed for Janbu analyses.

Elevations for far off timelines like 2082 would typically not be checked for stability but was performed in this case to give an idea for future ROW considerations. Table D:10-32 below summarizes these HSDRRS criteria.

Table D:10-32. Deep-Seated Global Stability FOS for Levee Embankment Design

COMPONENT	ANALYSIS TYPE		HSDRRS (Janbu instead of MOP), MINIMUM FOS
Levee Embankment	Deep-Seated Global Stability (Spencer's Method)	Design Hurricane (SWL)	1.50
		Water at Project Grade	1.40
		Low Water (hurricane condition)	1.40
		Low Water (non-hurricane condition) S- case	1.40
		Design Hurricane (SWL) w/ dry PS borrow pit	N/A
		Water at Construction Grade	1.20
		Water at Interim Construction Grade (levees designed with staged construction)	1.50
	Deep-Seated Global Stability (Design Check with Janbu's Method)	Design Hurricane (SWL)	1.30
		Water at Project Grade	1.20
		Low Water (hurricane condition)	1.30
		Low Water (non-hurricane condition) S- case	1.30
		Design Hurricane (SWL) w/ dry PS borrow pit	N/A
		Water at Interim Construction Grade	1.30

A simple levee section was analyzed for stability and settlement. A 10-foot crown with 1V:3H slopes were analyzed to check stability for a minimum levee section. Stability was checked at the highest 2032 design elevation along a given levee alignment. With more site-specific data, and a full HSDRRS analysis, the levee as designed here may no longer be feasible and stability berms, reinforcement geotextile, or deep-soil mixing may be necessary to meet HSDRRS criteria.

For settlement, a 10-foot crown with 1V:3H slopes were analyzed to give the loading for a simple levee section. Settlement was checked on an overbuild section that was 2.5

feet above the 2032 design elevation and plotted on a 50-year gradient line with 2032 to 2082 design elevations.

For estimating structural design features, HSDRRS criteria was followed to create pile capacity curves for 18-inch pipe piles. The parameters include the lateral earth pressure coefficient, k, and the bearing capacity factors, Nc and Nq. A summary of the design values that Geotechnical Branch (MVN-ED-F) assumed for these parameters for clay and sand material for non-displacement-type piles is included in Table D:10-33. Lane's weighted creep analyses were performed assuming water loads to the top of the wall for 2082 design elevations. These were used to help inform on sheet pile lengths for quantity estimates.

Table D:10-33. HSDRRS properties for Pile capacity for the Recommended Plan

MATERIAL	UNDRAINED CONDITIONS				DRAINED CONDITIONS			
	K_c	K_t	N_c	N_q	K_c	K_t	N_c	N_q
CH	1	0.7	9	1	1	0.7	0	10
ML	1	0.35	12.9	4.5	1	0.35	0	19.5
SM	1.25	0.5	0	22.5	1.25	0.5	0	22.5

10.23.5 BORING AND DESIGN INFORMATION

Test information taken from the borings found in the reports mentioned in Section 10.23.2. were used to create parameters for stability analysis. This test information can be found in Section 2 of Annex D3. Seven different strength lines were developed for the various features (floodgates/levees) throughout the alignment. These strength lines were plotted based on data acquired from UCT and one-point unconsolidated undrained overburden tests taken from the borings. Borings are 3-inch as they were not taken for USACE Mississippi Valley Division, MVN specific projects. For a typical USACE-MVN hurricane risk reduction project, 5-inch undisturbed borings would be taken and three-point unconsolidated undrained (Q) triaxial compression tests would be performed. Atterberg limit tests were also performed from these selected boring samples. The location of these borings can be found in Table D:10-34. Note that Eustis Engineering LLC reports had one set of coordinates per job, so jobs that have more than one boring have the same coordinates. The boring information that was available can be found in Section 2 of Annex D3.

