APPENDICES

Figures

TABLES

Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ACES	Applied Coastal Engineering Software
ADT	Average Daily Traffic
ASCE	American Society of Civil Engineers
CLARA	Coastal Louisiana Risk Assessment
cfs	Cubic Feet per Second
CPRA	Louisiana Coastal Protection and Restoration Authority
CPT	Cone Penetrometer Test
EADD	Expected Annual Damage Dollars
EM	Engineering Manual
DOTD	Louisiana Department of Transportation and Development
FEMA	Federal Emergency Management Agency
ft	Foot (Feet)
gal	Gallon
gpm	Gallons per Minute
H&H	Hydraulic and Hydrologic
hr	Hour
H_S	Significant Wave Height
HSDRRS	Hurricane and Storm Damage Risk Reduction System
in	Inch(es)
lb	Pound(s)
LF	Linear Feet
LFPDG	Louisiana Flood Protection Design Guidelines
LiDAR	Light Detection and Ranging
MHW	Mean High Water
mi	Mile
MLW	Mean Low Water
mph	Miles per Hour
MTL	Mean Tide Level
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NSI	Neel-Schaffer, Inc.
O&M	Operations and Maintenance
PCC	Portland Cement Concrete
PIANC	World Association for Waterborne Transport Infrastructure
ROW	Right-of-Way
sec	Second(s)
SPM	Shore Protection Manual
SWL	Still Water Level
T_p	Wave Period
TOW	Top of Wall
TSP	Tentatively Selected Plan
TWI	The Water Institute of the Gulf

USACE	United States Army Corps of Engineers
UU	Unconsolidated Undrained
WSEL	Water Surface Elevation

EXECUTIVE SUMMARY

The Eden Isles community in Slidell, Louisiana has experienced repeated flooding from coastal storm surge and elevated water levels associated with Lake Pontchartrain. Although regional flood risk reduction systems proposed by the U.S. Army Corps of Engineers (USACE) and the State of Louisiana will provide protection to surrounding areas, the Eden Isles community remains outside the limits of those systems. As a result, the Coastal Protection and Restoration Authority (CPRA) commissioned this feasibility study to evaluate conceptual flood risk reduction alternatives for the community.

This study builds upon prior planning efforts completed for CPRA and St. Tammany Parish. The goal of the project is to provide a level of protection of +11.0 feet NAVD88 to the community while maintaining access, accommodating navigation, minimizing operational complexity, and allowing flexibility for phased implementation as funding becomes available. Existing elevations along the Oak Harbor levee to the north and Interstate 10 to the east are mostly at or above +11.0 feet, requiring only minor modifications to meet the target elevation. More substantial modifications are required along the U.S. Highway 11 corridor to the west and the lakefront area to the south to meet the target elevation. This report focuses on the lakefront as the first phase of the overall project, specifically on three primary project features: Lakeview Drive flood protection improvements, Harbor Drive flood protection improvements, and a navigable floodgate at Grand Lagoon.

This effort included collection and evaluation of extensive survey, geotechnical, roadway, utility, coastal, and hydraulic data. Numerical hydrologic and hydraulic modeling was performed to evaluate storm surge, rainfall runoff, interior drainage, overtopping resiliency, and operation of the proposed systems. This information was used to develop conceptual designs and associated costs with alternatives for each project feature.

Lakeview Drive Alternatives

Lakeview Drive forms the primary southern boundary of the Eden Isles community and presents the most significant design challenges due to limited right-of-way, adjacent homes, existing utilities, driveway access requirements, and the need to maintain traffic during construction.

Four primary alternatives were evaluated for Lakeview Drive:

Alternative A – 2-Lane Raised Roadway with Future Floodwall

Alternative A consists of a phased approach in which the existing two-lane roadway of Lakeview Drive is first raised to +8.5 feet, followed by construction of a floodwall in the roadway median to the final protection elevation of +11.0 feet. Two floodwall concepts were evaluated, including a sheetpile-supported floodwall and a reinforced concrete floodwall integrated into the roadway section.

This alternative provides meaningful interim protection while preserving the ability to phase future improvements. However, the constrained right-of-way requires directional U-turn openings and introduces significant challenges related to driveway access, utility relocations, and long-term operations.

Alternative B – 4-Lane Divided Roadway with Future Floodwall

Alternative B raises the roadway to a uniform elevation of +6.0 feet during the initial phase and constructs a taller floodwall within the roadway median during a second phase. A key feature of this alternative is that two travel lanes would be on each side of the floodwall, reducing access issues to existing homes.

While this alternative improves constructability and operational functionality compared to Alternative A, it provides less interim flood protection between phases and results in a larger structural footprint and increased overall project cost relative to Alternative A. Also, the taller wall height would obstruct lake views from ground level.

Alternative C – Raised Roadway with Passive Flood Barrier System

Alternative C utilizes self-deploying passive flood barriers, such as FloodBreak, integrated into the roadway corridor. The roadway would initially be raised to approximately +8.0 feet, followed by installation of passive flood barriers to achieve the final protection elevation.

This alternative improves aesthetics and traffic circulation because the barriers remain flush with the roadway surface during normal conditions. However, it is significantly more expensive than the other alternatives and carries substantially higher long-term operations and maintenance requirements associated with inspection, certification, and specialized system maintenance. Also, barriers can only be installed on straight road segments, meaning approximately 25% of the roadway will still require permanent floodwall.

Alternative D – Levee North of Lakeview Drive

Alternative D consists of construction of a conventional earthen levee north of Lakeview Drive. This alternative minimizes construction costs, operational complexity, and long-term maintenance requirements while avoiding many of the roadway access issues associated with floodwalls. However, implementation would require costly acquisition of all residential properties and structures north of Lakeview Drive and does not readily support phased construction.

Lakeview Drive Alt A 2-Lane Raised Roadway with Future Floodwall	**Lakeview Drive Alt B** 4-Lane Divided Roadway with Future Floodwall	**Lakeview Drive Alt C** Raised Roadway with Passive Flood Barrier System	**Lakeview Drive Alt D** Levee North of Lakeview Drive
Capital Cost ~$43M–$54M **Annual O&M Cost ~$150K–$200K** **Key Pros** • Cost-efficient relative to other alternatives • Provides interim protection • Supports phased implementation **Key Cons** • ROW constraints • Complex driveway access (U-turns required) • Utility conflicts and operational complexity	**Capital Cost ~$63M–$69M** **Annual O&M Cost ~$50K–100K** **Key Pros** • Improved access vs Alt A • Better traffic operations • Easier constructability **Key Cons** • Higher total cost • Lower interim protection (+6 ft) • Larger footprint, aesthetic concerns	**Capital Cost ~$143M** **Annual O&M Cost ~$550K–600K** **Key Pros** • Best traffic flow & aesthetics • Less permanent wall obstruction • Flexible access (barriers retract) **Key Cons** • Highest cost • Very high O&M (inspection/certification) • Specialized system reliance	**Capital Cost ~$58-59M** **Annual O&M Cost ~$125K–$250K** **Key Pros** • Cost-effective • Low operational complexity • Minimal long-term maintenance **Key Cons** • Requires property acquisition/demolition • No practical phasing • Significant community impacts

Harbor Drive Alternatives

Several alternatives were also evaluated for Harbor Drive, including:

- Raising Harbor Drive to the target protection elevation (embankment only or phased with floodwall)
- Construction of levees adjacent to or within the vacant properties south of the roadway
- Replacement of the existing bulkhead system along the lakefront

Because the Harbor Drive area is largely undeveloped, these alternatives present fewer access and utility challenges than Lakeview Drive. Earthen levee alternatives were found to be among the most cost-effective solutions but would impact the usability of adjacent lots. Use of the existing bulkhead system would require substantial rehabilitation or replacement to serve as part of a reliable flood protection system. Raising Harbor Drive to an elevation of +11.0 feet is recommended as a preferred alternative due to its moderate cost and impacts remaining mostly within the existing right-of-way.

Harbor Drive Alt A Raising Harbor Drive	**Harbor Drive Alt B** Levee	**Harbor Drive Alt C** Replace existing bulkhead
Capital Cost ~$4.0M-$8.7M	**Capital Cost ~$1.3M-$9.6M**	**Capital Cost ~$19.6M**
Annual O&M Cost ~$5K-50K	**Annual O&M Cost~$10K-15K**	**Annual O&M Cost ~$50K - $75K**
Key Pros • Option for phased implementation • Moderate cost	**Key Pros** • Lowest cost • Minimal roadway disturbance	**Key Pros** • Uses shoreline alignment • High protection elevation (+12 ft)
Key Cons • Access restrictions (wall option only) • Requires grading & some acquisition	**Key Cons** • Access grading required • Partial property impacts	**Key Cons** • Highest cost • Requires full reconstruction

Grand Lagoon Floodgate

A navigable floodgate structure at Grand Lagoon was evaluated as a critical component of the overall flood protection system. Both sector gate and barge gate concepts were considered. Based on the evaluation of cost, navigation requirements, constructability, and operational considerations, a barge gate concept is recommended, with a minimum gate opening width of 40 feet.

Hydraulic modeling demonstrated that closure of the gate during storm events will require interior drainage improvements to manage rainfall runoff within the protected area. A series of culverts through the floodgate complex is sufficient to facilitate drainage within the system. A pump station does not significantly impact water surface elevations within the system but does reduce the time required to draw the water levels back down to normal conditions.

Costs were developed for two gate sizes (40-ft and 90-ft widths) to represent the possible cost range associated with this feature.

Grand Lagoon Floodgate (40-ft)	Grand Lagoon Floodgate (90-ft)
Capital Cost **~$35.9M**	**Capital Cost** **~$43.6M**
Annual O&M Cost **~$200K–$250K**	**Annual O&M Cost** **~$250K–$300K**
Key Pros • Enables navigation • Lower cost option	**Key Pros** • Navigation Constraints
Key Cons • Greater navigation capacity • More flexible for large vessels	**Key Cons** • Higher capital cost • Higher O&M cost

Conclusions

This feasibility study demonstrates that multiple technically feasible flood risk reduction solutions exist for the Eden Isles community. Each alternative presents tradeoffs between construction cost, property impacts, access, constructability, operational complexity, aesthetics, and long-term maintenance requirements.

Planning-level construction costs were developed for all alternatives and include estimated utility relocations, property acquisition, and contingency allowances appropriate for feasibility-level design. Costs were escalated to future-year dollars to represent anticipated construction timing. The total cost range for all combined features varies from $80 to $206 million depending on the combination of features selected for implementation.

Operations and maintenance (O&M) requirements were also considered at a planning level. Expected annual O&M costs range from $255,000 to $975,000 depending on the combination of features selected for implementation.

Alternatives that raise existing roadways provide opportunities for phased implementation and incremental resilience improvements but introduce operational and access constraints within the limited roadway corridors. Conventional levee alternatives reduce operational complexity and long-term maintenance requirements but require significant impacts to property owners.

The study findings provide CPRA, St. Tammany Parish, and local stakeholders with a framework for selecting and advancing preferred alternatives into future environmental review, permitting, funding, and preliminary engineering design phases. Continued stakeholder coordination and refinement of operational strategies will be important as the project advances toward implementation.

1 INTRODUCTION

Neel Schaffer Inc. (NSI) is under contract with the Coastal Protection and Restoration Authority (CPRA) to analyze alternatives and further the conceptual design for implementing flood risk reduction measures for the Eden Isles community. This effort is a continuation of a previous analysis by NSI that culminated in the March 15, 2024 memorandum titled *Eden Isles Resilience & Flood Risk Reduction Planning Memorandum and Recommended Alternative*. This memorandum recommended a comprehensive resilience plan for the community against lower-level coastal storms consisting of elevating roads around the community and adding flood control structures at select locations. The focus of this effort is to advance a portion of the recommended alternative, namely the corridor along Lake Pontchartrain including Lakeview Drive, Grand Lagoon, and Harbor Drive.

2 PROJECT DESCRIPTION AND BACKGROUND

2.1 PROJECT LOCATION

St. Tammany Parish, located northeast of New Orleans along the north shore of Lake Pontchartrain (**Figure 1**), is one of the most populous parishes in Louisiana.

Figure 1: Project Location, Eden Isles Community, Slidell, St. Tammany Parish, LA

The Eden Isles community is a group of subdivisions in Slidell, Louisiana in the southeast corner of St. Tammany Parish. It is bound by Highway 11 on the west, U.S. Interstate 10 on the east, Lake Pontchartrain on the south, and Oak Harbor subdivision on the north as seen in **Figure 2**.

Figure 2: Project Area

Many of the residences have access to Lake Pontchartrain by way of canals and the Grand Lagoon waterway connection. With its connection to Lake Pontchartrain, the Eden Isles community is subject to tidal impacts and has experienced significant coastal flooding in the past. During tropical events, water levels increase in Lake Pontchartrain, pushing water through Grand Lagoon and into Eden Isles. Portions of the existing roadways surrounding Eden Isles are below +4.0 ft. During storm events, these roads can become inundated, creating difficulties for citizens to access their homes, schools, and businesses.

2.2 Purpose and Background

In January 2020, the United States Army Corps of Engineers (USACE) initiated the *St. Tammany Parish, Louisiana Feasibility Study* to investigate flood risk reduction and coastal storm risk management solutions to address flood damage caused by rainfall and coastal storm events in St. Tammany Parish. The study was finalized in 2024 and authorized by the Water Resources Development Act of 2024. The project includes the construction of levees, floodwalls, storm surge barriers, and improved drainage systems, with the goal of providing protection to over 26,000 structures. The proposed alignment of the project takes advantage of the existing levees within the Slidell area but excludes the Eden Isles community from federal protection.

In 2023, CPRA released the 4th edition of *Louisiana's Comprehensive Master Plan for a Sustainable Coast* (Master Plan) which includes a Slidell Ring Levee project (Project ID 001.HP.13). The alignment for this project also utilizes the existing levees with the Slidell area, coincides with the USACE TSP alignment, and excludes Eden Isles. The Master Plan states that structural flood risk reduction measures will be implemented to supplement the State and Federal projects to reduce flood risk in the Eden Isles community.

NSI was previously contracted by St. Tammany Parish Government (STPG) for the PO-167 St. Tammany Parish Coastal Study. This study developed a plan for implementing flood risk reduction measures within coastal St. Tammany Parish including for the Eden Isles, Kingspoint (East Slidell), and West Slidell communities. The goal was to increase flood risk reduction opportunities for residents and increase resiliency following natural disasters impacting coastal St. Tammany Parish. This study also recommended protection for the Eden Isles community, providing a Parish-level recommendation consistent with the State's Master Plan.

Following those efforts, NSI was contracted by CPRA and completed the *Eden Isles Resilience & Flood Risk Reduction Planning Memorandum and Recommended Alternative* in March 2024. The memorandum recommended a comprehensive flood risk reduction alternative for Eden Isles consisting of elevating U.S. Highway 11, Lakeview Drive, and Harbor Drive to interim elevations of +8.5 ft NAVD88 and providing adequate space to later add a floodwall to increase the protected elevation to +11.0 ft NAVD88. The alternative also included a flood gate and pump station at Grand Lagoon and one-way flow control structures at two locations along Interstate 10. These improvements, along with the existing, elevated features of Interstate 10 and the Oak Harbor Levee, would form a comprehensive flood risk reduction system around the community. The memorandum recommended a phased approach allowing for implementation of the various project features as funding becomes available.

2.3 Project Benefits

When all resiliency features are fully implemented around Eden Isle, they will provide a significant risk reduction from coastal storms. The Water Institute of the Gulf (TWI) calculated the reduction in Expected Annual Damage Dollars (EADD), a measure of annual damages that can be expected from storm surge flooding events, resulting from implementation of the proposed projects. With the USACE TSP levee system and the recommended resiliency measures around the Eden Isles community in place, the analysis estimated a reduction in EADD of approximately $120 million.

2.4 Project Attributes and Features

This report summarizes the feasibility and alternatives analysis of the proposed project features along Lakeview Drive, Grand Lagoon, and Harbor Drive, as shown in **Figure 3**. Proposed improvements to Highway 11 and the one-way flow control structure under I-10 will be further evaluated in future efforts.

Project attributes and alternatives considered for this project consist of the following:

- Level of protection of +11.0 ft.
  - Elevation of Lakeview Drive and/or installation of a floodwall structure along Lakeview Drive or construction of a levee north of Lakeview Drive
  - Elevation of Harbor Drive and/or installation of a floodwall structure along Harbor Drive
  - Installation of a navigable floodgate structure at Grand Lagoon
  - Features to facilitate interior drainage of the community when the proposed floodgate at Grand Lagoon is closed

- A drainage structure at the Harbor Drive / Interstate 10 tie-in as necessary to provide local drainage

Project features that allowed phased construction at interim elevations were prioritized to provide greater flexibility and phased funding opportunities.

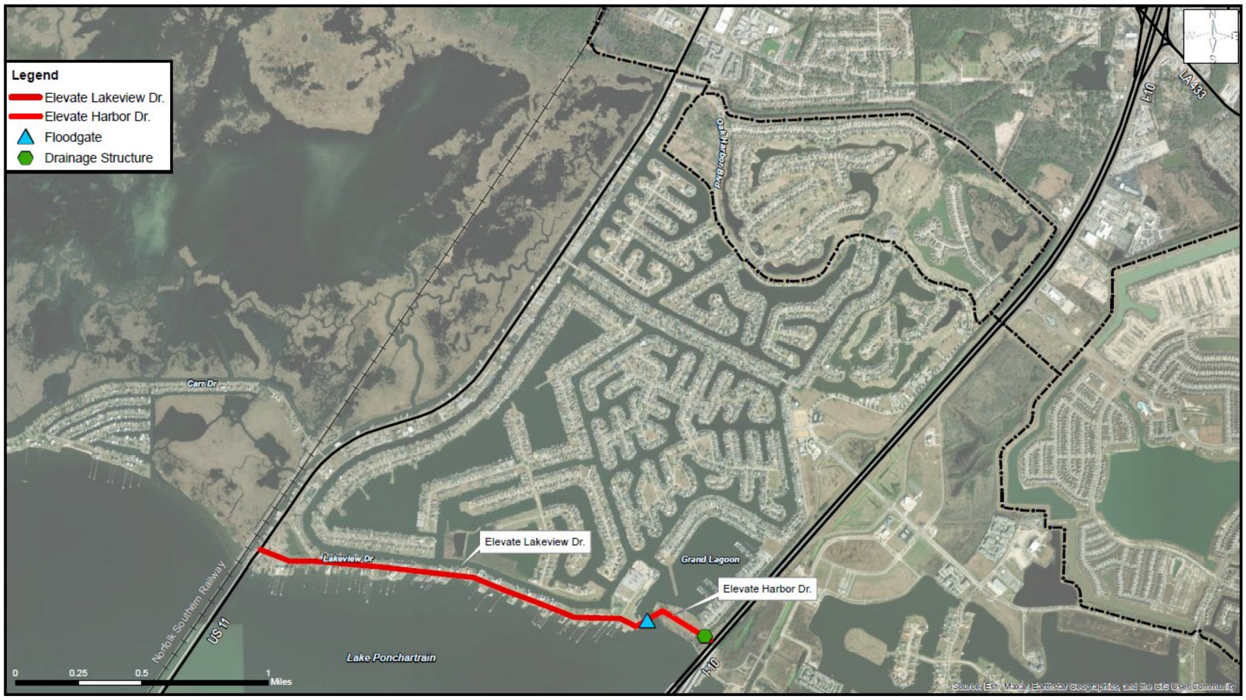


Figure 3: Overview of Proposed Project Features

2.5 Design Criteria

A Design Criteria Document (**Appendix A**) was developed to establish the technical basis for the project. The document defines the design standards, assumptions, and parameters that guided development of the project alternatives and will continue to govern subsequent design phases. It outlines the governing codes, references, and performance requirements applicable to each major project feature, ensuring that all analyses and designs are performed using consistent and defensible criteria.

3 Existing Conditions and Data Collection

3.1 Existing Conditions

Existing features surrounding the community currently provide some resilience against extreme high tides and minor tropical events. To the north, the existing Oak Harbor levee is at elevation +12.5 ft and Interstate 10 elevations on the east side range from +10.7 ft to +12.7 ft. Along the lakefront, Lakeview Drive and Harbor Drive range from +3.3 ft to +6.9 ft, with typical elevations from +5.0 ft to +6.5 ft and elevations along Highway 11 range from +4.5 ft to +14.6 ft, with typical elevations in the +5 ft to +8 ft range.

3.2 Data Collection Summary

An extensive data collection effort was undertaken for this feasibility phase to supplement previous planning phase data collection. This included survey, geotechnical, and hydrology and hydraulics.

Environmental data was not collected during this phase but will be initiated as the project moves into design.

3.2.1 Survey

Survey data was collected by All South Consulting Engineers between October 2024 and February 2025. Topographic, utility, and right-of-way data were collected along Lakeview Drive and Harbor Drive, as shown in **Figure 4.** This data supported design of embankment, roadway, and structural project features. Bathymetric, magnetometer, and side scan data were collected in Grand Lagoon to support design of the floodgate structure. Additionally, data was collected at locations within the interior canals, roads, bridges, and drainage features to support hydrologic and hydraulic (H&H) modeling. Survey data is included as **Appendix B**.

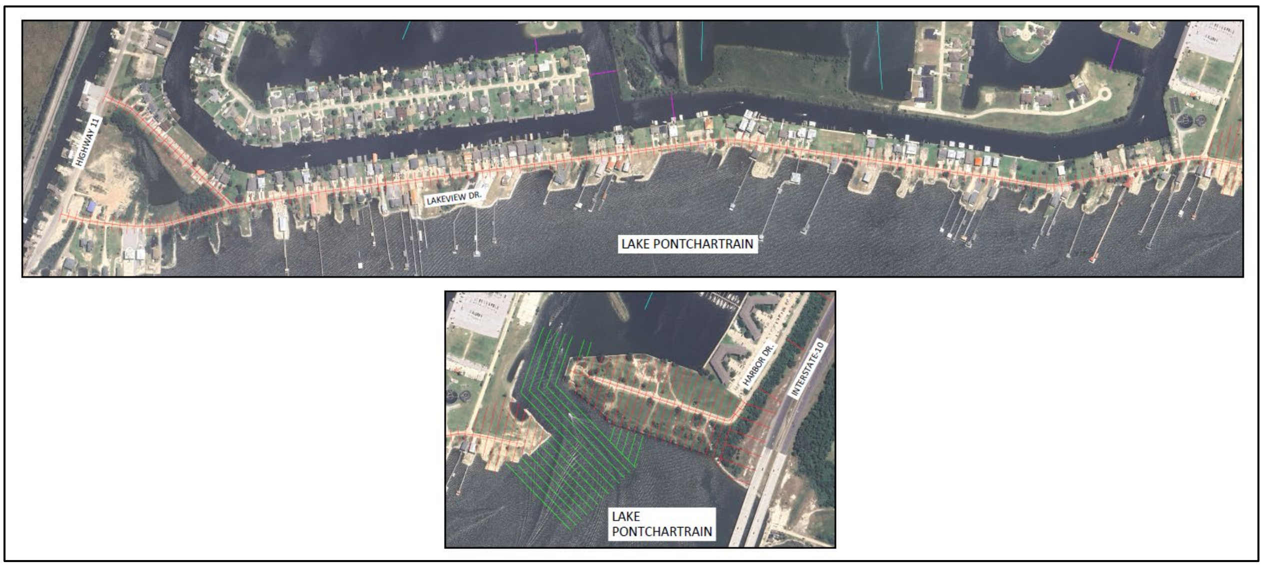


Figure 4: Survey Data collected along Lakeview Drive, Grand Lagoon, and Harbor Drive

3.2.2 Geotechnical

Geotechnical field investigation was conducted by GeoEngineers between October 9 and December 8, 2024. Two 40-ft deep soil borings and six 6-ft deep pavement borings were collected along Lakeview Drive. Two borings were collected within Grand Lagoon, one to a depth of 75-ft and the second to a depth of 100-ft. Along Harbor Drive, two 6-ft deep pavement borings were collected and one 45-ft deep soil boring at the proposed location of the drainage structure at the I-10 tie-in. Boring locations are shown in **Figure 5**, along with borings and Cone Penetration Tests (CPTs) collected by Eustis Engineering as part of the previous effort.

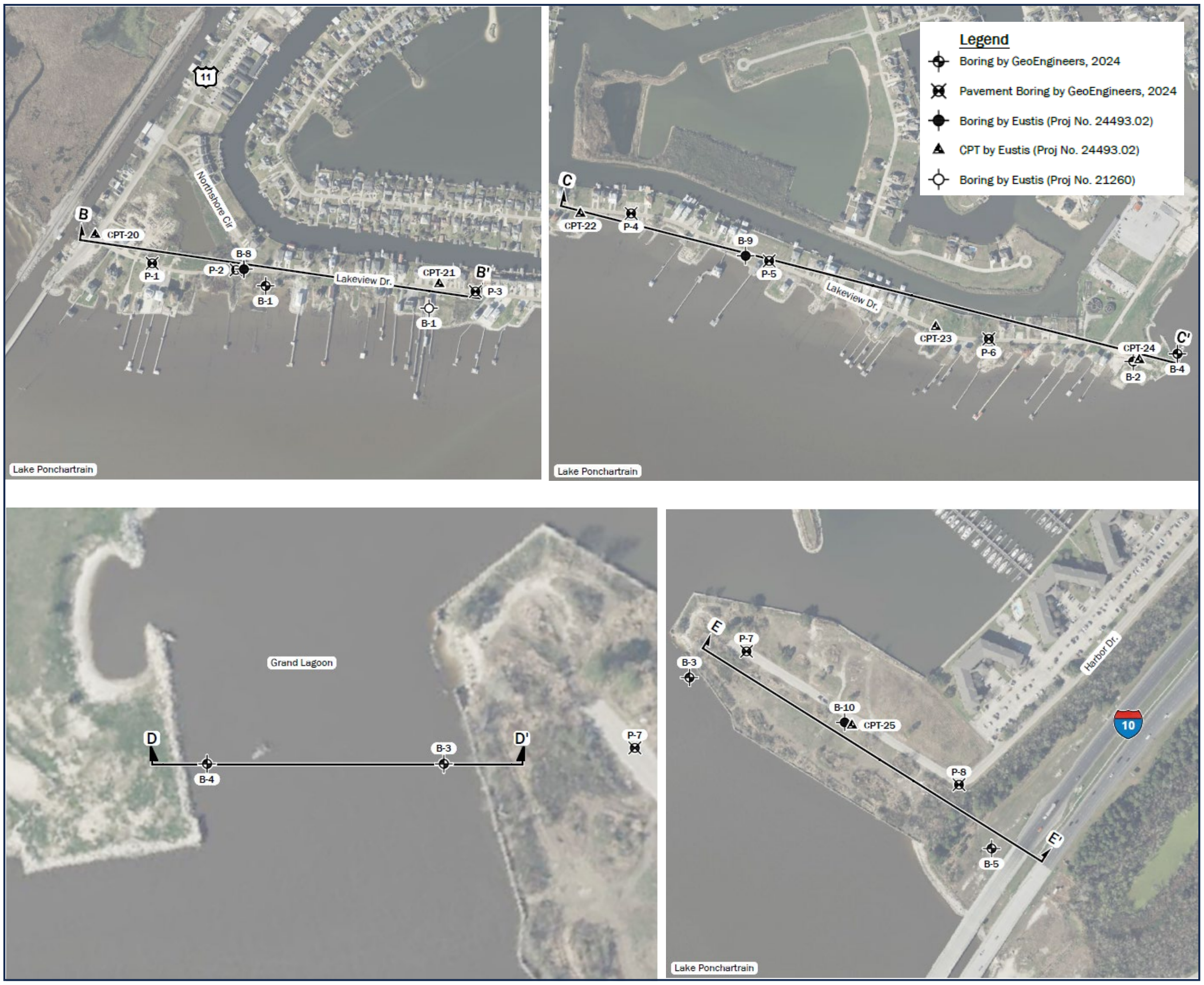


Figure 5: Borings and CPTs in Project Area

Laboratory testing included moisture content, unit weight, unconsolidated undrained (UU) triaxial compression, percent passing the U.S. No. 200 sieve, grain size distribution, one-dimensional consolidation, specific gravity, and Atterberg limits.

Lakeview Drive was constructed on 6 to 13 feet of fill, over interspersed layers of sand and clay, likely placed during construction of the community. The two borings (Borings B-3 and B-4) drilled in Grand Lagoon were quite different from one another. The western boring consists of clay over sand while the boring on the east side consists of sand over clay. Along Harbor Drive, subgrade material consists of varied fill materials including clayey silt, sandy clay, and sand. Soil design profiles were developed for each region of the project.

Because of the variability in the Grand Lagoon borings, pile capacity calculations were completed to determine if additional field exploration was warranted. Three different pile types (H-piles, pipe piles, and concrete piles) were analyzed. All three pile types indicated that Boring B-3 has less capacity than B-4. After discussion with CPRA, it was decided that Boring B-3 will be used as a more conservative design case and no additional field investigation is warranted at this time.

Geotechnical information is included in **Appendix C**.

3.2.3 COASTAL DATA SUMMARY

Available coastal data was compiled for the area from various sources including the National Oceanic and Atmospheric Administration (NOAA), Federal Emergency Management Agency (FEMA), USACE, and CPRA. The following sections provide a summary of tidal datums, precipitation, wind, and waves. A comprehensive report of all coastal data is included in **Appendix D**.

3.2.3.1 TIDAL DATUMS

The local tidal datum was determined from the nearest NOAA station with a long-term record, New Canal Station 8761927, on the south shore of Lake Pontchartrain near New Orleans (**Figure 6**).

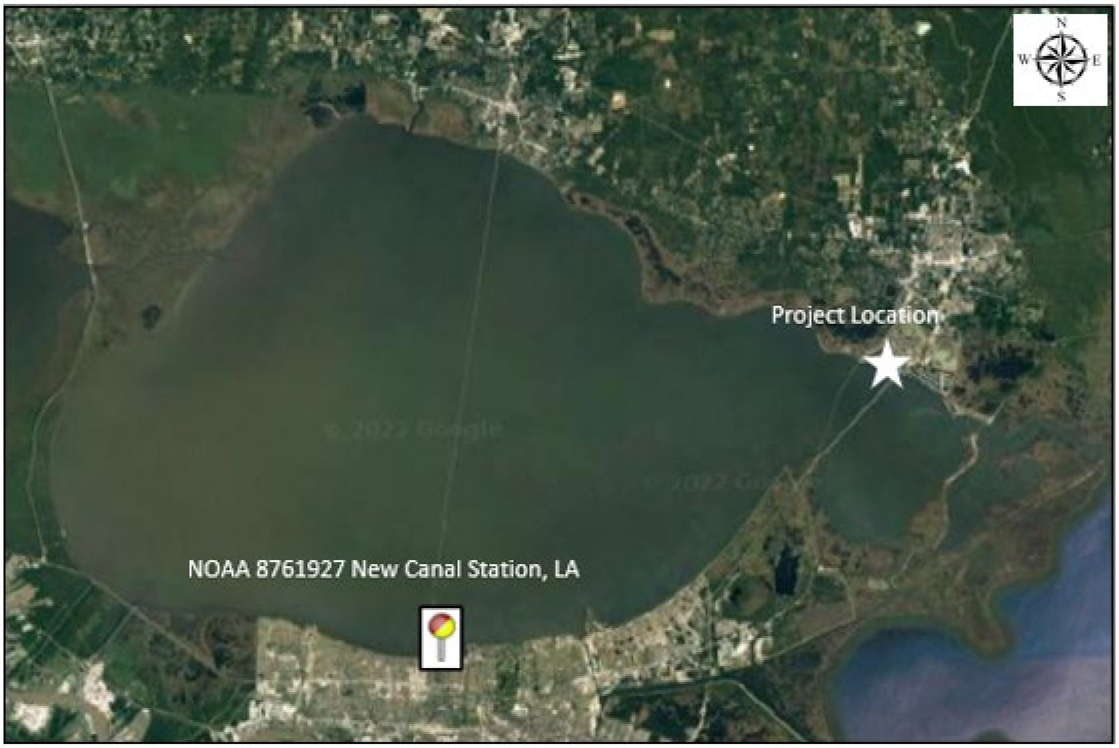


Figure 6: Location of NOAA New Canal Station, LA

Values from 2019 to 2024 were used to calculate the modified 5-year tidal datum as recommended by NOAA for areas with increased subsidence rates like Louisiana. The data is shown below in **Table 1**:

Table 1: Tidal Datum, NOAA New Canal Station, LA

Datum	2019-2024 Data [ft, NAVD88]
Mean High Water (MHW)	0.97
Mean Tide Level (MTL)	0.67
Mean Low Water (MLW)	0.36

3.2.3.2 PRECIPITATION

Rainfall data was obtained by extracting data for the Eden Isles area from the NOAA Atlas 14 precipitation frequency data server for a 24-hour storm. The point precipitation frequency estimates for 10-, 25-, 50-, and 100-year average recurrence intervals are presented in **Table 2**. The rainfall hydrographs utilized were prepared based on Type III, 24-hour Natural Resources Conservation Service (NRCS) rainfall distributions.

Table 2. Point Precipitation frequency estimates

Duration	Average Recurrence Interval (years)			
	10	25	50	100
24-hr	8.54	10.8	12.7	14.8

3.2.3.3 WIND

An analysis of available wind data in the vicinity was conducted. Wind data was collected from NOAA National Data Buoy Center New Canal Station ID 8761927 (**Figure 6**). A statistical analysis of the wind climate was conducted utilizing the extracted wind data. The data was grouped into multiple wind speeds and direction intervals in an annual wind rose plot as shown in **Figure 7**.

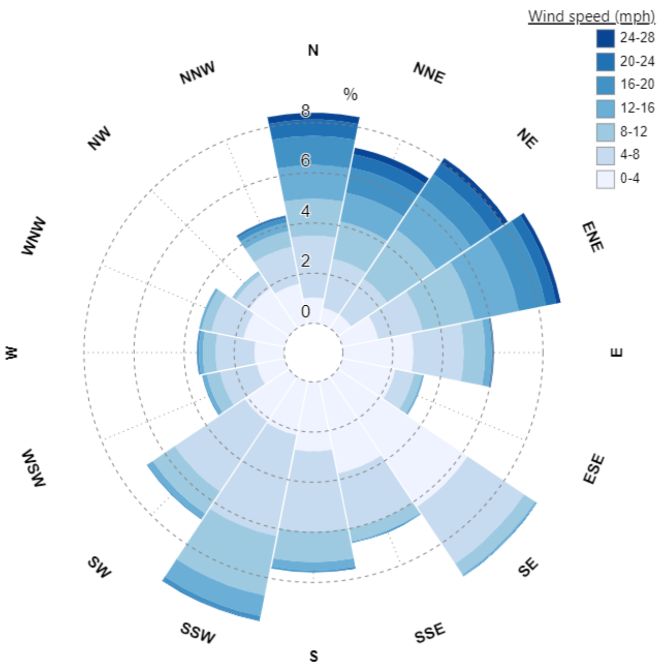


Figure 7: NOAA New Canal Station Wind Data

To further understand extreme events that may occur, a statistical analysis of the extreme wind climate was conducted using the extracted wind data. For this analysis, the annual maximum wind speed was determined, and extreme value distribution (Gumbel distribution) was subsequently utilized to estimate events of varying return periods. The extreme wind gust speeds calculated for the respective return intervals correspond well with values produced at this location by the American Society of Civil Engineers *ASCE 7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures* online Hazard Tool. The results for both methods and the ASCE values are summarized in **Table 3**

Table 3. Extreme Wind Speeds

Return Period	Calculated Wind Speed (mph)	Calculated Wind Gust Speed [mph]	ASCE Wind Speed (mph)
10	59	80	77
25	66	92	90
50	71	100	101
100	76	109	112

3.2.3.4 WAVES

The Applied Coastal Engineering Software (ACES) Program was used to calculate wave heights and wave periods for fully developed waves in Lake Pontchartrain. The program uses nonlinear equations for estimating wave conditions. Extreme wind speeds were input into the program, along with the fetch lengths for direct approaching waves and waves approaching from the maximum possible fetch length, as shown in **Figure 8**.

Figure 8 Fetch Distances Across Lake Pontchartrain

The following wave characteristics were calculated for each scenario:

Table 4. Maximum Wave Characteristics

Recurrence Interval	Wind Speed (mph)	Significant Wave Height (ft)	Wave Period (s)
10-year	59	5.0	5.5
25-year	66	5.4	5.8
50-year	71	5.7	6.0
100-year	76	6.0	6.1

Table 5. Direct Approaching Wave Characteristics

Recurrence Interval	Wind Speed (mph)	Significant Wave Height (ft)	Wave Period (s)
10-year	59	3.2	3.8
25-year	66	3.5	4.0
50-year	71	3.7	4.1
100-year	76	3.8	4.2

These waves represent the range of wave conditions that may occur in the vicinity of the project site for various events and provide a framework for design decisions.

As part of TWI's EADD analysis, as described in Section 2.3, a coupled ADCIRC+SWAN model was developed to simulate storm surge and wave heights with a suite of 90 synthetic storms generated by the USACE. Model geometry was developed from modeling conducted for the 2023 Coastal Master Plan, with revisions to represent the future Slidell Ring Levee project and the fully implemented Eden Isles project features. The statistical risk associated with the individual storms was estimated using the Coastal Louisiana Risk Assessment (CLARA) model. This model aggregates the peak flood depths associated with a suite of simulated synthetic tropical cyclones to estimate flood depth annual exceedance probability (The Water Institute, 2024). Following this analysis, peak surge and wave height was extracted for three locations, one along the south boundary, just offshore of Lakeview Drive, one along the west boundary near Highway 11, and one along the east boundary near Interstate 10. This data was used to develop curves for Annual Exceedance Probability vs Peak Surge Elevation and Peak Significant Wave Height, as shown in **Figure 9.**

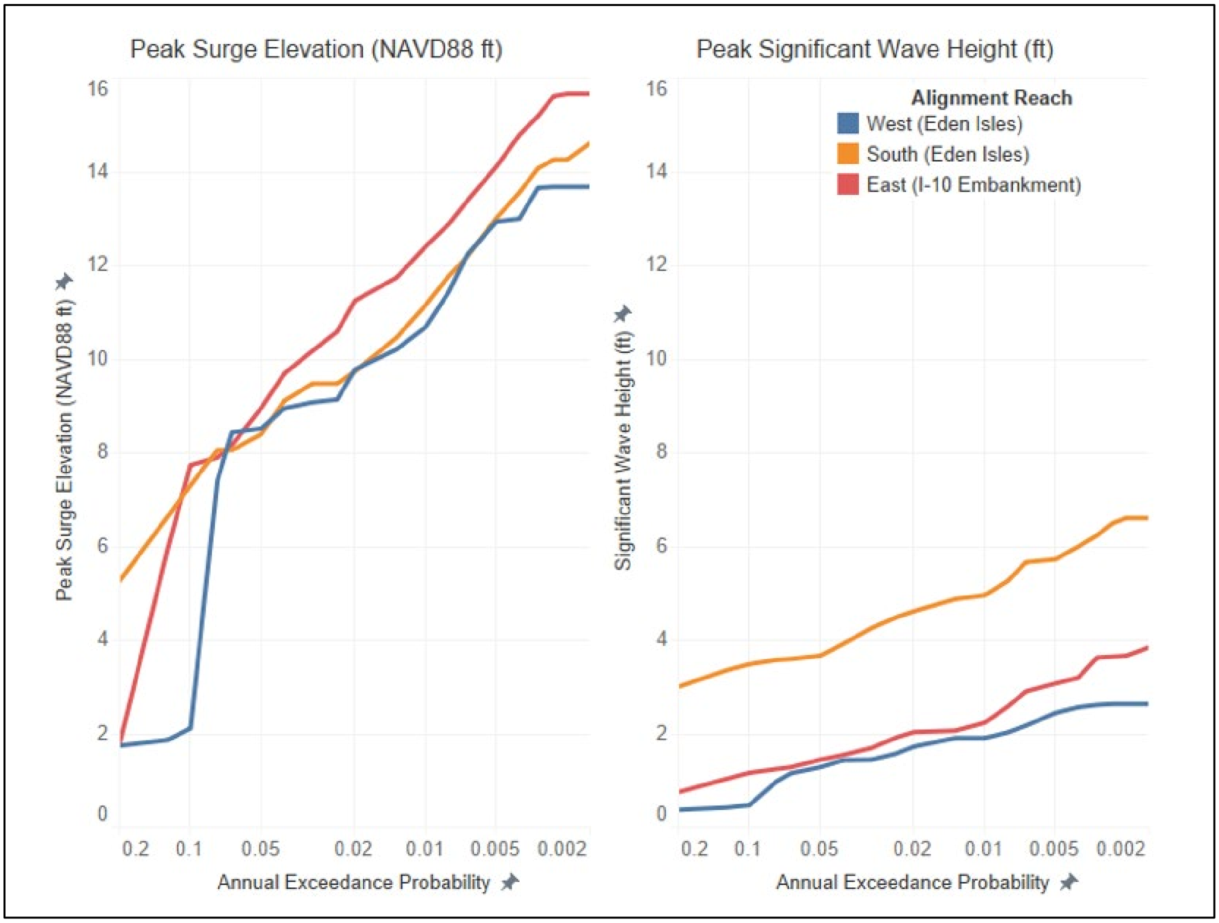
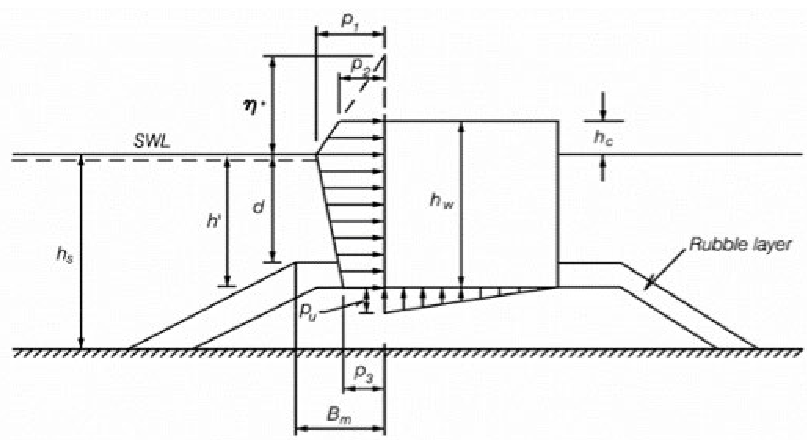


Figure 9: Annual Exceedance Probability vs Peak Surge and Wave Height (TWI, 2024)

The extraction location for the south boundary was closest to the location of the proposed project features for this effort and therefore was used in development of project design features and design criteria. Per USACE Hurricane and Storm Damage Risk Reduction System (HSDRRS) guidance, Top of Wall (TOW) elevation can be determined from the Still Water Level (SWL) plus ½ significant wave height (H_s). Since the TOW elevation is +11.0 ft, SWL and H_s can be back calculated from **Figure 9** as approximately +9.0 ft and 4.0 ft, respectively. This corresponds closely to a 0.04 Annual Exceedance Probability Event. Note that these values do not directly correlate to the wind and wave characteristics in **Table 4** and **Table 5**. Those values are generated based on linear wave theory from the recurrence interval of wind only, while **Figure 9** uses a statistical analysis of the storms input into the model, which has various storm surge elevations, wind directions, and wave heights.