Table D:10-34. Borings Used for the Recommended Plan

Eustis Job No. 13965 Used for West Doucette, Slidell Railroad, Bayou Bonfouca Floodgate, Bayou Liberty Floodgate, Bayou Pacquet Floodgate, West Bayou Pacquet				
Boring No.	Latitude	Longitude	Ground EL	Depth
B-1 (EE 13965)	30° 15’ 15.73” N	89° 47’ 18.18” W	NA	80’
***Eustis* Job No.13418 Used For Northeast Tie-in**				
B-1 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
B-2 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
B-3 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	40’
A-1 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	10’
A-2 (EE 13418)	30° 17’ 0.97” N	89° 46’ 6.10” W	NA	10’
Eustis Job No. 10120 Used for South Slidell Levee				
B-1 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-2 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-3 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-4 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-5 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-6 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-7 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-8 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-9 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-10 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-11 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-12 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-13 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-14 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-15 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	50’
B-16 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-17 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-18 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-19 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.26” W	NA	20’
B-20 (EE 10120)	30° 14’ 17.50” N	89° 46’ 53.**26**” W	NA	50’

***Eustis* Job No. 10463 Used for W-14 Floodgate, Lakeshore Estates Northeast Gate, Kingspoint**				
Boring No.	Latitude	Longitude	Ground EL	Depth
B-1 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	1.6	50’
B-2 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	2.15	100’
B-3 (EE 10463)	30° 14’ 29.80” N	89° 46’ 38.66” W	5	50’

No settlement test data was available, so correlations which use water contents from the strength and compression tests and the liquid limits from the Atterberg tests that taken in some areas.

The initial void ratio (or e0) was calculated by multiplying the water content results from the strength tests by the specific gravity and assuming the soil is saturated. The specific gravity was assumed to be 2.7, so the equation is as seen below.

E0 = wGs/S or 2.7w/100

The compression index (or Cc) was calculated as a function of the in-situ water content. All clays in analyses were classified as fat or lean clays (CH and CL) which is typical for southeast Louisiana. The Cc correlation data points were based on a Plasticity Index correlation found in “Correlation of Compression Index and Soil Properties of New Orleans Area Clays,” Table 16, dated 04 September 2011. The equation is shown below.

Cc = 0.017w – 0.299

Recompression index (or Cr) values were chosen as 20 percent of the Compression Index (or Cr) as recommended in the “Correlation of Compression Index and Soil Properties of New Orleans Area Clays” report. All soil was assumed as normally consolidated for this study, so the overconsolidation ratio (or OCR) is assumed as 1. This is a conservative assumption as OCR values are typically overconsolidated near the surface resulting in lower settlement estimates. Unit weights in the settlement analysis are taken from the same values used in the stability parameters.

The Cv correlation data points were based on a water content correlation which can be found in NAVFAC DM-7. 1, and is included as Figure D:10-43. Since the correlation is only provided in graphical form in the NAVFAC DM, an equation was estimated for the CV correlation using Microsoft Excel. The image was first imported into Microsoft Excel and points were manually added to match the CV correlation line. Next, a trend line was added to these points and the power function option was selected. The resulting equation may be found below. In areas where no liquid limit values are available, values between 10 and 50, which is in the Settle/3D recommended value range and among the high average of tested values from the available Atterberg test information. CV and CVr values were set to be equal to each other in the analysis.

$$C_V = 9809.9LL^{-2.847}$$

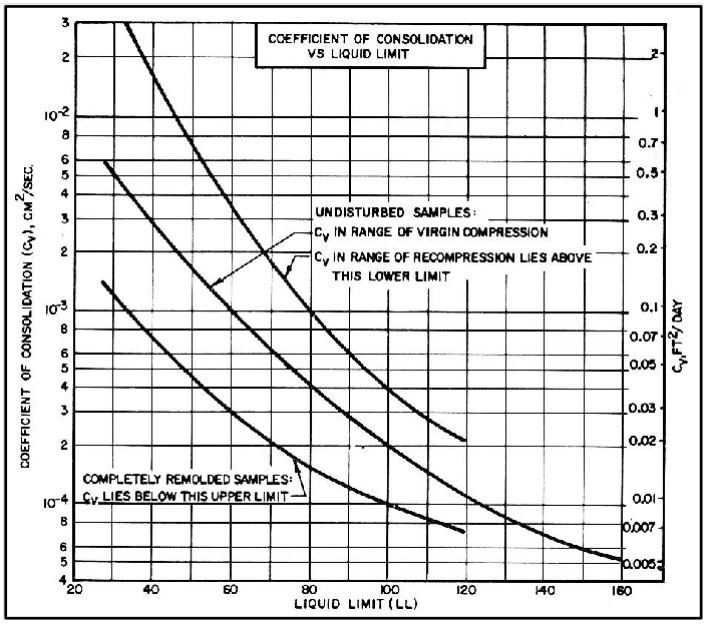


Figure D:10-43. Approximate Correlations for Consolidation Characteristics of Silts and Clays

Note: Figure D:10-43 is Figure 4 from the NAVFAC DM, 7.1-144.