3.2.3.4.1 Wave Loads

Wave forces on vertical walls were used as initial inputs to assist with determining structural flood protection features. The wave forces were computed using Goda's formulation in the Coastal Engineering Manual (USACE, 2011) with the assumption of a direct, breaking wave impacting the structure as the worst-case scenario (**Figure 10**). Loading calculations for the proposed permanent floodwall alternatives and floodgate were completed and are included in **Appendix E.**



$$\eta^* = 0.75(1 + cos\beta)\,\lambda_1 H_{design} \qquad \text{(VI−5−147)}$$

$$p_1 = 0.5(1 + cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design} \qquad \text{(VI−5−148)}$$

$$p_2 = \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right) p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \leq h_c \end{cases} \qquad \text{(VI−5−149)}$$

$$p_3 = \alpha_3\, p_1 \qquad \text{(VI−5−150)}$$

$$p_u = 0.5(1 + cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design} \qquad \text{(VI−5−151)}$$

Figure 10: Goda Wave Loading (USACE, 2011)

3.3 DESIGN ELEVATIONS

As detailed in the March 15, 2024 *Eden Isle Resilience & Flood Risk Reduction Planning Memorandum and Recommended Alternative,* a variety of sources were evaluated to determine historic and projected storm surge elevations in and around Eden Isles. These values were evaluated with input and guidance from CPRA, and the values in **Table 6** were adopted as design targets for varying levels of protection.

Table 6: Storm Surge / Design Elevations for Eden Isles

Level of Protection	Design Elevation (NAVD 88)
High	+18.0 ft
Medium	+11.0 ft
Low	+8.5 ft

Through correspondence between CPRA and the St. Tammany Levee, Drainage, and Conservation District (STLDCD), CPRA has committed to provide a “medium” level of protection of +11.0 ft along Lakeview Drive, Grand Lagoon, and Harbor Drive (Scalco, 2025).

4 Project Alternatives Development

Several alternatives were developed and analyzed for each major project feature to identify the most effective and cost-efficient design solutions. The following subsections discuss the development and details of each alternative. A drawing set for each can be found in **Appendix F**.

4.1 Lakeview Drive

Lakeview Drive forms most of the southern boundary of the Eden Isles community within the project area. The roadway extends approximately 8,000 linear feet (LF) from U.S. Highway 11 on the west to its terminus near Grand Lagoon on the east. It is classified as an urban, local two-lane roadway with a posted speed limit of 25 miles per hour (mph). Numerous residences are located along and in close proximity to the roadway. The existing right-of-way (ROW) varies in width between 60 and 66 feet, with most utilities located along the north side of the road within the ROW.

Several alternatives were considered for resiliency features along this alignment. Design constraints included available ROW width, maintaining access to existing homes, placement of turnarounds, and desire to phase construction. Additionally, given uncertainty in future Operations and Maintenance (O&M) budgets and personnel, project features that minimized pre-storm labor and operational requirements were prioritized. Four alternatives were developed:

- Alternative A: +8.5 ft Elevation of Lakeview Drive, +11.0 ft Elevation Floodwall
- Alternative B: +6.0 ft Elevation of Lakeview Drive, +11.0 ft Elevation Floodwall
- Alternative C: +8.0 ft Elevation of Lakeview Drive, +11.0 ft Elevation Self-Deploying Flood Barriers
- Alternative D: +11.0 ft Levee North of Lakeview Drive

Several variations of these alternatives were considered based on alignment, construction methodology, and cost. Floodwall alignment options considered the roadway centerline and the north and south shoulders. Placement along either shoulder was deemed infeasible due to required driveway access and right-of-way constraints. Consequently, all floodwall alignment alternatives were analyzed with placement along the roadway centerline.

To maintain access to existing driveways, directional U-turn openings will be required for alternatives within the ROW. Where directional U-turn openings are proposed along Lakeview Drive, gaps in the floodwall will require floodgates that can be closed prior to a storm event. Gate types evaluated included roller gates and passive automatic barrier systems.

All alternatives will involve utility relocations including water meters, power poles, gas lines, sewer manholes, sewer lift stations, and stormwater drainage structures. Other impacts within the ROW include mailboxes, driveways, and landscaping.

The following sections detail the development and evaluation of each alternative.

4.1.1 Lakeview Drive Alternative A: +8.5 ft Elevation of Road, +11.0 ft Elevation Floodwall

Alternative A consists of a two-phased approach to achieve the desired level of protection along Lakeview Drive. In the first phase, the existing roadway would be elevated to an interim elevation of +8.5 ft using earth fill and base course material, providing initial resiliency benefits. In the second phase, a floodwall would be constructed to the final elevation of +11.0 ft.

To make this location feasible in a phased manner, adequate space was required within the roadway median to accommodate future floodwall construction. Using the existing ROW and roadway design criteria, approximately six feet is available for a median. Floodwall designs were selected to fit within this area and to minimize disturbance of the roadway section during future construction.

Two options were considered for Alternative A:

1. Option 1 - Cantilever sheetpile with concrete jersey barrier style cap floodwall
2. Option 2 – Portland Cement Concrete (PCC) Pavement floodwall with tie-in to PCC travel lanes

4.1.1.1 Lakeview Drive Alternative A, Option 1 – Cantilever Sheetpile Floodwall

A cantilever sheetpile foundation option with a concrete jersey-barrier-style cap is shown in **Figure 11**. This option consists of an initial phase of embankment and roadway construction to +8.5 ft with a 6-ft wide grass median. The second phase would include sheetpile installation in the median with a connection to a concrete jersey barrier style floodwall reaching the final elevation of +11.0 ft. Stability and overturning analyses were performed in the CWALSHT program to develop a sheet pile section and tip elevation. A PZ-27 section was selected with a tip elevation of -6.5 ft. The sheet pile type and tip elevation were selected to resist anticipated horizontal loads transferred from the floodwall and to satisfy stability and overturning criteria. CWALSHT stability runs are included in **Appendix G**.

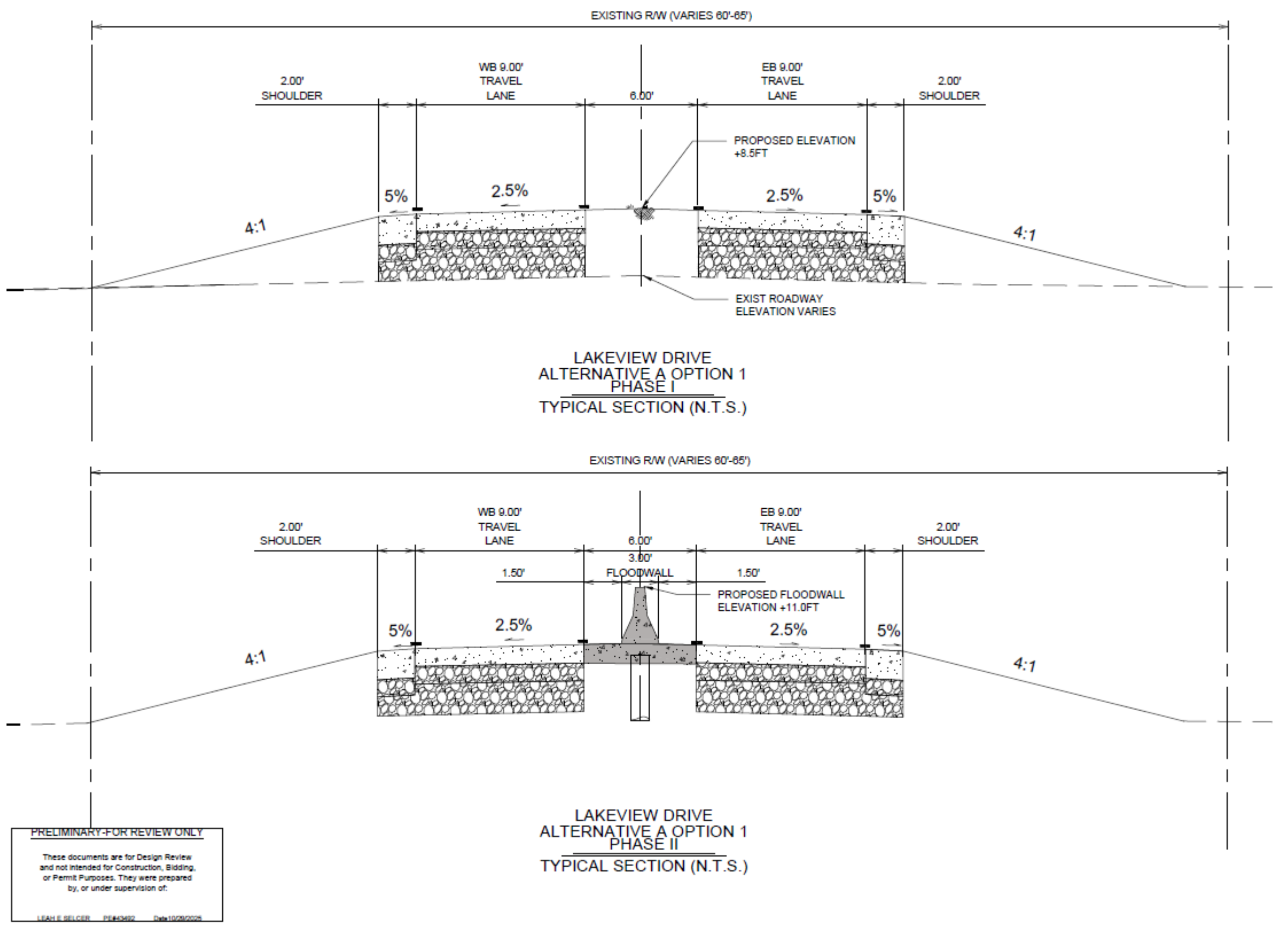


Figure 11: Alternative A, Option 1 - +8.5 ft Embankment, +11.0 ft Sheetpile Foundation Floodwall

Costs for this alternative are shown in **Table 7.** Note that costs are full capital costs and include construction, contingency, land acquisition, and utility costs. Costs have also been escalated to a future year (2029), to represent the time at which construction could begin. Detailed cost information is available in **Appendix H**.

Table 7: Alternative A, Option 1 Capital Costs

Phase	Work Items	Cost
Phase 1	Embankment & Road	$ 23,900,000
Phase 2	Sheetpile Floodwall	$ 30,500,000
	TOTAL:	**$ 54,400,000**

4.1.1.2 LAKEVIEW DRIVE ALTERNATIVE A, OPTION 2 – PCC FLOODWALL

A second floodwall option was developed that consisted of a poured PCC roadway section with toe walls on the outer edges of the pavement and a 6-ft wide grass median as part of the first phase of embankment and roadway construction. For the second phase, a concrete floodwall would be formed and poured in the roadway median. The reinforcing steel of the floodwall would be tied in to the existing PCC roadway slab, resulting in a continuous floodwall and roadway as shown in **Figure 12.** Stability and overturning analyses were performed to develop a conceptual cross section, and costs are included in **Table 8**. Overturning analyses are included in **Appendix G**. Detailed cost information is available in **Appendix H**.

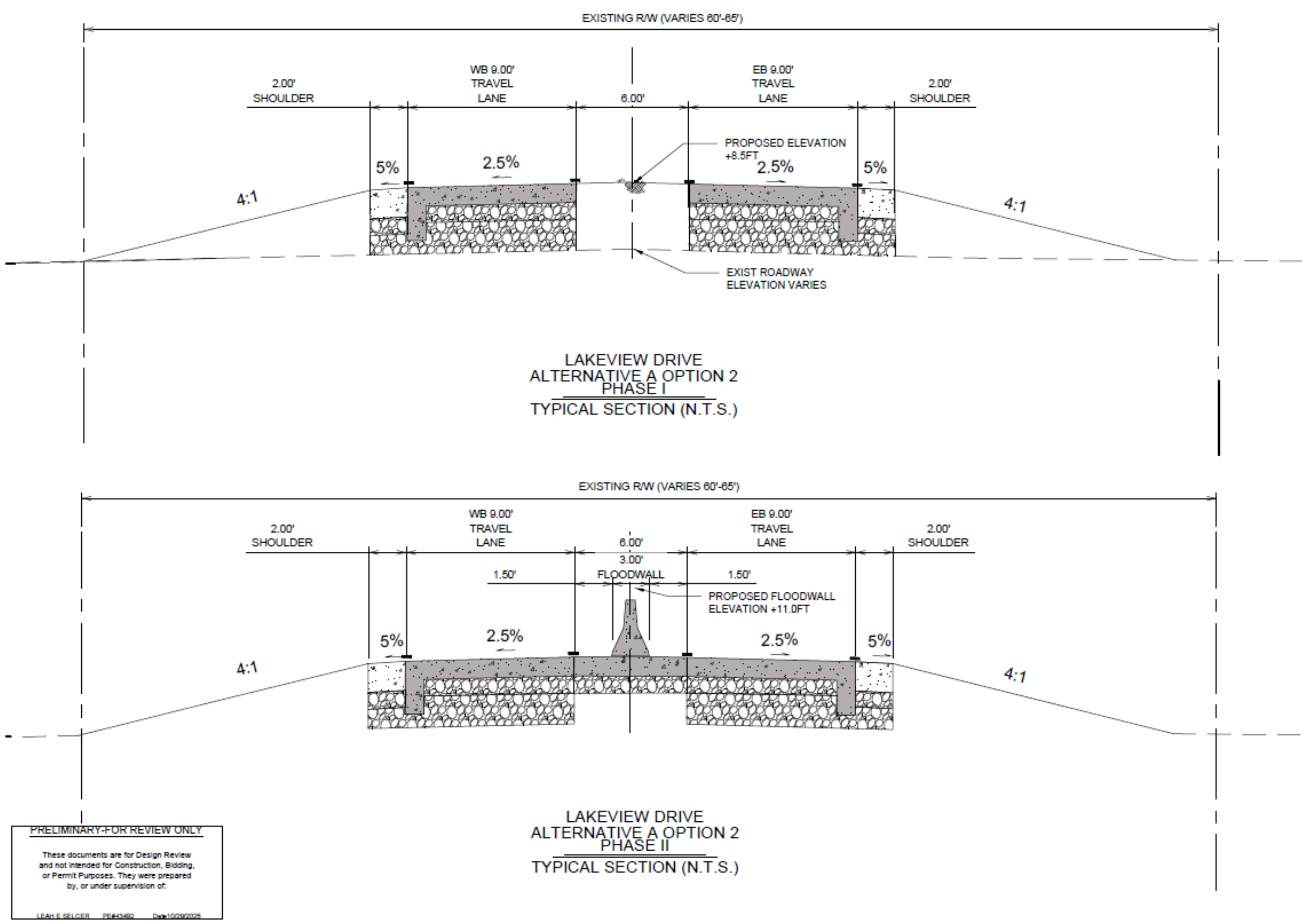


Figure 12: Alternative A, Option 2 - +8.5 ft Embankment, +11.0 ft PCC Floodwall

Table 8: Alternative A, Option 2 Capital Costs

Phase	Work Items	Cost
Phase 1	Embankment & Road	$ 23,100,000
Phase 2	PCC Floodwall	$ 20,200,000
	TOTAL:	**$ 43,300,000**

Key construction considerations for the sheetpile and PCC options include maintaining residential access during the initial embankment lift to +8.5 ft and accommodating construction equipment and vehicle access during floodwall installation. As a result, the embankment will be constructed using base course aggregate, allowing temporary bypass of traffic and access to individual driveways at the conclusion of each workday during construction.

4.1.1.3 Lakeview Drive Alternative A - Design Considerations

Design considerations common to both options for Alternative A include: limited ROW, utilities, driveway access, and turnaround requirements.

Right of Way Width and Road Design Standards

The challenge for this alternative is maximizing the embankment elevation while allowing adequate width at the top of embankment for both travel lanes, road shoulders, and the floodwall with its required offsets. Because embankment is less expensive than concrete, maximizing embankment features and minimizing concrete floodwall features is the most cost-efficient design solution. Increasing embankment height increases the horizontal length of side slopes within the ROW. This reduces the available width at the top of the embankment for roadway and floodwall features. To achieve the highest possible embankment elevation, the minimum roadway design parameters allowable by Louisiana Department of Transportation and Development (DOTD) standards for the roadway classification were selected, as shown in **Table 9**.

Table 9: Roadway Design Parameters

Roadway Design Parameters	
Lane Width	9 ft
Shoulder Width	2 ft
Cross Slope of Travel Lane	2.5%
Cross Slope of Shoulder	5.0%
Longitudinal Grade	5.0% max
Lateral Offset	≥ 1.5 ft to fixed objects (4 ft min. without curb)
Clear Zone	7–10 ft for 4:1 foreslopes (adjust per ADT)
Embankment Slope	4H:1V

Representative embankment sections with the proposed roadway parameters are shown in **Figure 11** and **Figure 12**.

Utility Impacts

Impacts to existing utilities are a major consideration for construction. Utilities located at least partially within the ROW include water, storm sewer, sanitary sewer, gas, electric, and telephone/fiber optic lines. Placement of embankment material within the ROW will impact a number of these utilities and is anticipated to require relocation in several areas. However, along much of the alignment, the main lines for several utilities are located near the northern extent of the ROW and outside the footprint of the proposed embankment. In these areas, impacts are generally limited to service connections crossing the roadway.

Additionally, in typical roadway construction projects, privately owned utilities located within the existing right-of-way that are impacted by roadway improvements are often required to be relocated by the utility owner, unless prior property rights or agreements dictate otherwise. (See DOTD. *Title 70, Part II: Utilities*, *Chapter 5* (2018)). Accordingly, for planning-level cost estimating, utility relocation costs were assumed to be limited to impacts occurring outside the existing right-of-way but within the anticipated limits of construction.

A major distinction between Option 1 and Option 2 is the continuous sheetpile wall located along the centerline of Lakeview Drive in Option 1. This configuration impacts utility services crossing the roadway from the main lines and presents significant constructability challenges associated with numerous penetrations through the wall. Therefore, for planning-level cost estimating, it was assumed that parallel utility services would be installed on the opposite side of the roadway in lieu of multiple wall penetrations. This results in significantly higher costs for Option 1 versus Option 2. The difference between the estimated costs for these options provides a reasonable planning-level range for potential utility impacts. Because the impacts for Option 1 are driven primarily by the proposed floodwall configuration rather than conventional roadway embankment impacts, the associated relocation costs were carried as project costs rather than utility-owner costs.

Access During Construction

Key construction considerations for both the sheet pile and PCC floodwall options include maintaining residential access and accommodating construction equipment and vehicle movement throughout construction. During embankment placement, access to homes must be preserved at all times. To achieve this, base course aggregate is proposed as the preferred embankment material, allowing the surface to be driven on immediately after each workday.

Construction will proceed by placing base course to the maximum allowable lift thickness for the length of roadway that can be completed daily. Two-way traffic will be managed with a flagger operating in the opposite lane during active work. At the end of each day, the aggregate will be graded to restore ingress and egress to all driveways and to provide a smooth transition to the next section of roadway. This sequence will continue until the end of the alignment is reached, at which point the process will be repeated until the goal elevation is achieved.

After the full embankment height has been achieved, roadway paving will follow, ensuring that temporary traffic access and driveway connections are maintained at the conclusion of each workday.

Driveway Access

Another design challenge associated with increasing roadway elevation is maintaining access to existing residences. DOTD requires certain geometric conditions to be met, as shown in **Figure 13** and **Table 10** to maintain safe access for driveways. The proposed embankment slope of 4:1 (20% grade) corresponds with the "special case" driveway grade described in Table 10: Driveway Design Parameters Table 10; however, a transition between this slope and the relatively flat existing driveways is required for those cases.

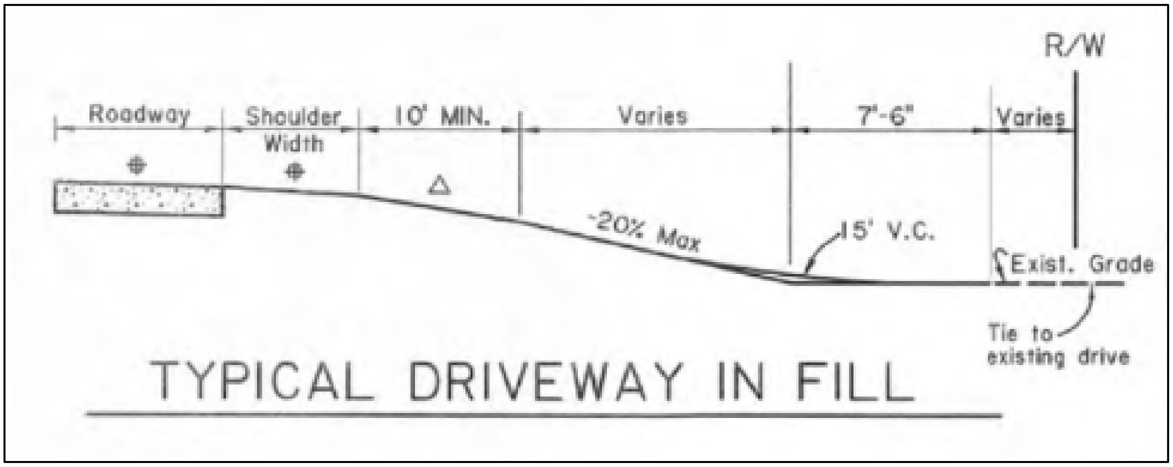


Figure 13: Driveway Access Geometric Requirements (Source: LA DOTD Standard Plan DW-01)

Table 10: Driveway Design Parameters

Driveway Design Parameters	
Driveway Grade	20% max (25% in special cases)
Driveway Break in Grade	10% crest, 9% sag, at ≥ 10-ft intervals, or 15' vertical curve

To evaluate the feasibility of these requirements, thirteen driveways were selected along Lakeview Drive with homes in close proximity to the existing roadway. The evaluated driveways can all meet the conditions; however, in many instances, the toe of the driveway embankment will extend near the existing structures and parking areas.

In addition to the issue of driveway slope, driveway access becomes more challenging once the floodwall is installed, restricting turns from the opposite travel lane. This issue is most prevalent when entering or exiting driveways while towing a trailer. To analyze this scenario, AutoTURN (vehicle swept path analysis software) was used to evaluate the feasibility of ingress and egress while towing. AASHTO's *A Policy on Geometric Design of Highways and Streets* (Green Book) was used to select a design vehicle, as shown in **Table 11**.

The most common towing vehicle anticipated is a pickup truck hauling a recreational fishing vessel. For design purposes, the AASHTO "Car with Camper Trailer (P/T)" was selected, as shown in **Figure 14,** as its overall dimensions and turning characteristics are most representative of the pickup truck with fishing vessel.

Table 11: AASHTO Minimum Turning Radii of Design Vehicles

Design Vehicle Type	Passenger Car	Single-Unit Truck	Single-Unit Truck (Three Axle)	Intercity Bus (Motor Coach)		City Transit Bus	Conventional School Bus (65 pass.)	Large[a] School Bus (84 pass.)	Articulated Bus	Intermediate Semi-trailer
Symbol	P	SU-30	SU-40	BUS-40	BUS-45	CITY-BUS	S-BUS36	S-BUS40	A-BUS	WB-40
Minimum Design Turning Radius (ft)	23.8	41.8	51.2	41.7	44.0	41.6	38.6	39.1	39.4	39.9
Center-line[b] Turning Radius (CTR) (ft)	21.0	38.0	47.4	37.8	40.2	37.8	34.9	35.4	35.5	36.0
Minimum Inside Radius (ft)	14.4	28.4	36.4	24.3	24.7	24.5	23.8	25.3	21.3	19.3
Design Vehicle Type	**Interstate Semitrailer**		**"Double Bottom" Combination**	**Rocky Mtn Double**	**Triple Semi-trailer/ Trailers**	**Turnpike Double Semi-trailer/ Trailer**	**Motor Home**	**Car and Camper Trailer**	**Car and Boat Trailer**	**Motor Home and Boat Trailer**
Symbol	WB-62*	WB-67**	WB-67D	WB-92D	WB-100T	WB-109D*	MH	P/T	P/B	MH/B
Minimum Design Turning Radius (ft)	44.8	44.8	44.8	82.0	44.8	59.9	39.7	32.9	23.8	49.8
Cen-terline[b] Turning Radius (CTR) (ft)	41.0	41.0	40.9	78.0	40.9	55.9	36.0	30.0	21.0	46.0
Minimum Inside Radius (ft)	7.4	1.9	19.1	55.6	9.7	13.8	26.0	18.3	8.0	35.0

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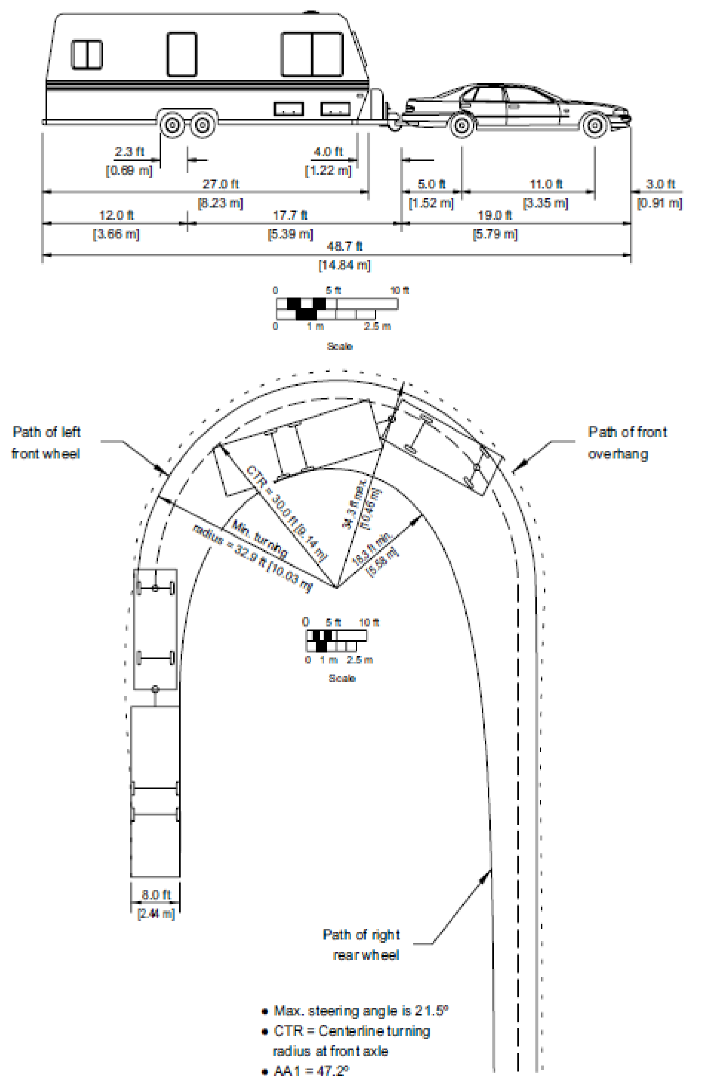

Figure 2-30. Minimum Turning Path for Passenger Car and Camper Trailer (P/T) Design Vehicle

Figure 14: Design Vehicle Parameters Selected for AutoTURN Analysis of Floodwall and Driveways

Four driveways - two north of Lakeview Drive and two to the south - were analyzed in AutoTURN for ingress and egress using the designated design vehicle. **Figure 15** shows an example analysis. These driveways were selected as representative test cases based on their proximity to the roadway and expected difficulty of access. The analyses indicate that ingress and egress are feasible; however, substantial widening of the driveway aprons will be required. It should be noted that vehicles larger than the design vehicle may be unable to access certain driveways. Further case-by-case analyses should be conducted if this alternative advances to detailed design.

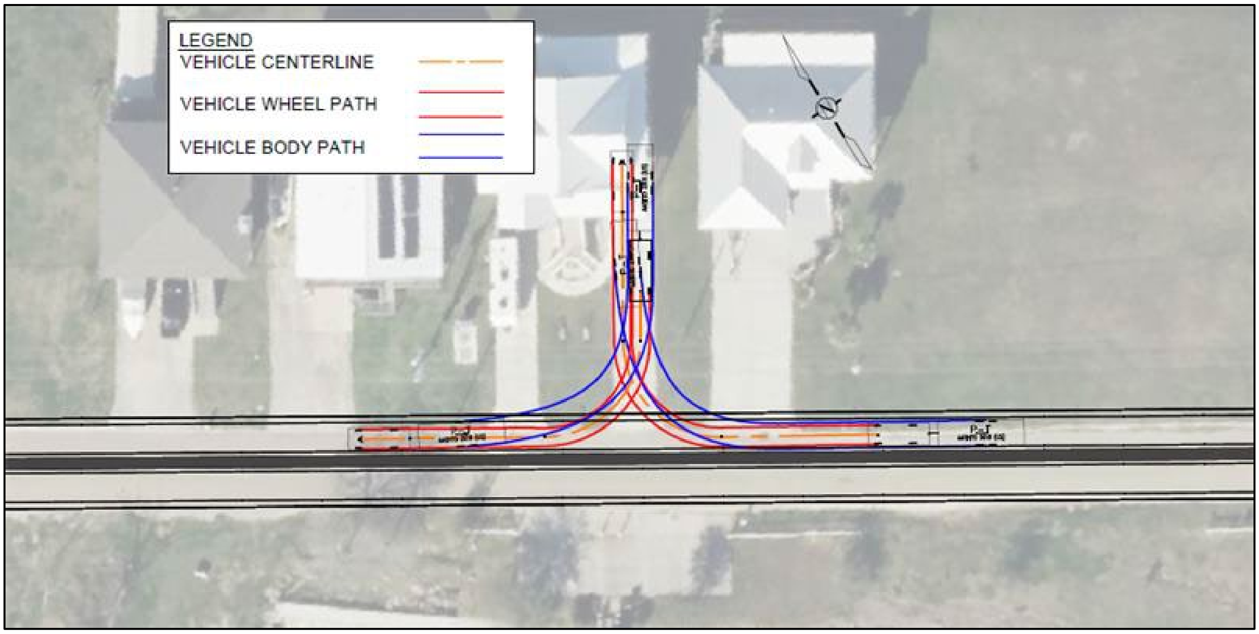

Figure 15: Example AutoTURN analysis for driveway access.

Directional U-Turn Opening Requirements

DOTD directional U-turn opening guidance require spacing at a maximum interval of 0.5 mi in each direction. With openings alternating in opposite directions, the interval becomes 0.25 mi between opposing directions. With a cul-de-sac at the east end of Lakeview Drive, this results in four required openings - two in each direction. A significant design challenge for this alternative includes selecting locations for directional U-turn openings that minimize encroachment on existing structures. Methodology for this analysis is similar to the analyses conducted for the driveways. AutoTURN was used along with these parameters based on the same Car with Camper Trailer (P/T) design vehicle and DOTD design guidance for directional U-turn opening widths and spacing. The program was also checked to see if emergency vehicles could be accommodated. The St. Tammany Fire Protection District #1 provided information for three different truck types (engine, tender, and ladder truck), used in the fleet that serves Eden Isle. AutoTURN uses pre-populated vehicle parameters, and none of the St. Tammany fire trucks were pre-set in the program, so an engine available in the program with slightly larger dimensions than the St. Tammany engine was analyzed and found to have a similar turning path as the Car with Camper Trailer (P/T) design vehicle. For this design phase, the Car with Camper Trailer (P/T) was maintained as the design vehicle, but additional analysis may be necessary in the next design phase to verify emergency vehicle turning dimensions. Note that the directional U-turn openings will not accommodate vehicles larger than the design vehicles, including the tender and ladder truck used by Fire Protection District #1. Accommodation of oversize vehicles will require either increasing size of openings, further encroaching on existing properties, or traveling to the proposed cul-de-sac at the end of Lakeview Drive to turn around. Design directional U-turn opening parameters used are shown in **Table 12**, and an example AutoTURN output is shown in **Figure 16**.

Table 12: Direction U-Turn Opening Design Parameters

Directional U-Turn Opening Design Parameters	
Design Vehicle	Single Unit Vehicle (SU), Car, Camper Trailer (P/T)
Minimum Design Radius	32.9 ft
Centerline Radius	30.0 ft
Minimum Inside Radius	18.3 ft
Opening Width	30 ft
Spacing	0.25 mi

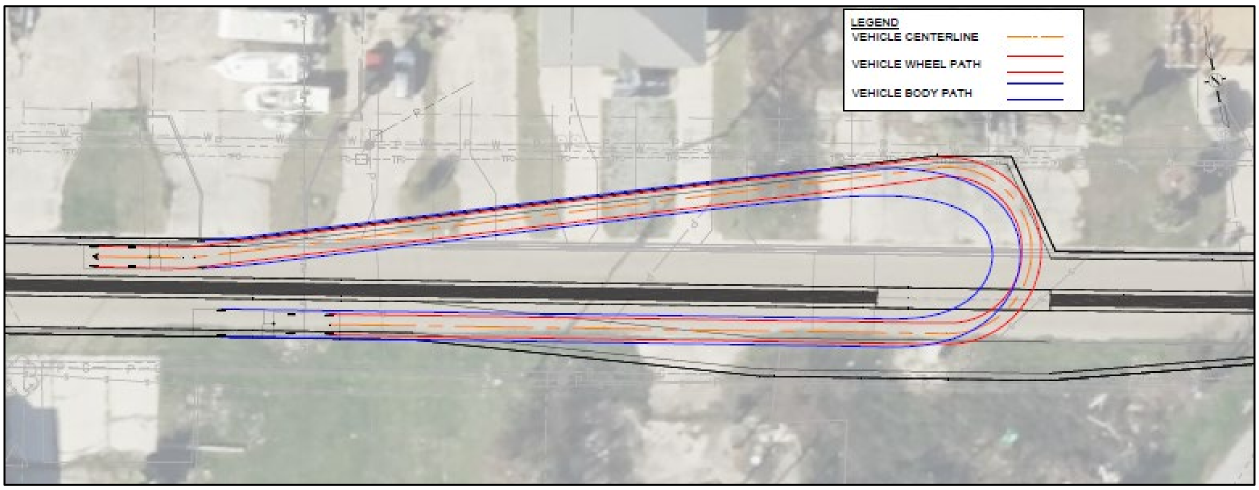

Figure 16: Example AutoTURN output for a representative Lakeview Drive Directional U-Turn Opening

As shown in **Figure 16**, the opening dimensions require a wider footprint than other areas of the pavement section to accommodate the turn radius of the design vehicle. Opening locations were placed near vacant lots where possible to minimize impacts to existing homes. Because these locations do not conform precisely to the 0.25 mi spacing requirement from DOTD, a design variance will be required.

4.1.2 Lakeview Drive Alternative B: +6.0 ft Elevation of Road, +11.0 ft Elevation Floodwall

Alternative B initially raises the roadway a consistent elevation of +6.0 ft using earth fill and base course material. This provides some limited initial resiliency benefits of a consistent elevation across Lakeview Drive. In the second phase, a floodwall would be built to a final elevation of +11.0 ft along Lakeview Drive. Two travel lanes (one in each direction) would be constructed on either side of the wall, effectively creating two separate roadways: one to the north and one to the south. This design eliminates the challenges of maintaining driveway access via directional U-turn openings. This alternative does not meet the preferred “low” interim +8.5 ft level of protection, but it alleviates many of the design challenges associated with raising the embankment in such a confined area.

Considering a phased construction approach, adequate space must be available within the roadway median for future floodwall construction. Similar to Alternative A, approximately six feet of width will be needed within the median. Floodwall designs were selected to fit within this area to avoid disturbing the roadway section during future construction.

Two options were considered for Alternative B:

1. Option 1 - Cantilever sheetpile with concrete jersey barrier style cap floodwall
2. Option 2 - PCC Pavement floodwall with tie-in to PCC travel lanes

4.1.2.1 Lakeview Drive Alternative B, Option 1 – Cantilever Sheetpile Floodwall

The cantilever sheetpile foundation option with a concrete jersey-barrier-style cap is shown in **Figure 17**. This option consists of an initial phase of embankment and roadway construction to +6.0 ft with with a 6-ft wide median. The second phase would consist of sheetpile installation in the median with a connection to a concrete jersey barrier style floodwall reaching the final elevation of +11.0 ft. The sheet pile type and tip elevation were selected to resist anticipated horizontal loads transferred from the floodwall and to satisfy stability and overturning criteria. A PZ-27 section was selected with a tip elevation of -9.0 ft. CWALSHT stability runs are included in **Appendix G**.

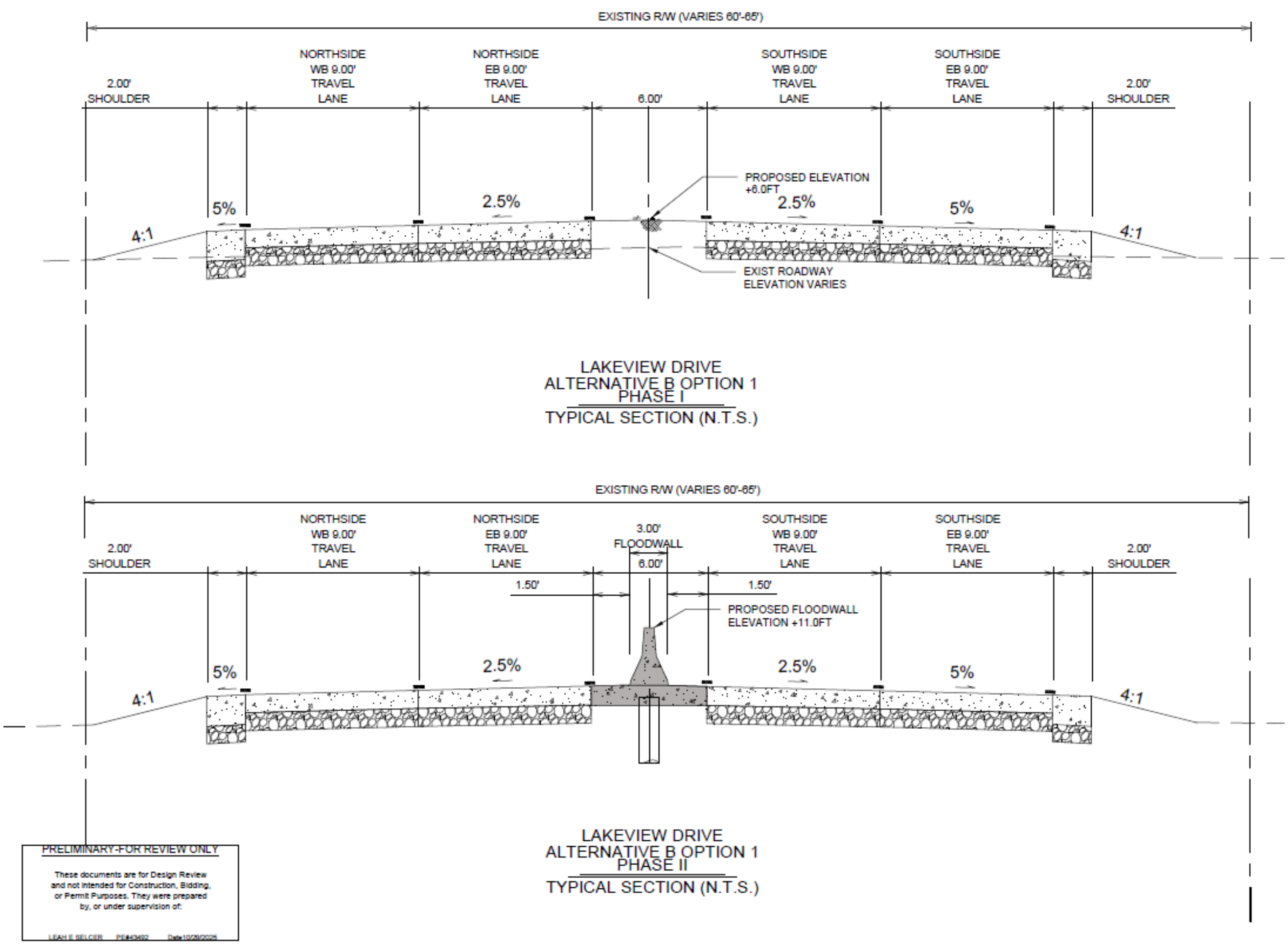

Figure 17: Alternative B, Option 1 - +6.0 ft Embankment, +11.0 ft Sheetpile Foundation Floodwall

Costs for this option are shown in **Table 13.** Note that costs are full capital costs and include construction, contingency, land acquisition, and utility costs. Costs have also been escalated to a future year (2029), to represent the time at which construction could begin. Detailed cost information is available in **Appendix H**.

Table 13: Alternative B, Option 1 Capital Costs

Phase	Work Items	Cost
Phase 1	Embankment & Road	$ 32,400,000
Phase 2	Sheetpile Floodwall	$ 36,300,000
	TOTAL:	**$ 68,700,000**

4.1.2.2 Lakeview Drive Alternative B, Option 2 – PCC Floodwall

A second floodwall option was developed that consisted of a poured PCC roadway section with toe walls on the outer edges of the pavement and a 6-ft wide grass median as part of the first phase of embankment and roadway construction. For the second phase, a concrete floodwall would be formed and poured in the roadway median. The reinforcing steel of the floodwall would be tied in to the existing PCC roadway slab, resulting in a continuous floodwall and roadway as shown in **Figure 18.** Stability and overturning analyses were performed to develop a conceptual cross section, and costs are included in **Table 14**. Overturning analyses are included in **Appendix G**. Detailed cost information is available in **Appendix H**.