Design parameters, correlation plots, and strength plots can be found in Section 3 of Annex D3.

10.23.6 LEVEE STABILITY ANALYSIS

The stability of the earthen levees was analyzed in SLOPE/W version 10.0.0.17401 from the Geostudio Suite of programs. The methodology used in cross sections was Spencer's Method with design checks done with Janbu's method analyses. Spencer's stability was performed on overbuild sections to account for settlement. Entry-exit and block searches with tension cracks applied to the driving side were checked for water at the top of the levee, at project grade water level, and low water at the ground surface. Failure surfaces were then optimized. Janbu checks were performed with block searches done with no tension cracks or optimization. S-cases were performed in both Spencer's and Janbu analyses on each section for low water analysis. All S-case runs are not optimized and have no tension cracks.

Table D:10-35. 2032 Spencer's Factor of Safety for the Recommended Plan

Project Feature	2032 Design EL	2032 Overbuild EL	FS at top of levee	FS at project grade	FS at low water	FS S-case Land Side	FS S-case flood side
West Slidell	+13.5	+14.5	1.38	1.56	1.41	2.49	2.21
Oak Harbor	+14	+15.5	2.24	2.27	2.28	1.48	1.48
I-10 crossing	+15	+16.5	2.13	2.14	2.15	1.40	1.40
Slidell East & NE	+16	+17.5	2.01	2.03	2.03	1.41	1.41

Table D:10-36. 2032 Janbu Factor of Safety for the Recommended Plan

Project Feature	2032 Design EL	2032 Overbuild EL	FS at top levee	FS at project grade	FS at low water	FS S-case Land Side	FS S-case flood side
West Slidell	+13.5	+14.5	1.31	1.35	1.43	2.32	2.50
Oak Harbor	+14	+15.5	1.95	2.00	2.07	1.32	1.32
I-10 crossing	+15	+16.5	1.84	1.89	1.95	1.31	1.31
Slidell East & NE	+16	+17.5	1.74	1.79	1.85	1.30	1.30

The stability results for these levee sections can be found in Table D:10-37 below. Checks were performed for 2082 levee sections. This was performed to get an idea if levee sections without design berms would continue to fit in the proposed ROW at the end of the 50-year life cycle of the levee. The West Slidell levee section transitions into high ground just south of S. Tranquility Road where the ground surface is higher than the 2032 design elevation of +13.5 ft. A small levee section (without berm) was checked for a slight overbuild 2082 elevation of +19.0 ft. The 2082 stability results for Spencer's and Janbu's methods can be found in tables D:10-43 and D:10-44 respectively.

Table D:10-37. Results of 2082 Spencer's Factor of Safety for the Recommended Plan

Project Feature	2082 Design EL	2082 Overbuil d EL	FS at top of levee	FS at project grade	FS at low water	FS S-case Land Side	FS S-case flood side
West Slidell	+17.5	+19	1.43	1.42	1.42	1.41	1.41
Oak Harbor	+17.5	+19	1.86	1.88	1.88	1.99	1.99
I-10 crossing	+18.5	+21.5	2.07	2.09	2.08	1.40	1.40
Slidell East & NE	+20	+21.5	1.67	1.68	1.68	1.41	1.41

Table D:10-38. Results of 2082 Janbu Factor of Safety for the Recommended Plan

Project Feature	2082 Design EL	2082 Overbuild EL	FS at top of levee	FS at project grade	FS at low water	FS S-case Land Side	FS S-case flood side
West Slidell	+17.5	+19	1.49	1.55	1.71	1.31	1.31
Oak Harbor	+17.5	+19	1.61	1.66	1.71	1.98	1.98
I-10 crossing	+18.5	+21.5	1.84	1.93	1.95	1.31	1.31
Slidell East & NE	+20	+21.5	1.43	1.47	1.53	1.30	1.30

The West Slidell levee section starts at the Norfolk Southern Railway Corporation railroad west of U.S. Highway 11 in the vicinity of Dellwood Pump Station and tracks westward until tying into high ground on the south side of U.S. Highway 190 and South Tranquility Road. This entire section is in undeveloped marsh area and consists of floodside and land side stability berms.

These bermed areas required slightly higher shear strengths for stability. These strength gains were within reasonable expectations for strength gain after decades of consolidating pressure. A reinforcement geotextile was also used to keep the berm sizes within the established ROW of the 2032 levee section. For this section, stability was checked through, around, and in front of the geotextile. To meet 2082 design elevations, a 110 ft geotextile with 13,200 lbft/ft in strength. The 2082 stability results for the West Slidell berm section for Spencer's and Janbu can be found in Tables D:10-39 and D:10-40 respectively.