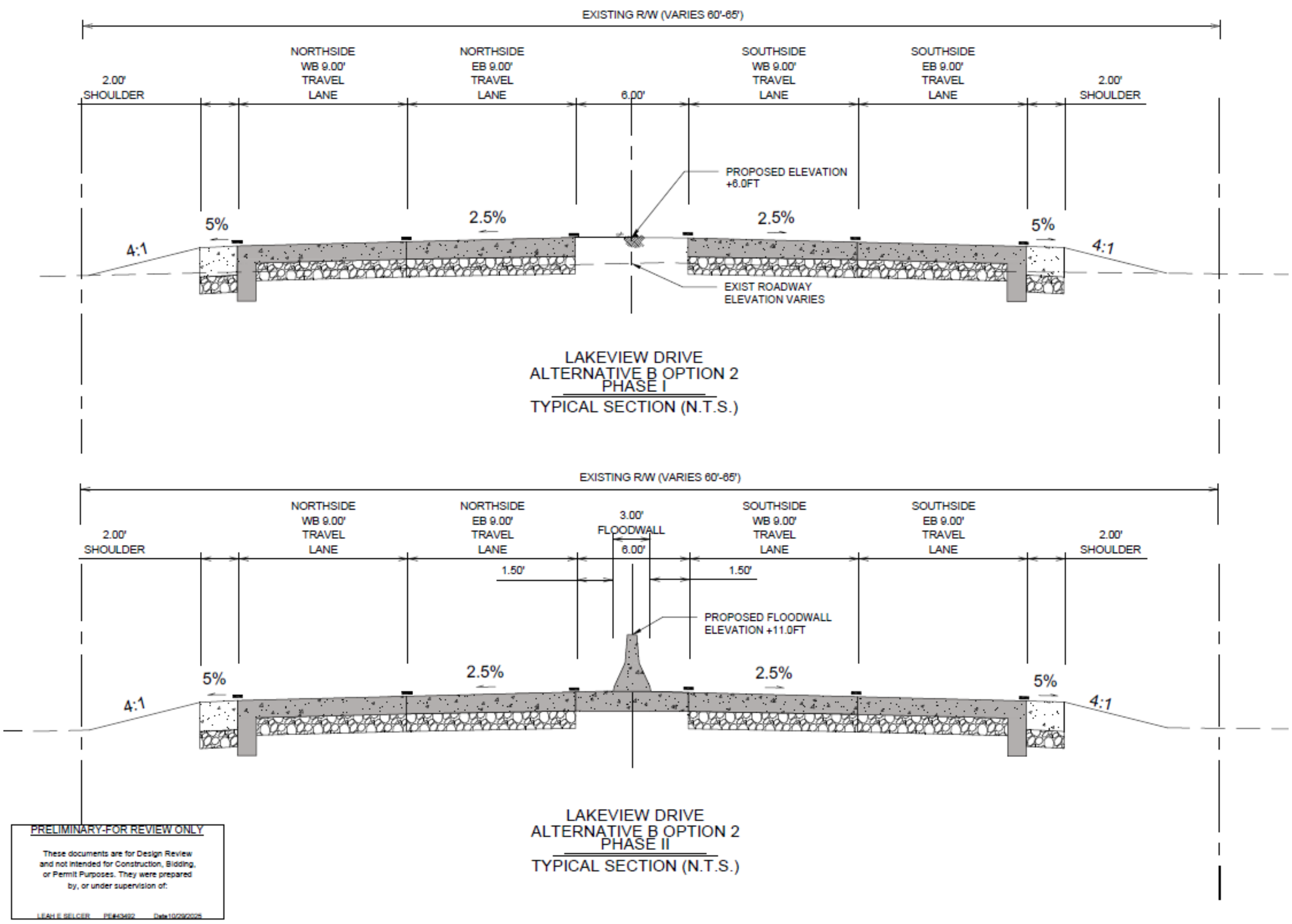

Figure 18: Alternative B, Option 2 - +6.0 ft Embankment, +11.0 ft PCC Floodwall

Table 14: Alternative B, Option 2 Capital Costs

Phase	Work Items	Cost
Phase 1	Embankment & Road	$ 36,200,000
Phase 2	PCC Floodwall	$ 26,500,000
	TOTAL:	**$ 62,700,000**

4.1.2.3 LAKEVIEW DRIVE ALTERNATIVE B - DESIGN CONSIDERATIONS

Alternative B alleviates many of the design challenges that Alternative A presents. The existing elevation varies along Lakeview Drive, but averages approximately +5.5 ft. As a result, embankment quantities are minimal to bring the finished elevation of the median to +6.0-ft and driveway access concerns are reduced or eliminated. The two travel lanes on each side of the floodwall allow turns to be made across the opposing travel lanes into existing driveways, providing ingress and egress to all driveways and eliminating the need for turnarounds.

The drawbacks of Alternative B are its limited initial benefit and higher overall cost. The first phase achieves a uniform elevation of +6.0 feet, providing only modest resiliency until subsequent phases are completed. This alternative is more expensive than Alternative A and may raise aesthetic concerns, as the proposed wall height will obstruct visibility at ground level.

Construction Access

The two lanes on each side of the floodwall will allow two-way traffic to be maintained throughout construction, reducing overall logistical challenges. However, sequencing will still be required to ensure daily access to homes adjacent to lanes under active construction. Similar to Alternative A, base course aggregate material will be used to construct the road embankment so it can be driven on at the end of each workday and to provide daily ingress and egress of driveways in active work zones.

Utilities

Impacts to existing utilities for Alternative B are similar to Alternative A. Because Alternative B results in a slightly wider overall impact area, there are minor increases in utility impacts. As with Alternative A, utility relocation costs were limited to impacts occurring outside the ROW but within the anticipated limits of construction. Additionally, for Option 1, it was assumed that parallel utility services would be installed on the opposite side of the roadway in lieu of multiple penetrations through the proposed sheetpile wall.

4.1.3 LAKEVIEW DRIVE ALTERNATIVE C: +8.0 FT ELEVATION OF ROAD, SELF-DEPLOYING FLOOD BARRIERS

Alternative C raises the roadway to elevation +8.0 ft using earthen embankment, providing initial resilience. During the second phase, a passive automatic flood barrier such as FloodBreak, or similar, would be installed along Lakeview Drive, with one travel lane on each side of the floodwall. An initial road section using PCC is required for Phase 1 of this alternative to accommodate incorporation of the passive flood barrier foundation in Phase 2. Examples of these flood barriers are shown below in **Figure 19**.

Figure 19: Passive flood barrier shown in passive state (left) and deployed state (right) (Source: www.floodbreak.com)

As with Alternatives A and B, adequate space is required in the roadway median to accommodate future floodwall construction. This alternative requires a slightly wider median than the others, at 10 ft 2 inches, to allow for installation of the receiving foundation for the flood barrier system, as shown in **Figure 20**.

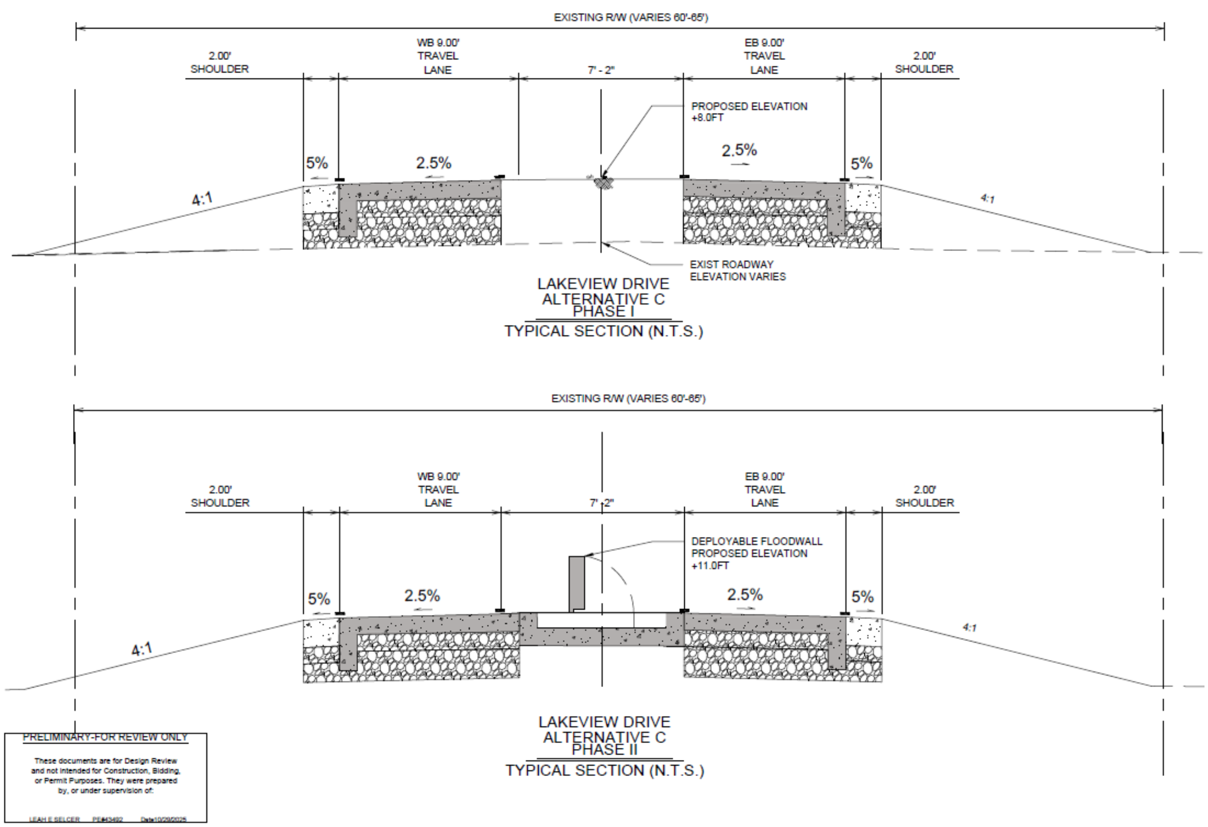

Figure 20: Alternative C: +8.0 ft Embankment, +11.0 ft

Costs for this Alternative are in **Table 15**. Note that costs are full capital costs and include construction, contingency, land acquisition, and utility costs. Costs have also been escalated to a future year (2029), to represent the time at which construction could begin. Detailed cost information is available in **Appendix H**.

Table 15: Alternative C Capital Costs

Phase	Work Items	Cost
Phase 1	Embankment & Road	$ 22,700,000
Phase 2	Floodbreak & Floodwall	$ 120,700,000
	TOTAL:	**$ 143,400,000**

4.1.3.1 LAKEVIEW DRIVE ALTERNATIVE C - DESIGN CONSIDERATIONS

Alternative C alleviates many of the design challenges that Alternative A presents. The issue of directional U-turn openings and driveway access from cross traffic is lessened. When the flood barriers are not deployed, they allow for ingress and egress to driveways even with larger sized vehicles and/or trailers. The passive flood barriers are traffic rated and allow for turns from the opposing traffic lane across the barrier. When the barriers are not deployed, they do not impede the view across Lakeview Drive like a permanent floodwall does.

Alternative C does have drawbacks. The passive flood barriers require "wiper walls" to serve as watertight seals at the end of each section, limiting installation to straight sections of road. **Figure 21** shows potential locations of passive flood barriers and permanent floodwall. Approximately 25% of Lakeview Drive will still require a permanent floodwall. Similar to Alternative A, the initial elevation of +8.0-ft restricts access to existing driveways at the grades as required by DOTD, shown in **Table 10**. While the embankment height for Alternative C is slightly lower than Alternative A, the median width is larger, requiring a nearly identical footprint. Alternative A is the most expensive of all the Alternatives considered.

Figure 21: Permanent versus Passive Flood Barrier Locations Along Lakeview Drive

Construction Access

Alternative C presents the same challenges during construction as Alternative A. Construction would be phased in the same way as Alternative A, with daily construction lifts using base course aggregate and two-way traffic handled in the opposite lane with a flagger.

Utilities

Impacts to existing utilities and resulting relocation costs for Alternative C are similar to, but slightly greater than, Alternative A, Option 2 and Alternative B, Option 2. Although the overall footprint is similar to Alternatives A and B, there are slightly more impacts to electric, water, and gas for Alternative C. As with the previous alternatives, relocation costs were limited to impacts occurring outside the ROW but within the anticipated limits of construction. Because no sheetpile foundation is proposed at this stage for this alternative, no costs were carried for installation of parallel main utility lines.

4.1.4 Lakeview Drive Alternative D: +11.0 ft Levee North of Lakeview Drive

Alternative D consists of constructing a continuous earthen levee along the north side of the roadway to achieve the target elevation of +11.0 ft. The intent of this alternative is to provide a more conventional levee configuration that reduces roadway design and operational constraints while meeting the desired level of protection. The continuous embankment configuration minimizes the need for roadway crossings or floodgate structures, reducing structural complexity and long-term O&M requirements. This alternative does, however, require acquisition of all property along the north side of Lakeview Drive and removal of all structures.

The proposed levee alignment generally parallels Lakeview Drive from Highway 11 to its terminus at Grand Lagoon and is approximately centered on the land between Lakeview Drive and the canal to the north. Typical levee sections were developed with a crest elevation of +11.0 ft and an overbuild of 1.0 ft to account for settlement and future maintenance. A top width of approximately 10 ft was assumed for inspection and maintenance access. In the absence of site-specific geotechnical data, a 4H:1V side slope has been assumed, consistent with typical levee configurations in south Louisiana. A landside toe riprap scour pad was incorporated into the design since overtopping may occur during the life of the design. Scour pad dimensions cover approximately the bottom 1/3 of the levee slope with a key in at the levee toe. A minimum offset of 25 feet was assumed from the existing bulkhead and waterway north of Lakeview Drive. A representative section is shown below in **Figure 22.** Additional details are included in **Appendix F**. Note that survey data collected during this phase focused primarily on Lakeview Drive, and full survey coverage was not available for the proposed levee alignment. As a result, the existing layout for this alternative is based on available Light Detection and Ranging (LiDAR) data. If this alternative is advanced, additional survey would be required to support detailed design.

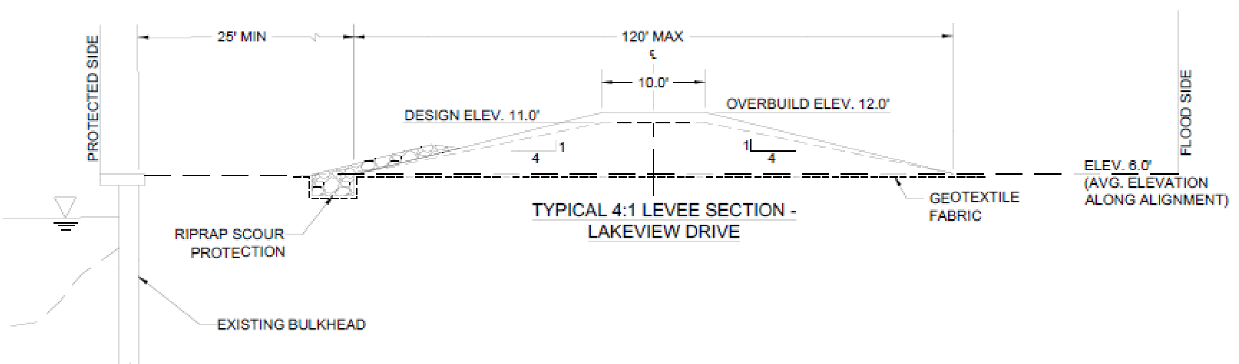

Figure 22: Alternative D, Typical Levee Section Example

4.1.4.1 Lakeview Drive Alternative D, Option 1 – Levee North of Northshore Circle

Option 1 maintains the levee entirely north of Lakeview Drive and Northshore Circle, as shown in **Figure 23**. East of the Lakeview Drive and Northshore Circle intersection, the alignment is identical to Option 2. West of the intersection, the levee continues north of the roadway without crossing Lakeview Drive.

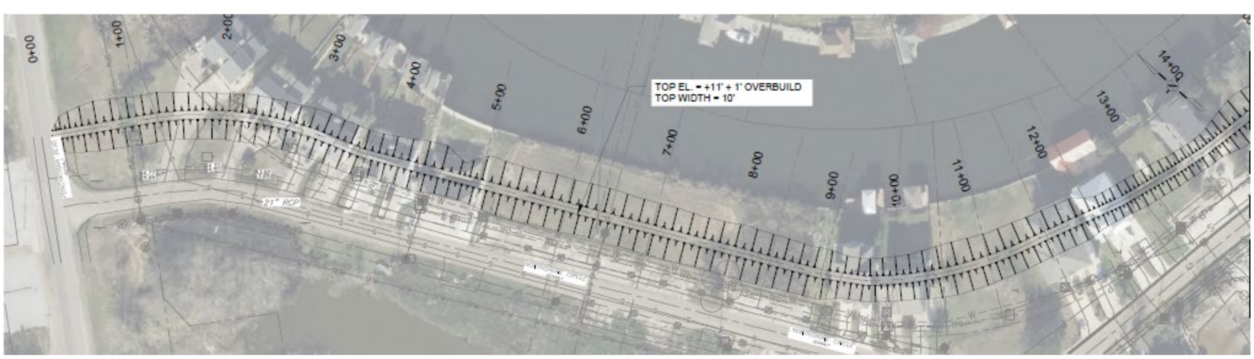

Figure 23: Alternative D, Option 1

The continuous embankment configuration eliminates the need for roadway crossings or floodgate structures along Lakeview Drive, reducing structural complexity and long-term O&M requirements. This option does, however, require acquisition of all property along the north side of Northshore Circle.

Estimated costs for Alternative D, Option 1 are summarized in **Table 16**. Real estate acquisition costs are based on a cost of $150,000/ac for undeveloped land south of Northshore Circle, $350,000/ac for vacant lots along Lakeview Drive, and $375,000 per dwelling. Note that costs have been escalated to a future year (2029), to represent the time at which construction could begin, and include a 35% contingency. Detailed cost assumptions are provided in **Appendix H**.

Table 16: Alternative D, Option 1 Costs

Item	Cost
Real Estate / Utility Costs	$39,900,000
Construction Costs	$19,500,000
Total:	**$59,400,000**

4.1.4.2 Lakeview Drive Alternative D, Option 2 – Levee South of Northshore Circle

Option 2 modifies the levee alignment west of the Lakeview Drive and Northshore Circle intersection by crossing Northshore Circle and transitioning the levee to the south side of Northshore Circle, as shown in **Figure 24**. East of the intersection, the alignment is identical to Option 1. This configuration avoids direct impacts to residential properties located north of Northshore Circle and reduces the number of acquisitions required in that area. This option requires construction of a road crossing at Lakeview Drive, assumed at this stage as a Floodbreak passive gate system.

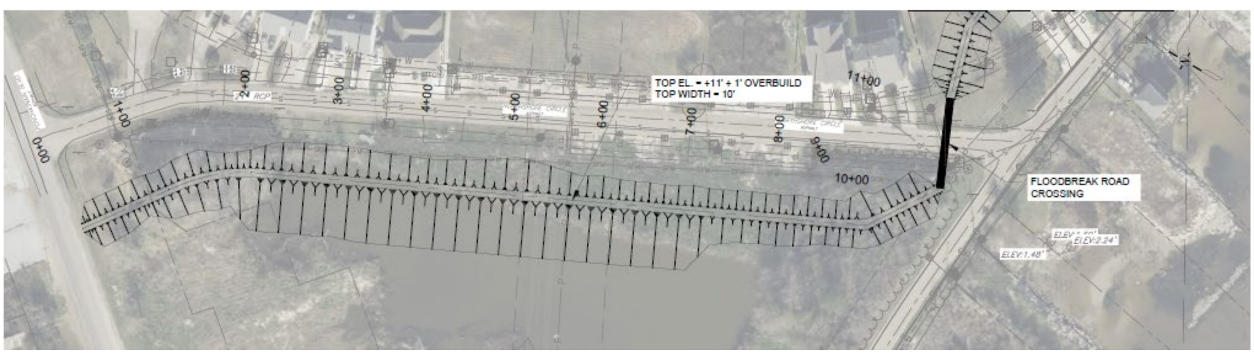

Figure 24: Alternative D, Option 2

Option 2 requires slightly more embankment volume due to the presence of an existing pond along the revised alignment. The pond would need to be at least partially filled to establish a stable levee foundation and achieve the required embankment geometry. This increases earthwork quantities and introduces additional foundation preparation considerations relative to Option 1.

Estimated costs for Option 2 are summarized in **Table 17**. Note that costs have been escalated to a future year (2029), to represent the time at which construction could begin, and include a 35% contingency. Detailed cost assumptions are provided in **Appendix H**.

Table 17: Alternative D, Option 2 Costs

Item	Cost
Real Estate / Utility Costs	$35,600,000
Construction Costs	$22,700,000
Total:	**$58,300,000**

4.1.4.3 *LAKEVIEW DRIVE, ALTERNATIVE D – DESIGN CONSIDERATIONS*

Alternative D would require acquisition of all properties north of Lakeview Drive, as the levee footprint, side slopes, and construction access are located primarily north of the existing public right-of-way. Accordingly, residential property buyouts have been assumed for planning-level evaluation. Advancement of this alternative would require coordination with affected property owners and compliance with applicable state and federal acquisition and relocation requirements.

Unlike Alternatives A through C, this alternative does not readily lend itself to phased construction to interim levels of protection. Earthen levee construction is most efficiently implemented by constructing the full section to the target elevation in a single phase. As a result, phasing to provide an interim level of protection is not considered practical for this alternative.

Utilities

Impacts to existing utilities for Alternative D differ from the previous alternatives since the work occurs mostly to the north outside of the ROW. For both options of this alternative, the levee alignment was optimized to minimize impacts to utilities. In locations where the southern work limits did overlap utilities, a relocation impact was assumed. Because this alternative requires removal of all structures, utility service disconnections for each structure were assumed.

4.1.5 LAKEVIEW DRIVE COST SUMMARY

Table 18 summarizes feasibility phase costs associated with all the potential alternatives along Lakeview Drive. These costs represent future (2029) construction costs and include land acquisition costs, utility relocation costs, and a 35% contingency to reflect the conceptual level of design at the feasibility stage.

Table 18: Lakeview Drive Cost Summary

Lakeview Drive Alternatives		
Alternative	**Description**	**Total Cost**
Alt A, Option 1	Phase 1 Asphalt Roadway Raising to +8.5 ft	$23,900,000
	Phase 2 Sheetpile Floodwall to +11.0 ft	$30,500,000
	Total:	***$54,400,000***
Alt A, Option 2	Phase 1 PCC Roadway Raising to +8.5 ft	$23,100,000
	Phase 2 Floodwall to +11.0 ft	$20,200,000
	Total:	***$43,300,000***
Alt B, Option 1	Phase 1 Asphalt Roadway Raising to +6.0 ft	$32,400,000
	Phase 2 Sheetpile Floodwall to +11.0 ft	$36,300,000
	Total:	***$68,700,000***
Alt B, Option 2	Phase 1 PCC Roadway Raising to +6.0 ft	$36,200,000
	Phase 2 Floodwall to +11.0 ft	$26,500,000
	Total:	***$62,700,000***
Alt C	Phase 1 PCC Roadway Raising to +8.0 ft	$22,700,000
	Phase 2 Floodbreak to +11.0 ft	$120,700,000
	Total:	***$143,400,000***
Alt D, Option 1	Levee N. of Northshore Cir. to +11.0' ***Total:***	**$59,400,000**
Alt D, Option 2	Levee S. of Northshore Cir. to +11.0' ***Total:***	**$58,300,000**

4.1.6 Operations and Maintenance Considerations

Detailed O&M costs were not developed as part of this feasibility-level evaluation. However, planning-level cost ranges were derived based on anticipated long-term maintenance requirements of major features for each alternative.