Table D:10-39. Results of 2082 West Slidell Berm Section Spencer's Factor of Safety for the Recommended Plan

Analysis	Around	Through	In Front
Q-case High Water (top)	1.34	1.38	1.97
Q-case Project Grade	1.40	1.42	1.96
Q-case Low Water Flood Side	1.40	1.43	2.59
Q-case Low Water Land Side	1.66	1.43	1.97
S-case Flood Side	3.64	3.28	2.31
S-case Land Side	4.63	3.32	2.06

Table D:10-40. Results of 2082 West Slidell Berm Section Janbu Factor of Safety for the Recommended Plan

Analysis	Around	Through	In Front
Q-case High Water (top)	1.40	1.32	2.02
Q-case Project Grade	1.49	1.37	2.02
Q-case Low Water Flood Side	1.63	1.44	2.69
Q-case Low Water Land Side	2.10	1.46	2.02
S-case Flood Side	3.66	3.06	1.41
S-case Land Side	4.70	2.97	1.44

10.23.7 SETTLEMENT ANALYSIS

Settle3D Version 4.013 by Rocscience Inc., was used for the settlement analysis of the levee sections. Embankment sections with a 10-foot crown with 1V:3H slopes were used to model the loading on the soil. No settlement test data was available, so correlations were used from the test data. The lift schedules were created with a family of settlement curves based on CEMVN's experience with soft soils in southeast Louisiana. Settlement was calculated at the levee centerline for overbuilt levee sections to give time for the levee section to settle before needing to be lifted to stay above future hydraulic design grades. Lift schedules were created to serve as a guide for fill estimates for the 50-year life of the project. The settlement output can be found in

Section 5 of Annex D3. The family of curves and lift schedule can be found in Section 6 of Annex D3.

10.23.7.1 West Slidell Section: Lacombe, Bayou Bonfouca, Slidell West Levee

The preliminary design elevation is +13.5 ft for the levee. A levee section of elevation +14.5 ft (1 ft above the 2032 design elevation) with an assumed ground surface of +2.0 ft (taken from terrain raster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The levee section has a 99 ft berm on the land side and an 84 ft berm on the flood side. The first lift is estimated to elevation +16.0 ft in 2033 followed by a lift to elevation +17.5 ft in 2038, elevation +19.0 ft in 2051, and elevation +19.0 ft in 2076. The schedule was estimated for the West Slidell section with the berms (Reach 1) and applied to entire West Slidell levee section, including the tie-in section without berms (Reach 1A). Lifts for the tie-in section would not include the berms and ground surface is higher. As a result, lifts would not use as much fill, but the schedule was kept the same to stay conservative for fill quantity estimates given the lack of site specific geotechnical subsurface information.

10.23.7.2 South Slidell Section: Oak Harbor Levee

The preliminary design elevation is +14.0 ft for the levee. A general levee section of 15.5 ft (1.5 ft above the 2032 design elevation) with an assumed ground surface of 0 ft (taken from terrain raster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The first lift is estimated to elevation +17.0 ft in 2035 followed by a lift to elevation +18.0 ft in 2048, and elevation +19.0 ft in 2064. The RP alignment tracks along the southern edge of the existing Oak Harbor ring levee. The settlement analysis takes into account the entire levee section including the existing ring levee. The existing levee has likely prestressed the soil, so settlement estimates are likely conservative.

Estimates remain conservative for fill quantity estimates.

10.23.7.3 South Slidell Section: I-10 Crossing

A theoretical levee cross section with 1 on 3 slopes and a 10-foot crown was assumed for stability analysis. A 2,200 ft wider and flatter section that would sit over the existing Interstate 10 section was assumed for settlement. The wider section allows for a better approximation of fill that would be placed over the existing I-10 Highway and build it high enough that after settling the road elevation would remain over the 2082 elevation. Thus, design elevation used in analysis was +21.5 ft with an assumed ground surface at the I-10 Highway of +10.0 ft (taken from terrain raster dataset). The levee and the I-10 would be lifted during first construction in year 2032 to construction elevation of +21.5 ft with no future lifts planned to avoid future disruptions to the traffic on the interstate.

10.23.7.4 South Slidell Section: Slidell East and Northeast Levee

The preliminary design elevation for the Slidell East and Northeast levee section is +16.0 ft for the levee. A general levee section of +17.5 ft (1.5 ft above the 2032 design elevation) with an assumed ground surface of 0 (taken from terrain raster dataset), 1 on 3 slopes and a 10-foot crown were used as a cross section in the initial construction in 2032. The first lift is estimated to elevation +19.0 ft in 2034 followed by a lift to elevation +20.5 ft in 2047, and elevation +21.5 ft in 2064. The alignment of the RP runs along sections of the Lakeshore Estates Ring Levee and King's Point west levee. The settlement analysis considers the entire levee section including the existing ring levee. The existing levee has likely

prestressed the soil, so settlement estimates are likely conservative. Estimates remain conservative for fill quantity estimates.

10.23.8 SEEPAGE

Seepage results can be found in section 7 of Annex D3. Refer to Section 5 Seepage of the Final Array for general considerations on seepage.

10.23.9 STRUCTURAL ELEMENTS

Several structural elements are necessary throughout the alignment where levees are not practical. Floodwalls are implemented in areas where ROW is limited and gates are implemented where access is necessary. Design elevations for the structures is the 2082 hydraulic design grade for a 100-year return period storm. The 2082 elevations for structures are slightly lower than adjacent levee sections. The levee and wall/gate elevations were designed for an allowable overtopping rate of 0.1 cfs/ft (as was done in the HSDRRS design) using overtopping equations from the 2018 EurOtop manual. The equations for computing overtopping rates are slightly different for levees vs. floodwalls since the equations for a levee include an input for slope of the levee (and were designed with wave runup in mind) whereas for a floodwall there is no slope since floodwalls are vertical.

Only pile capacities and seepage analyses were performed to inform on quantities for 18- inch pipe piles and sheet piles necessary for all structures throughout the RP alignment.

Pile capacity curves for 18-inch diameter open ended steel pipe piles were developed. The 2010 version of the Pile Capacity software developed by Daniel Haggerty, P.E. of USACE MVN-EDG, was used. Ultimate (i.e. Factor of Safety equal to 1.0) tension and compression capacities for the undrained and drained conditions were developed. Unbalanced loads were not calculated as part of the study and as such, pile capacity curves are taken from the ground surface. Due to the nature of the pile (hollow pipe), the development of an inner soil plug was considered for compressive capacities. The theoretical frictional capacity of the soil plug along the inner surface of the pile was compared to the theoretical bearing capacity of the end area of the plug. The lesser of the two was taken as the end bearing of the plug and was added to the bearing capacity

of the pile (steel area only) and the frictional capacity of the outer shaft surface of the pile to obtain the total compressive capacity of the pile. End bearing capacity for cohesive soils for undrained conditions with cohesive values less than 1,000 psf were not considered. Overburden stress was limited to 3,500 psf and did not consider pile batter in determination of skin friction capacity. The pile capacity curves do not consider group effects. Densities were derived from the material tests from the individual Eustis Engineering LLC reports. Other parameters such as lateral earth pressure coefficient, k, and the bearing capacity factors, Nc and Nq were taken from HSDRRS design guidelines.

10.23.9.1 West Slidell Section: W Doucette Road Gate, West Bayou Pacquet Gate, Bayou Pacquet Gate, Bayou Liberty Gate, and Bayou Bonfouca Gate

The preliminary design elevation for all West Slidell structures including the W Doucette Road Gate, West Bayou Pacquet Gate, Bayou Pacquet Gate, Bayou Liberty Gate, and Bayou Bonfouca Gate is +16.0 ft to satisfy the 2082 required hydraulic design grade. All of these structural features used the boring information at Dellwood Pump Station from Eustis Engineering Job No. 13965 for soil properties such as unit weight, cohesion, and material type. Pile Capacity analyses and Lane's weighted Creep analyses were performed to determine sheet pile lengths and pile lengths for quantity estimates. Sheet pile length was recommended to be 30 ft.

10.23.9.2 Oak Harbor: Slidell Railroad

The floodwall along the West Slidell railroad divides the West Slidell levee section from the Oak Harbor levee section. Boring information was also taken from Dellwood Pump Station from Eustis Engineering LLC Job No. 13965 to establish soil properties such as unit weight, cohesion, and material type. The preliminary design elevation for the floodwall at the railroad is +16.5 ft. Pile capacity analyses and Lane's Weighted Creep Analysis were performed to help inform on pile lengths for quantity estimates. Sheet Pile length was recommended to be 30 ft.