Typical O&M activities include the following:

- Floodwalls
  - Inspections (Annual)
  - Monitoring Evaluation and Report (Every 10 yrs and/or after major storm events)
  - Repairing cracking and spalling of concrete floodwalls
  - Rehab or replacement of exposed metal or connections
- Floodbreak System and Floodgates
  - Inspections (Quarterly and Yearly)
  - Monitoring Evaluation and Report (Every 10 yrs and/or after major storm events)
  - Rehab / Replacement of gaskets, seals, hydraulic arms, structural components etc.
- Levees
  - Inspections (Quarterly and Yearly)
  - Monitoring Evaluation and Report (Every 10 yrs and/or after major storm events)
  - ROW Maintenance / Mowing
  - Addition of material to maintain levee elevation (Every 10 yrs or as necessary)

Planning-level operation and maintenance costs were developed using representative values and guidance presented in the USACE National Levee Safety Program Cost Brochure (2023), in combination with information obtained from FloodBreak regarding its “Flood Ready Certification” program. Under this program, annual inspections are performed and routine maintenance is performed to ensure proper performance. Participation in the program also extends the product warranty on an annual basis for each year the certification is maintained. These references were used to develop the conceptual annual O&M ranges presented in **Table 19**.

Table 19: Lakeview Drive Estimated O&M Costs

Lakeview Drive O&M Costs		
Alternative	**Description**	**Annual O&M Cost**
Alt A	2.5’ Tall Floodwall + Floodgates	\$150,000 - \$200,000
Alt B	5.0’ Tall Floodwall	\$50,000 - \$100,000
Alt C	3.0’ Tall Floodbreak System	\$550,000 - \$600,000
Alt D	+11.0’ Elevation Levee + Floodgates	\$150,000 - \$250,000

Costs are presented in 2031 dollars to represent costs in Year 0 following construction completion and would be subject to cost escalation over the project life. For alternatives incorporating FloodBreak gates, estimated O&M costs are largely driven by participation in the Flood Ready Certification program. Long-term O&M costs for these alternatives could potentially be reduced if recurring inspection and maintenance activities are conducted in-house rather than through the certification program.

4.2 Harbor Drive

The easternmost portion of the project alignment concludes along Harbor Drive, between Grand Lagoon and US Interstate 10. This area is comprised of multiple undeveloped parcels and a primary road for access. Properties along the lake have an existing bulkhead along the water's edge, providing protection from Lake Pontchartrain and Grand Lagoon.

The road through the area is constructed of concrete with mountable curbs along the edge of the pavement. There is one set of drainage inlets within the concrete curbs are found at the midpoint of the 900-foot-long road. The elevations along Harbor Drive are similar to Lakeview Drive with elevations ranging from +4.7 to +7.0 ft. Adjacent to the road, the land rises above +8.0 ft through most of the southern properties and the seawall has an elevation of +12.0 ft.

The western edge of the seawall has several points of failure which may have occurred during storm events in the area. This damage has exposed the supporting components of the bulkhead and exposed the shoreline to erosive wave conditions.

Four different alternatives were considered, as shown in **Figure 25.** These include elevating Harbor Drive, a levee adjacent to the road, a levee in the center of the properties south of the road, and repair or rebuilding the bulkhead along the lakefront. Phased implementation was considered when possible to achieve the full elevation of +11.0 ft. Costs for each alternative are shown in **Table 20**.

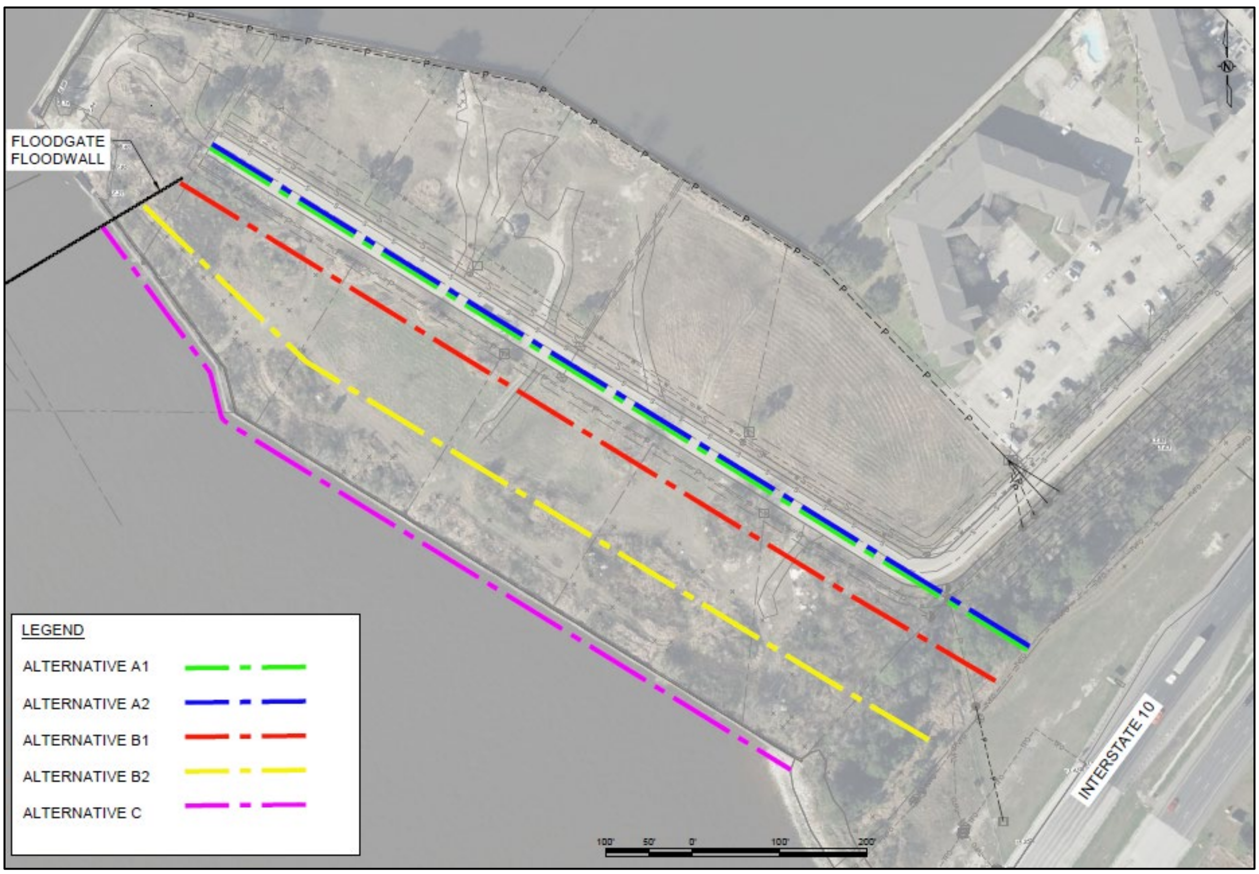

Figure 25: Alignments Considered for Harbor Drive

4.2.1 Harbor Drive Alternative A: Elevation of the Road

Alternative A proposed increasing the elevation along Harbor Dr. to achieve the target elevation. Two options were considered for this alternative:

Option 1 – Raise the road to an initial elevation to +8.5 ft and construct a floodwall as part of a second phase to an elevation of +11.0 ft (**Figure 26**).

Option 2 – Raise the road in a single construction phase to an elevation of +11.0 ft (**Figure 27**).

4.2.1.1 Harbor Drive Alt A, Option 1 – Phased Implementation

Option 1, is similar to Alternative A along Lakeview Drive, and would consist of embankment placed along Harbor Drive with a 6-ft median for a future floodwall. During the second phase, a floodwall would be constructed in the median. Some limited encroachment at the toe of the embankment would occur outside the ROW thus requiring property acquisition in these areas.

A median floodwall presents the same challenges as on Lakeview Drive regarding property access and turnarounds; however, the short road length alleviates some of these concerns. Since the road is less than 0.5 mile long, no openings in the floodwall would be required for turnarounds. If this alternative is implemented, property access would require traveling to the end of the roadway to turn around when entering or exiting, depending on the side of the road on which the property is located. Grading for driveway access would be required for this alternative, but because all the existing lots are vacant, grading could occur during embankment construction or by individual owners as their lots are developed. The higher elevation of these lots as compared to Lakeview Drive also makes grading less of a challenge.

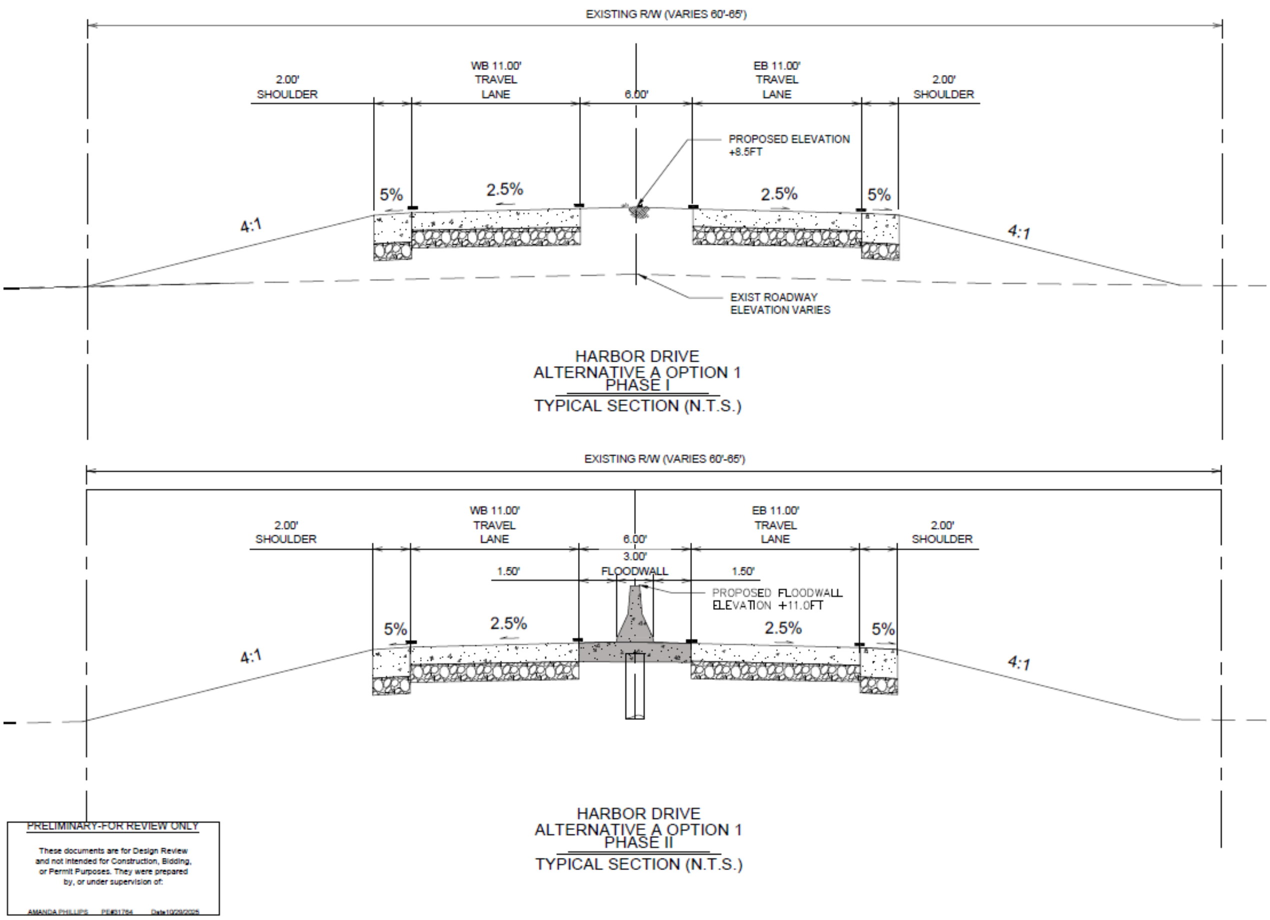

Figure 26: Phased Implementation of Road Raising for Harbor Drive

4.2.1.2 *Harbor Drive Alt A, Option 2 – Single Phase Implementation*

Option 2 embankment and road construction occurs in a single phase to the full elevation of +11.0 ft. This option eliminates the cost and of the floodwall and the requirement to travel to the end of the road to enter or exit each driveway. Grading individual driveways would be necessary but the elevations of the adjacent lots make this feasible. Impacts to adjacent properties would occur outside the ROW requiring partial acquisition of properties; however, these impacts would be minimal. **Table 20** shows this is the second most cost-effective option.

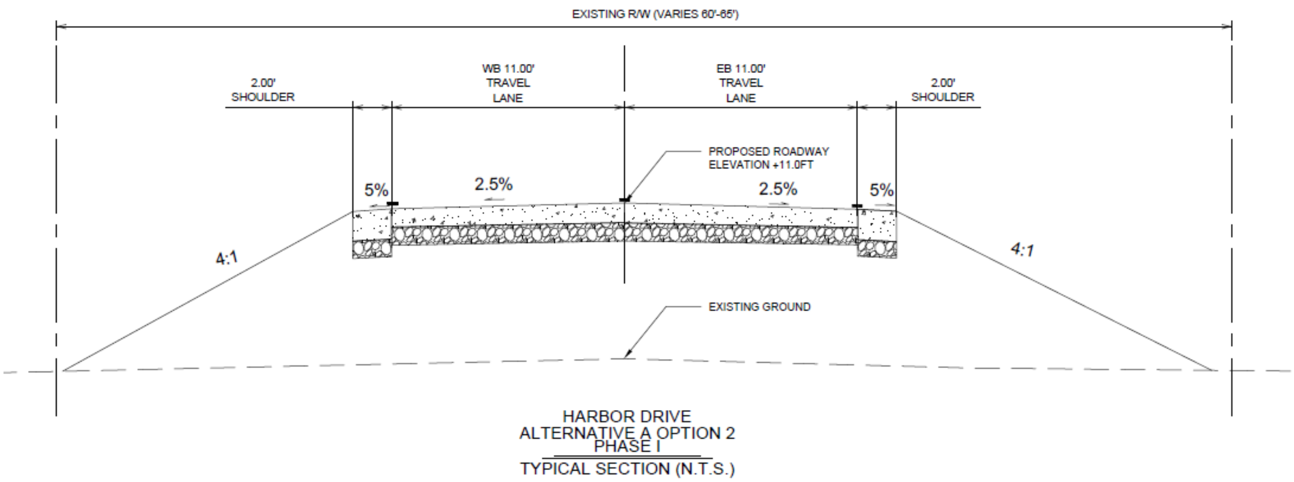

Figure 27: Single phase Implementation of Harbor Drive Road Raising to +11.0 ft

4.2.2 Harbor Drive Alternative B: Levee South of the Roadway

Alternative B consists of levee construction within the parcels to the south of Harbor Drive. Benefits of this alternative are minimal damage to the existing roadway and no impacts to residents or structures since all the lots are vacant. Also, the elevations of these lots are relatively high which minimizes overall impacts and cost. Two options were considered for this alternative:

Option 1 – Placement of the levee directly adjacent to the roadway (**Figure 28**).
Option 2 – Placement of the levee near the middle of the properties (**Figure 29**).

4.2.2.1 *Harbor Drive Alt B, Option 1 – Levee Adjacent to the Roadway*

Option 1 consists of construction of a levee immediately south of Harbor Drive. This option is a cost-effective embankment-only alternative to avoid disturbance to the roadway and minimizes impacts to adjacent properties. Phased construction was considered but based on the cost of this project feature it will be more efficient to construct in one phase. Impacts beyond the footprint of the levee were assumed to accommodate grading for vehicle access from the roadway up and over the levee and then return to the natural grade south of the levee. The placement of the levee centerline was selected considering the vehicle constraints for driveway access. Costs are shown in **Table 20** and include partial acquisition of properties that would be required.

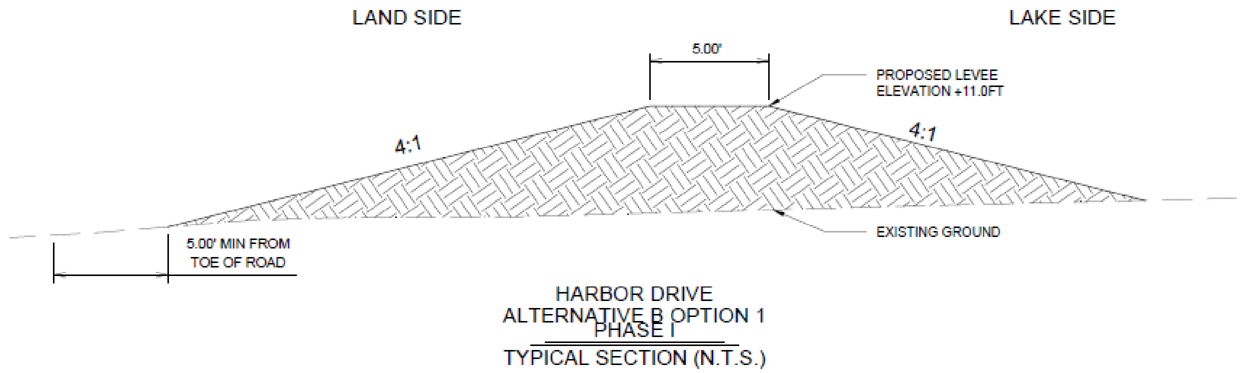

Figure 28: Levee Adjacent to Roadway Alternative

4.2.2.2 Harbor Drive Alt B, Option 2 – Levee In the Center of Properties

Option 2 places the levee south of the roadway approximately midway along the vacant land parcels. This location was selected by identifying the highest natural ground of the parcels to minimize the material required for construction. However, this option has the most impact to properties. Construction of a levee and acquisition of ROW for a levee across the properties greatly affects the usability of the properties. Therefore, full property acquisition was assumed for this option. The existing bulkhead along the lakefront is the responsibility of individual property owners and if full acquisition of the parcels occurs, the State will be responsible for the bulkhead. For this reason, costs to remove the bulkhead, slope the shoreline and install shoreline protection were included with this option, as shown in **Table 20**.

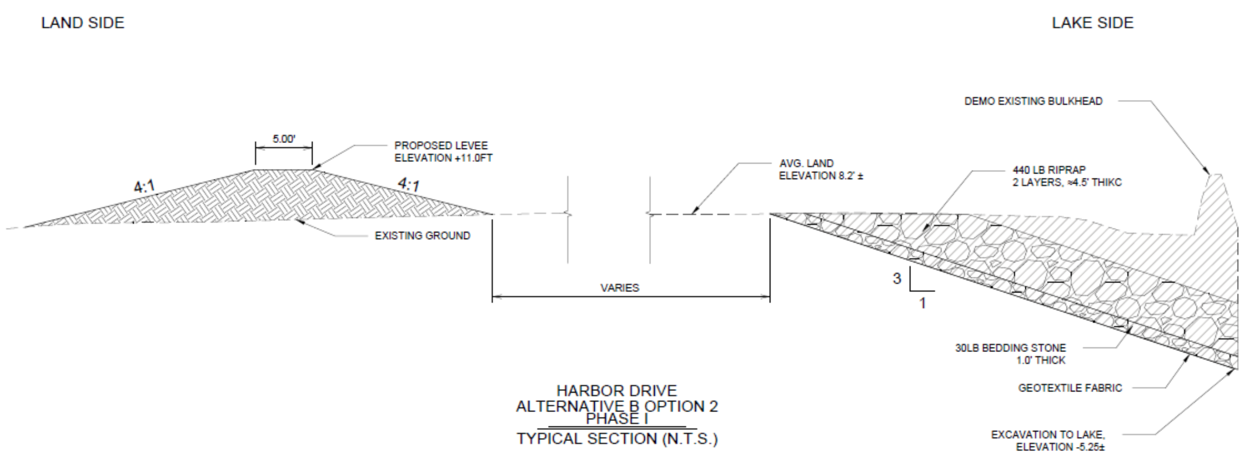

Figure 29: Levee in Center of Properties Alternative

4.2.3 Harbor Drive Alternative C: Bulkhead Replacement

Alternative C considers utilizing the existing bulkhead for flood protection. The elevation of the existing structure is approximately +12.0 ft, exceeding the targeted elevation of this project (+11.0 ft). Use of this structure was considered as part of the flood protection system, however, due to its poor condition, a full replacement of the structure was assumed to be necessary. The replacement structure proposed is a new steel sheetpile wall, lakeward of the existing structure, as shown in **Figure 30.** This location allows for new support pilings to be installed between the new and existing bulkheads. This area will then be backfilled with earthen material to complete the reconfiguration of the shoreline. As shown in **Table 20**, this was the most expensive alternative developed.

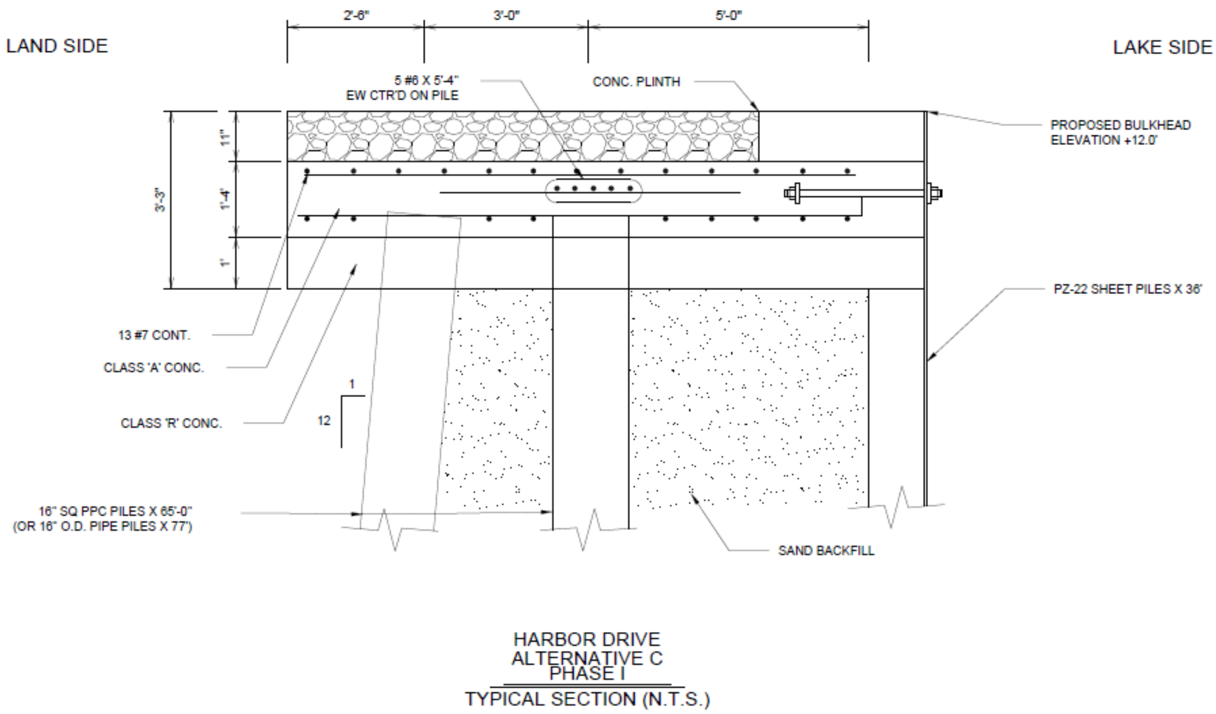

Figure 30: Bulkhead Replacement Alternative

4.2.4 HARBOR DRIVE COST SUMMARY

Table 20 summarizes feasibility phase costs associated with all the potential alternatives along Harbor Drive. These costs represent future (2029) construction costs and include land acquisition costs, utility relocation costs, and a 35% contingency to reflect the conceptual level of design at the feasibility stage.

Table 20: Harbor Drive Alternative Costs

Harbor Drive Alternatives		
Alternative	**Description**	**Total Cost**
Alt A Option 1	Phase 1 Roadway Raising to +8.5 ft	$3,400,000
	Phase 2 Floodwall Installation to +11.0 ft	$5,300,000
	Total:	***$8,700,000***
Alt A Option 2	Roadway Raising to +11.0 ft	***$4,000,000***
Alt B Option 1	+11.0’ Levee Adjacent to Roadway	***$1,300,000***
Alt B Option 2	+11.0’ Levee in Middle of Properties	***$9,600,000***
Alt C	Bulkhead Replacement	***$19,600,000***

4.2.5 Operations and Maintenance Considerations

Similar to Lakeview Drive, planning-level O&M estimates were derived based on the USACE National Levee Safety Program Cost Brochure (2023), in combination with information obtained from FloodBreak. These cost ranges are shown in **Table 21.**

Table 21: Harbor Drive Estimated O&M Costs

Harbor Drive O&M Costs		
Alternative	**Description**	**Annual O&M Cost**
Alt A, Option 1	2.5' Tall Floodwall + Floodgates	\$25,000 - \$50,000
Alt A, Option 2	+11.0' Road Lift	\$5,000 - \$10,000
Alt B	+11.0' Levee	\$10,000 - \$15,000
Alt C	+12.0' Bulkhead	\$50,000 - \$75,000

Costs are presented in 2031 dollars to represent costs in Year 0 following construction completion and would be subject to cost escalation over the project life.

4.2.6 Harbor Drive Drainage

At the eastern terminus of the project, Harbor Drive turns north and runs parallel to Interstate 10. The narrow area between the two roads facilitates local drainage from both roads and flows south to Lake Pontchartrain. To maintain this connection, a drainage structure is required at this location. Hydraulic modeling was performed using HEC-RAS 2D to determine structure size requirements and to inform location and elevation. Based on this analysis, a 42" reinforced concrete pipe with one-way flow control is proposed. The one-way feature will allow normal surface drainage to discharge southward while preventing backflow into the protected area during storm surge or elevated water levels in Lake Pontchartrain.

4.3 Grand Lagoon Floodgate

A floodgate across Grand Lagoon is a key component of the proposed system. Eden Isles is a waterfront community, with most homes having waterfront access, leading from interior canals to Lake Pontchartrain. A floodgate for the system must allow unimpeded access for the common vessel types entering and exiting the neighborhood water bodies.

4.3.1 Gate Type

Numerous floodgate options were considered based on usability, functionality, and costs of construction and O&M. The barge gate type and sector gate type were selected for detailed evaluation. These gate types are used extensively in South Louisiana, have a long design history, and a track record of successful performance during storm events.

A barge gate consists of a buoyant closure structure that floats into position across a channel during storm events and is ballasted down with water, forming a watertight barrier. After water levels recede, the barge is emptied and returned to its storage bay position. A sector gate consists of two curved steel leaves that rotate inward from recessed bays to close a channel. The curved leaves meet at the center forming a watertight seal and are reopened after the event. Examples of these gate types are shown below in **Figure 31** and **Figure 32**.

Figure 31: Barge Gate Example (Source: Terrebonne Levee & Conservation District, https://www.facebook.com/TerrebonneLevee/)

Figure 32: Sector Gate Example (Source: Southeast Louisiana Flood Protection Authority East https://www.floodauthority.org/the-system/bayous-bienvenue-and-dupre-sector-gates/)

Both gate types are suitable for this application. A sector gate exhibits several superior operations characteristics such as ease of operation, safety, and time to open and close. A barge gate is easier to construct, more prevalent in South Louisiana, and less costly.

A parametric cost estimate was initially developed for each gate type to establish an approximate construction cost and aid in the decision of a gate type. Construction costs of previously constructed barge gates and sector gates were used and inflated to a common year using the method and factors outlined in USACE EM-1110-2-1304 *Civil Works Construction Cost Index System* for Project Type: Locks. The overall length of each gate structure was determined by measuring from tie-in to tie-in on aerial photography. The costs were then divided by the length of structure to determine the cost per linear foot. The results were analyzed and appropriate values were selected for use and extrapolated to the anticipated gate structure length at Grand Lagoon. Based on this analysis, it is anticipated that a sector gate would cost approximately three times as much as a barge gate, which eliminated the sector gate from further consideration. The barge gate was carried forward for more detailed analysis.

4.3.2 Gate Opening Size

A key design parameter for the Grand Lagoon floodgate is the minimum gate opening width. This was developed using applicable design guidance and local research on vessel usage. The World Association of Waterborne Transport Infrastructure (PIANC) serves as an international authority on design standards and best practices for navigation, port, and waterway infrastructure. Its *Working Group (WG) 149 – Guidelines for Marina Design* includes beam widths for various boat lengths and provides guidance on navigation openings. **Table 22** and **Table 23** show values presented in WG-149.

Table 22: Typical Vessel Length vs Beam Width

Vessel Length	Typical Beam Range
18-22 ft	7.5 – 8.5 ft
23-26 ft	8.5 – 9.5 ft
27-30 ft	9.5 – 10.5 ft
31-36 ft	10.5 – 12 ft
37-42 ft	12 – 14 ft
43-50 ft	14 – 16 ft

Table 23: PIANC Guidance on Navigation Openings

Vessel Type (Vessel Length)	Recommended Clear Opening (one-way passage)	Recommended Clear Opening (two-way passage)
Small boats (<30 ft)	Vessel beam + 4-6 ft	2x beam + 6-10 ft
Medium Vessels (30-100 ft)	Vessel beam + 6-10 ft	2x beam + 10-15 ft
Large Yachts (100 ft)	Case-specific	Typically managed one-way

A phone interview was conducted with the manager of Oak Harbor Marina located in Grand Lagoon to establish typical vessel sizes in the marina. Based on this conversation and by measuring vessels visible on aerial photography, it was determined that typical vessel sizes that frequent the marina are recreational fishing vessels ranging in size from 18-30 ft in length. The boat slips in Oak Harbor Marina can accommodate boats up to 20 ft wide and 55 ft long. The maximum vessel size in the marina is a yacht 105 ft long with a draft of approximately 8-10 ft. Additional conversation with homeowners established that the largest vessel commonly known to enter the waterways is a barge approximately 30’ wide used for dock construction in the neighborhood.

Based on the representative vessel dimensions and PIANC guidance, a minimum 40-foot gate width is recommended. This opening would accommodate two-way passage for most recreational vessels typical of the marina and one-way passage for the largest vessels accessing Grand Lagoon. A sill elevation of -10 feet was established considering both the maximum vessel draft within the marina and reference elevations from comparable barge gates in South Louisiana.

The gate opening width and sill elevations should be considered preliminary and will be further evaluated during subsequent phases of design. While design guidance indicates that a 40-foot opening would adequately serve typical vessel traffic, the final configuration may consider a wider opening, potentially up to 90 feet, to balance navigation requirements, hydraulic performance, and structural considerations.

4.3.2.1 Gate Scour and Velocity

The proposed gate opening was evaluated to assess local scour potential during gate opening operations under differential head conditions.

Local scour is a concern during gate opening under differential head, when water level differences between Grand Lagoon and Lake Pontchartrain generate a high-velocity jet beneath the gate. This condition is most likely immediately following a flood event, when the system is being drawn down and the gate is transitioning open while a head difference still exists.

The preliminary gate opening size was evaluated for scour potential using an empirical USACE relationship for submerged jet scour with conservative assumptions for head difference and tailwater level. Based on these assumptions, a preliminary maximum local scour depth on the order of approximately 2 ft was estimated for a 1-ft head differential. Scour protection can be designed to accommodate these conditions or more severe emergency operations under higher head conditions, if required.

The addition of the proposed gate structure connecting Grand Lagoon and Lake Pontchartrain will modify hydraulic conditions at the structure by constricting the canal cross-section at the gate opening. A detailed hydraulic analysis will be performed during the next phase of design to determine flow velocities through the gate and confirm that navigation and drainage functions can be safely accommodated.

4.3.3 Receiving Structure Design

The conceptual gate receiving structure design is based on similar installations in South Louisiana and consists of a vertical sheet pile wall supported by battered H-piles connected by a continuous waler. The battered pile-waler system provides lateral support to the sheet pile wall by resisting earth and hydrostatic loads and transferring forces to deeper foundation elements. The waler is located 2 feet below the top of the wall where there are no culverts and 1 foot below culverts where they do exist, as shown in **Figure 33**.

CWALSHT analyses were performed to evaluate sheet pile sizing and embedment depth. Based on these analyses, a preliminary sheet pile section of PZ35 with an embedment depth to elevation −43 ft was selected. Batter piles and walers have been preliminarily sized as HP 12×74 and W12 sections, respectively. CWALSHT analyses and member sizing are provided in **Appendix G**. All structural details are at a feasibility level and should be refined during subsequent design phases.

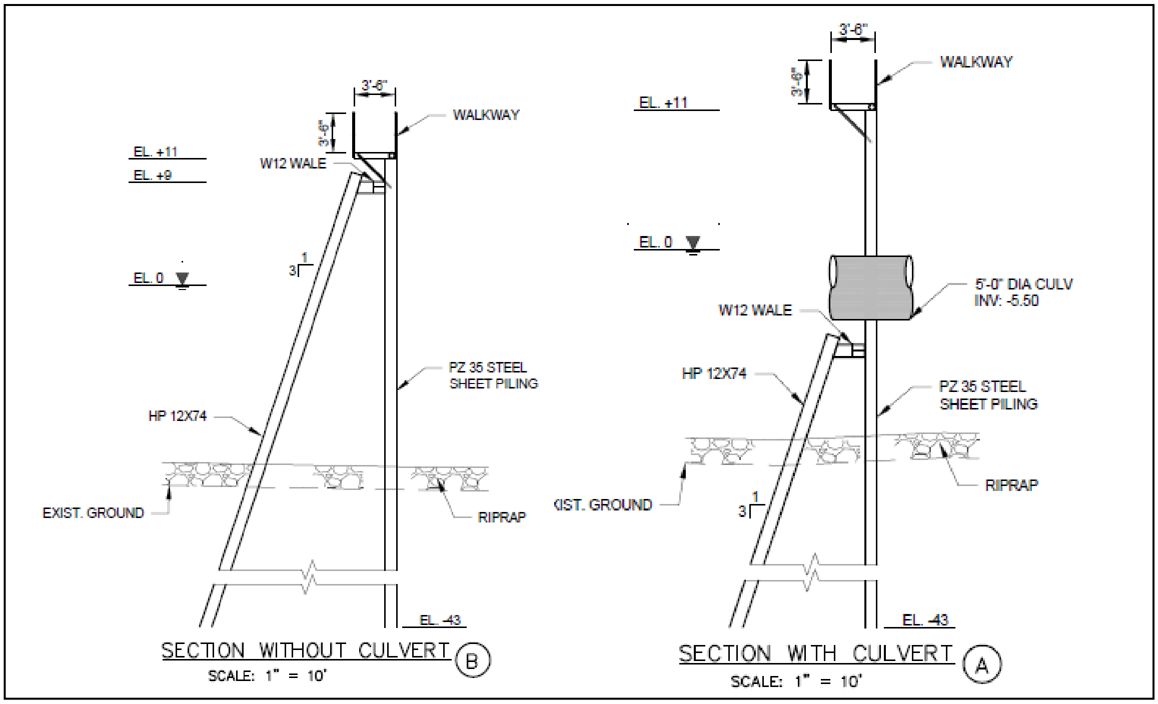

Figure 33: Grand Lagoon Closure

4.3.4 Floodgate Fender System

The conceptual fender system for the Grand Lagoon floodgate is based on guidance consolidated from the following references:

1. *EM 110-2-2502 Floodwalls & Other Hydraulic Retaining Walls*
2. *EM 110-2-1604 Hydraulic Design of Navigation Locks*
3. *EM 110-2-1613 Hydraulic Design of Deep-Draft Navigation Projects*
4. *EM 1110-2-2610 Mechanical and Electrical Design for Lock and Dam Operating Equipment*
5. *EM 1110-2-3402 Barge Impact Forces for Hydraulic Structures*
6. *EM 110-2-2602 Planning and Design of Navigation Locks*
7. *UFC 4-152-01 Piers and Wharves*
8. *EM 1110-2-1611 Hydraulic Design to Shallow Draft Waterways*

EM 1110-2-2602 and *EM 1110-2-1611* provides the most pertinent guidance. These documents emphasize that the longer side of the approach wall is typically to block the predominant wind direction, and the length is usually the usable length of the lock chamber. For this project, the layout of the walls was based on the prevailing southwest to northeast wind direction, and the "usable length of lock" is interpreted as the longest length of vessel that anticipated to pass through the proposed gate.

The shorter approach wall is positioned to prevent contact with the barge gate and placed parallel with the opposite approach wall to guide vessel passage. Because most traffic through the pass consists of small recreational fishing boats, a continuous fender system was selected instead of spaced dolphins to prevent small craft from entering gaps between structures. Dolphins are provided at the upstream and downstream end of the gate opening, spaced at 10 feet and angled at 45 degrees to allow mooring and to guide boats through the navigation channel.

A conceptual layout of the Grand Lagoon Floodgate Complex is shown in **Figure 34**.

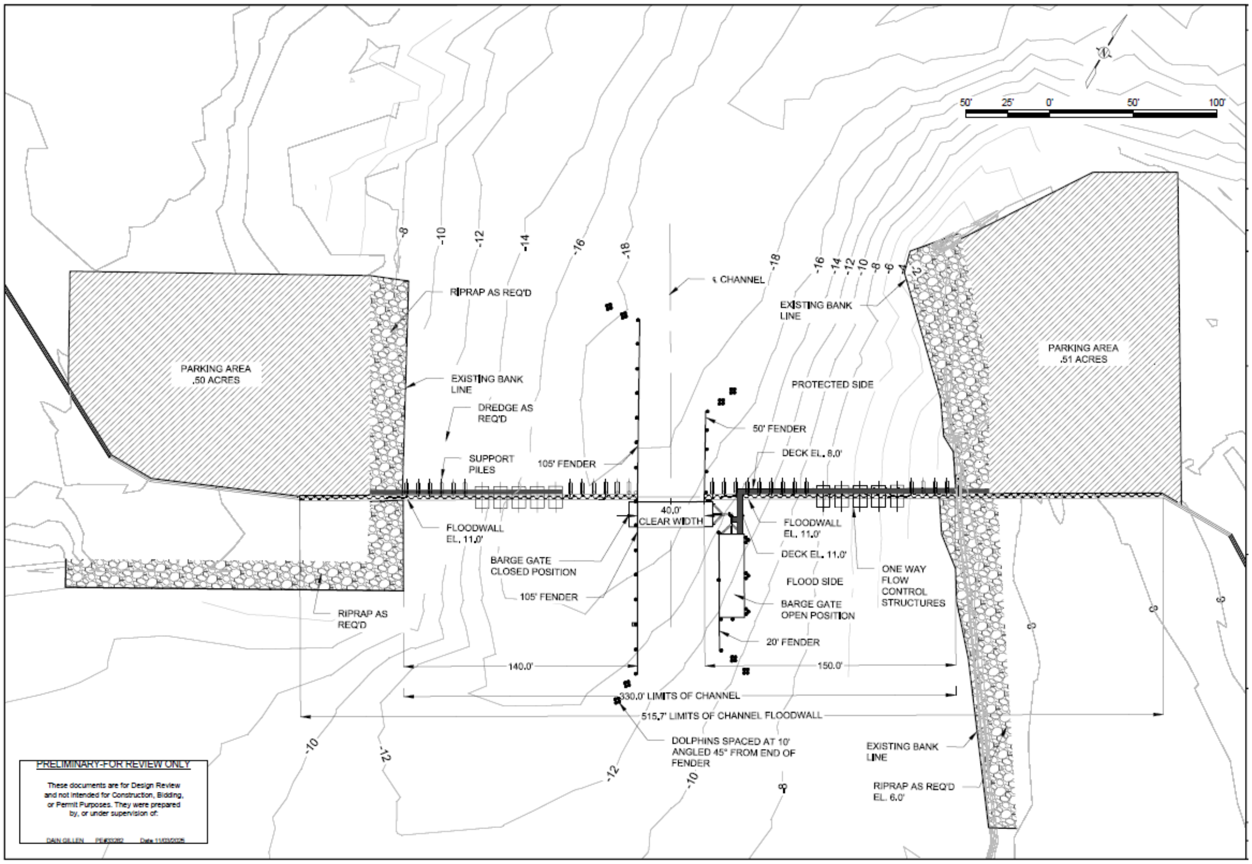

Figure 34: Conceptual Layout of Grand Lagoon Floodgate Complex

4.3.5 Gate Cost

Construction cost estimates for the gate complex were developed using both 40-foot and 90-foot gate opening widths to provide a possible range of cost. Estimates are based on preliminary layout, assumed foundation conditions, and anticipated major structural and mechanical components. These estimates, shown in **Table 24**, are intended to support project planning and funding discussions. Note that costs have been escalated to a future year (2029), to represent the time at which construction could begin, and include a 35% contingency. Detailed estimates are included in **Appendix H.**

Table 24: Barge Gate Construction Cost

Grand Bayou Barge Gate Complex Estimated Construction Cost		
	40-foot gate	**90-foot gate**
Real Estate / Utility Costs	$2,800,000	$2,800,000
Construction Costs	$33,100,000	$40,800,000
Total:	**$35,900,000**	**$43,600,000**

4.3.6 Operations and Maintenance Considerations

Typical O&M activities for barge gates include the following:

- Inspections (Annual)

- Monitoring Evaluation and Report (Every 10 yrs and/or after major storm events)
- Electrical (Monthly cost)
- Painting and Corrosion Protection (Annual, or as required)
- Mechanical maintenance (lubrication, filters, hydraulics)
- Generator Service and Fuel
- Repairs and replacement of components as required (compressors, sensors, pumps, etc.)
- Structural repairs as necessary
- Dry dock and blast and paint gate (Every 10 yrs)
- Personnel to operate and maintain gate

Planning-level O&M costs for the gate were estimated at approximately 0.6% of construction costs based on conversations with local levee districts and O&M to construction cost relationships from previous floodgate projects in south Louisiana and Texas. These costs are shown in **Table 25** and provide a general representation of anticipated long-term maintenance considerations for each alternative.

Table 25: Grand Lagoon Estimated O&M Costs

Grand Lagoon O&M Costs	
Gate Alternative	**Cost**
40’ Gate	$200,000 - $250,000
90’ Gate	$250,000 - $300,000

O&M costs are presented in 2031 dollars to represent costs in Year 0 following construction completion and would be subject to cost escalation over the project life.

5 Hydraulic and Hydrologic Modeling

H&H modeling was performed to evaluate the drainage needs of the Eden Isles community when the project features are fully implemented, and the Grand Lagoon gate is closed. Under the scenario of an approaching or active tropical event and gate closed, rainfall will not be able to leave the system without drainage features to facilitate it. A two-dimensional (2D) H&H model was developed using the USACE River Analysis System (HEC-RAS) software, version 6.6. The 2D HEC-RAS model evaluated the effectiveness of several different types of drainage features at managing interior water surface elevations of the community. The following sections provide an overview and summary of the modeling. Full details are included as **Appendix I.**

5.1 Model Domain

To develop modeled water surface elevation maps, a computational mesh was generated using the topographic and bathymetric data for hydrodynamic analysis. This mesh, shown in **Figure 35**, measures approximately 1.9 miles (3,000 m) in the east-west direction and 2.3 miles (3,700 m) in the north-south direction. The model domain was digitized using an unstructured, flexible mesh consisting of over 61,000 mesh elements (cells), with a base mesh size of 100 ft. The mesh was further refined along roads, channels, and elevated terrain features using break lines to more accurately capture terrain features and flow dynamics.