10.23.9.3 South Slidell: Lakeshore Estates Northeast Gate, Kingspoint W-14 Canal Gate

The Lakeshore Estates Northeast Gate and Kingspoint W-14 canal gate both have preliminary design elevations of +18.5 ft to satisfy the 2082 required hydraulic design grade. Soil properties such as unit weight, cohesion, and material type were taken from the boring at Oak Harbor Pump Station from Eustis Engineering LLC Job No. 10463 and used for analyses. Pile capacity analyses and Lane's Weighted Creep Analysis were performed to help inform on pile lengths for quantity estimates. Sheet Pile length was recommended to be 30 ft.

10.23.9.4 South Slidell Section: Northeast Tie-in Wall

The preliminary design elevation for the Northeast Tie-in is +18.5 ft to satisfy the 2082 required hydraulic design grade. Soil properties such as unit weight, cohesion, and material type were taken from the boring at Slidell Memorial Hospital from Eustis Engineering LLC Job No. 13418 and used for analyses. Pile capacity analyses and Lane's Weighted Creep Analysis were performed to help inform on pile lengths for quantity estimates. Sheet pile length was recommended to be 40 ft.

Pile capacities and Lane's weighted Creep results can be found in sections 9 and 10 respectively in Annex D3.

10.23.10 CONCLUSIONS AND RECOMMENDATIONS

Geotechnical analyses were performed on levee features and structural features on the RP alignment. Geotechnical data available was limited. For many features, the closest geotechnical investigations were used rather than site specific borings. Also, the number of borings available for a given feature were limited. This resulted in using borings not on the alignment to help inform decisions on the RP. As a consequence, analyses made many assumptions to complete preliminary checks. These assumptions were primarily made to aid in quantity estimates, but a full geotechnical exploration program should be developed during PED and implemented with associated testing to better inform on soil conditions along the alignment. This should be completed before any design sections are analyzed during PED.

SECTION 11
References and Resources

11.1 PROJECT REFERENCES:

NA

11.2 WEBSITES:

Note for all imagery in GIS maps in this appendix:

Source: ESRI, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community.

Full Citation: World Imagery provides one meter or better satellite and aerial imagery in many parts of the world and lower resolution satellite imagery worldwide. The map includes 15m TerraColor imagery at small and mid-scales (~1:591M down to ~1:288k) for the world. The map features Maxar imagery at 0.3m resolution for select metropolitan areas around the world, 0.5m resolution across the United States and parts of Western Europe, and 1m resolution imagery across the rest of the world. In addition to commercial sources, the World Imagery map features high-resolution aerial photography contributed by the GIS User Community. This imagery ranges from 0.3m to 0.03m resolution (down to ~1:280 in select communities). For more information on this map, including the terms of use, visit http://goto.arcgisonline.com/maps/World_Imagery.

For site specific analysis, the National Agriculture Imagery Program (NAIP (2021)) was used.

11.3 SOFTWARE:

NA

SECTION 12
List of Acronyms and Abbreviations

BBMNWR	Big Branch Marsh National Wildlife Refuge
CFS	Cubic Feet Per Second
C/P	cohesion to effective overburden pressure
CPT	Cone Penetrometer Test
CSRM	Coastal Storm Risk Management
DIFR-EIS	Draft integrated feasibility report with environmental impact statement
EB	East Bound
ED	Engineering Division
EPA	**Environmental Protection Agency**
FRM	Flood Risk Management
GS	Galvanized Steel
HH&C	Hydraulics, Hydrology and Coastal Branch
HIS	Hydrographic and Impairment Statistics
HSDRRS	Hurricane and Storm Damage Risk Reduction System
KSI	kilo-pound per square **inch**
La DOTD	**Louisiana Department of Transportation & Development**
MRL	Mississippi River Levee
MVN	New Orleans District
NAVD	North American Vertical Datum
NOV-NF	New Orleans to Venice Non-Federal West
NS	Nonstructural
PCF	Pounds per Cubic feet

PDT PSF	Project Delivery Team Pounds per square foot
RCP	Reinforced Concrete Pile
ROW	Right of Way
RP	Recommended Plan
RR	Railroad
TRS	Temporary Restraining Structure
TSP	Tentatively Selected Plan
USACE	U.S. Army Corps of Engineers
VFZ	Vegetation Free Zone
WB	West Bound
WSE	Water Surface Elevation
WSLP	West Shore Lake Pontchartrain