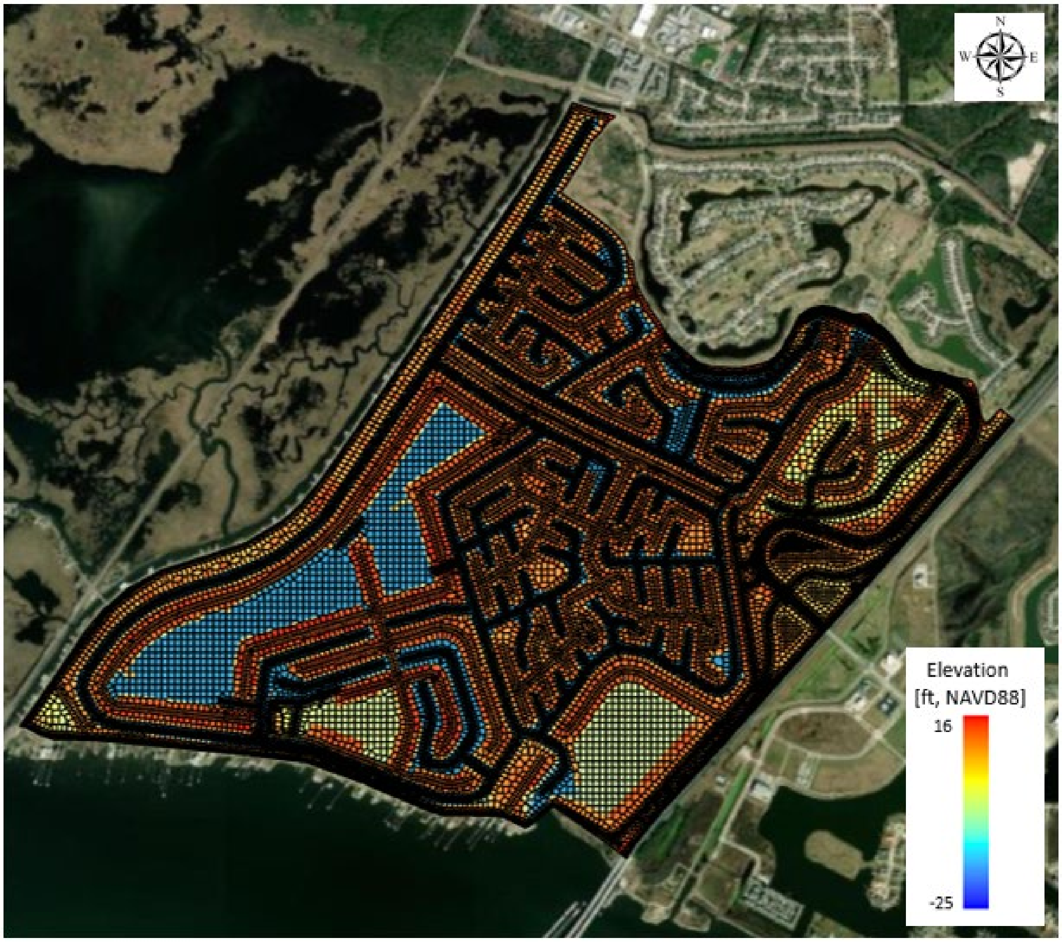

Figure 35: Computational Domain

5.2 BOUNDARY CONDITIONS

Boundary conditions for the modeled scenarios include rainfall on the interior of the system and tidal fluctuations on the exterior of the system. The rainfall intensities are in **Table 2**. The rainfall intensity values used varied according to the recurrence interval being evaluated in each simulation. Scenarios considering both tropical storm surge and rainfall events, were considered joint probability events. For these events, the 10-yr, 24-hr storm was used to determine and size drainage features, based on direction from CPRA and HSDRRS guidance. For scenarios with storm events as the tidal boundary condition, two synthetic storm hydrographs generated by TWI were used as the boundary conditions. These hydrographs are shown below as **Figure 36.** The hydrograph varied by model scenario run. Additionally, depending on what was being analyzed in the specific runs, the hydrographs were scaled up or down accordingly. A simple tidal signal was produced based on the NOAA New Canal gauge values in **Table 1** and used as normal water level once storm surge hydrographs receded.

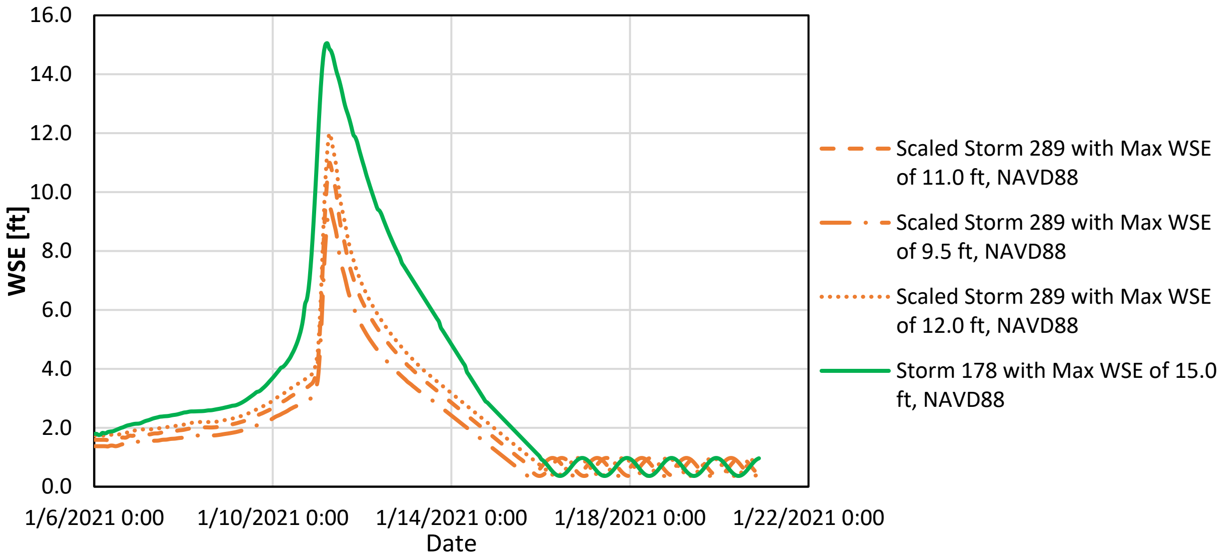

Figure 36: Storm Surge Hydrographs Used for Boundary Conditions

5.3 Simulations

A series of hydraulic model simulations were conducted to evaluate the effects of the proposed project features on interior water levels. Drainage configurations tested included various combinations of 5-foot-diameter culverts with one-way flow control and pump stations of different capacities to accommodate interior flows. For the culvert scenarios, the one-way feature allows normal drainage out of the system while preventing backflow into the protected area during storm surge events or elevated water levels in Lake Pontchartrain. **Table 26** summarizes each simulation, its objective, boundary conditions, and the associated proposed drainage features.

Table 26: Model Simulations Completed to Evaluate System Parameters

Goal	Run	Downstream Boundary Conditions	Rainfall	Proposed Features
Evaluate Baseline Condition	1	Normal Tidal	10-yr, 24-hr	None
Evaluate System Capacity	2	Normal Tidal	10-yr, 25-yr, 50-yr, 100-yr, 24-hr	Full Protection System to +11.0 ft, Gate Closed, no culverts or pumps
Recommendation for proposed system	3	Surge: +11.0 ft	10-yr, 24-hr	Top of Wall at +11.0 ft, 6 culverts
	4	Surge: +11.0 ft	10-yr, 24-hr	Top of Wall at +11.0 ft, 10 culverts
	5	Surge: +11.0 ft	10-yr, 24-hr	Top of Wall at +11.0 ft, 10 culverts + 220 cfs pump (pump on at +6.0 ft surge)
	6	Surge: +11.0 ft	10-yr, 24-hr	Top of Wall at +11.0 ft, 10 culverts + 440cfs pump (pump on/off at +6.0 ft surge)
Evaluate Impacts from a Minor Overtopping Event After Phase 1 Construction / Prior to Phase 2	7	Surge: +9.5 ft	10-yr, 24-hr	Top of Wall at +8.5 ft, Recommended System based on Runs 1-4
Evaluate Impacts from a Minor Overtopping Event After Phase 2	8	Surge: +12.0 ft	10-yr, 24-hr	Top of Wall at +11.0 ft, Recommended System based on Runs 1-4
Evaluate Impacts from a Major Overtopping Event	9	Surge: +14.4	10-yr, 24-hr	Top of Wall at +11.0 ft, Recommended System based on Runs 1-4
Resiliency Check for Higher Rainfall Event	10	Surge: +11.0 ft	25-yr, 24-hr	Top of Wall at +11.0 ft, Recommended System based on Runs 1-4
	11	Surge: +11.0 ft	50-yr, 24-hr	Top of Wall at +11.0 ft, Recommended System based on Runs 1-4
	12	Surge: +11.0 ft	100-yr, 24-hr	Top of Wall at +11.0 ft, Recommended System based on Runs 1-4

5.4 Results

The results of the two-dimensional (2D) HEC-RAS modeling provide a detailed assessment of the hydraulic performance of the proposed Eden Isles drainage system under combined rainfall and storm-surge conditions. Multiple configurations were evaluated to determine the most efficient and cost-effective drainage system capable of maintaining acceptable interior water surface elevations during both non-overtopping and overtopping events.

5.4.1 Baseline Conditions and System Capacity

The first runs completed were to assess baseline performance of the system.

Run 1 was completed to determine impacts to the neighborhood under existing conditions during the design event (10-year, 24-hour rainfall combined with a +11.0 ft non-overtopping storm surge).

Run 2 was completed to determine the increase in interior water levels with the flood protection system fully implemented to +11.0 ft with the Grand Lagoon gate closed and no drainage features. Under the design event, interior water surface elevations peaked at approximately +2.3 ft with the floodgate closed and no proposed drainage system, as shown in **Figure 37**.

5.4.2 Evaluation of Proposed Systems

Runs 3 and 4 were completed to show the difference in performance of six or ten 5-ft-diameter culverts. These runs indicated very similar results, with the ten-culvert configuration providing slightly faster drawdown with the receding storm surge hydrograph. Once exterior surge elevations receded enough to create a positive head differential, the culverts effectively drained the system to near-normal tidal levels, confirming that a culvert-only configuration is hydraulically sufficient for the design event. Although the performance of these was similar, the ten-culvert configuration was used in subsequent runs due to its ability to provide system redundancy.

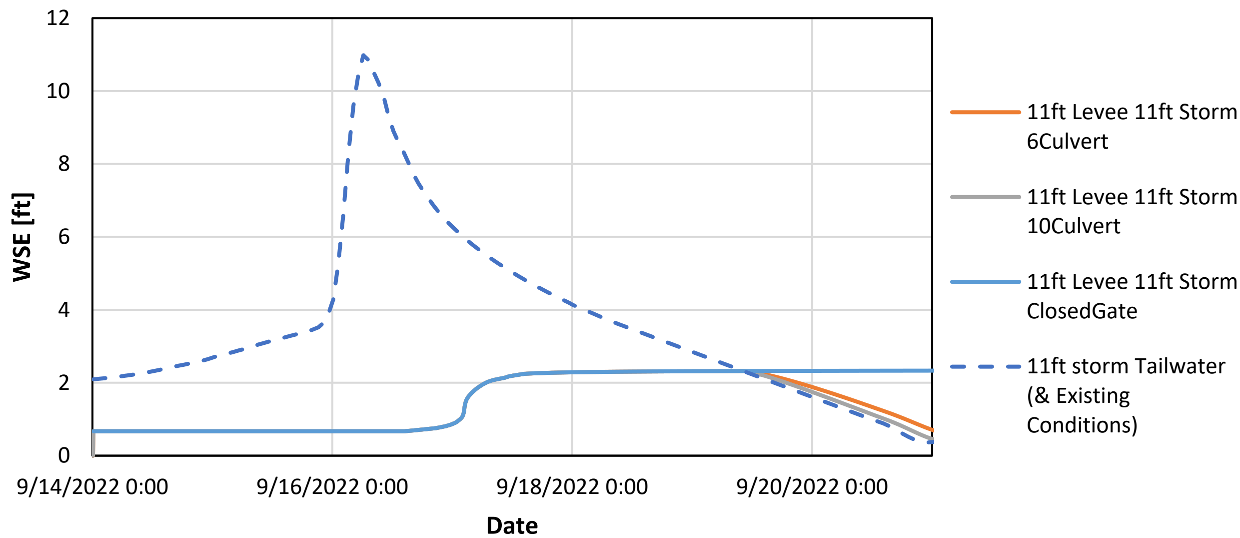

Figure 37: Closed Gate and Culvert-Only Results

Runs 5 and 6 evaluated the addition of 220 cubic feet per second (cfs) and 440 cfs pump stations, respectively, to the ten-culvert configuration. With the addition of the pump stations, drawdown times improved but the reduction in peak water surface elevation was modest. The 220 cfs configuration lowered the maximum interior water surface elevation to approximately +1.97 ft and achieved the target elevation of +1.67 ft in roughly 23 hours. The 440 cfs configuration achieved a similar peak of +1.84 ft with drainage time reduced to approximately 12 hours. While pump stations expedite drawdown following a storm, their effect on reducing peak flooding during non-overtopping events is limited. Results for runs 5 & 6 are shown in **Figure 38**, along with the closed gate and culvert-only runs for comparison. Based on these results, the ten-culvert option with no pump station was selected as the Proposed System for subsequent resiliency check runs.

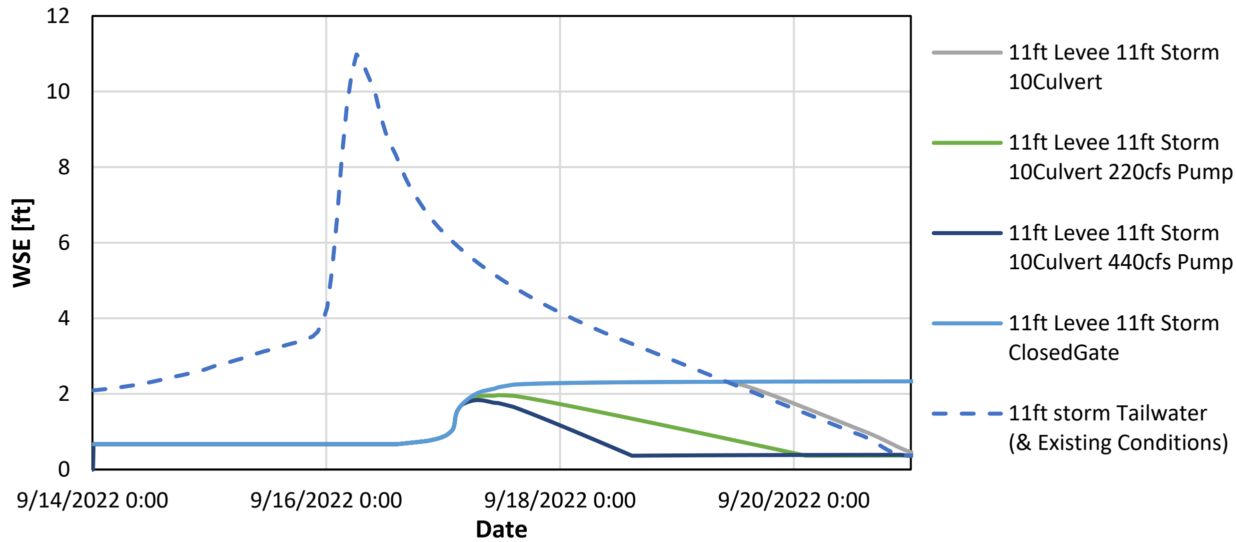

Figure 38: Pump Station + Culvert Runs

5.4.3 Resiliency Check for Overtopping Scenarios

Runs 7, 8, and 9 evaluated performance of the Proposed System under various overtopping scenarios. These analyses assessed the performance of the ten-culvert configuration under minor and major surge events.

- Run 7 analyzed a minor overtopping event following a first phase of construction that brings the system elevation to +8.5 ft, but prior to a full buildout to +11.0 ft. The storm surge boundary condition was scaled up to peak at 1 ft above the level of protection. Under this scenario, the peak interior water surface elevation reached +8.3 ft. Results are shown in **Figure 39.**
- Run 8 analyzed a minor overtopping event following the full buildout to +11.0 ft. The storm surge boundary condition was scaled up to peak at 1 ft above the level of protection. Under this scenario, the peak interior water surface elevation reached +9.7 ft. Results are shown in **Figure 40.**
- Run 9 analyzed a major overtopping event following full buildout to +11.0 ft. The maximum Water Institute storm surge boundary condition of +15.0 ft was used. Under this scenario, the peak interior water surface elevation reached +14.8 ft. Results are shown in **Figure 41.**

In all overtopping cases, the culverts initiated drainage once the surge receded below interior levels, progressively returning water levels to normal conditions.

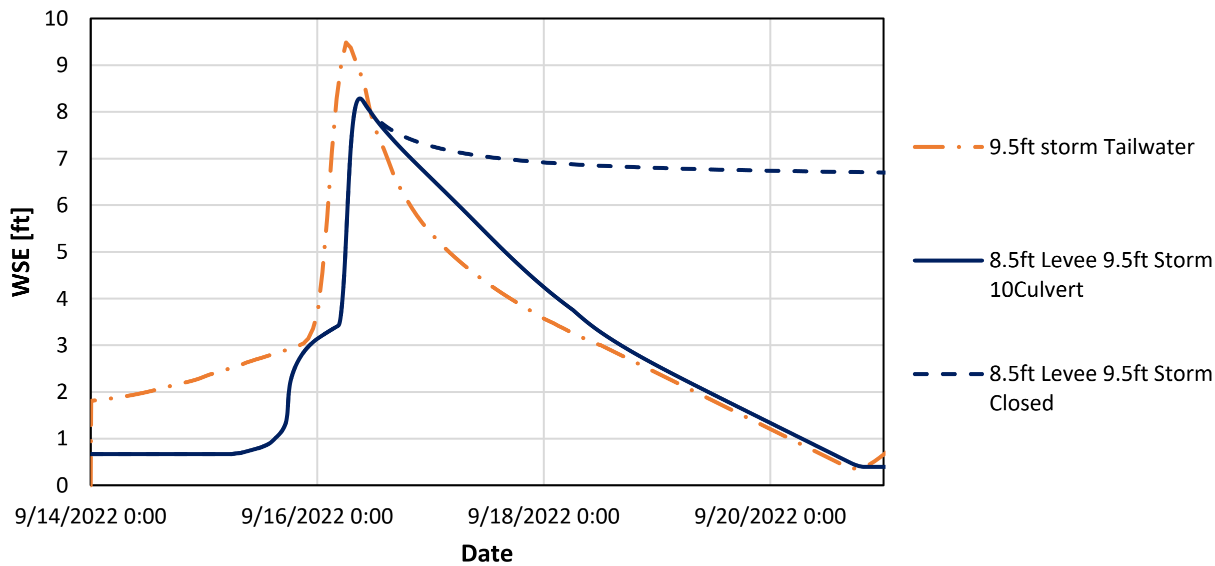

Figure 39: Phase I Minor Overtopping Event

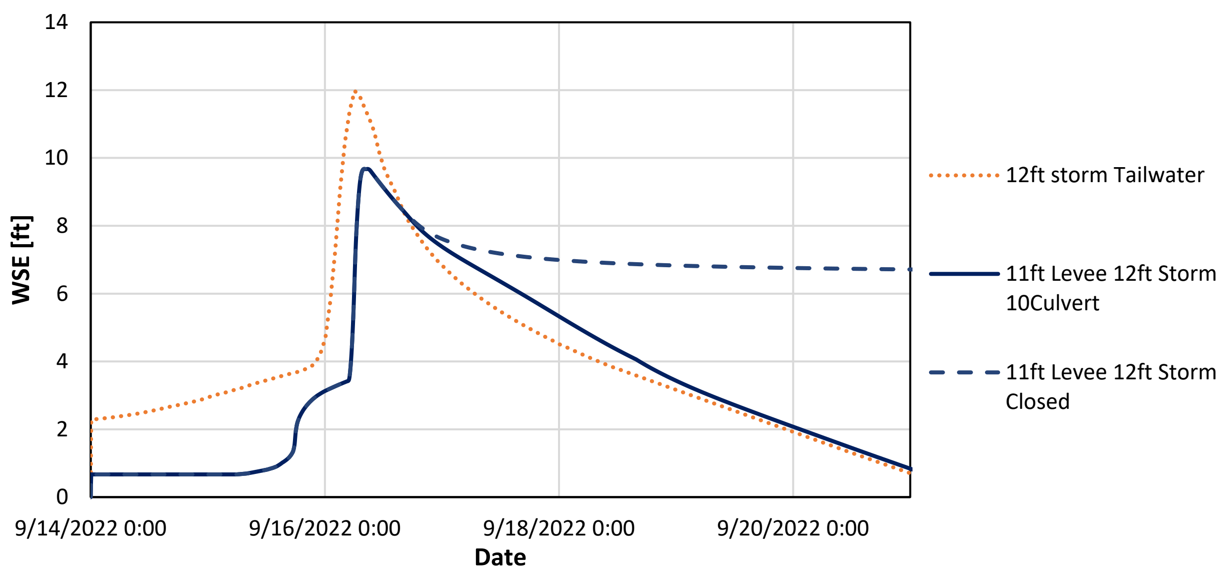

Figure 40: Phase II Minor Overtopping Event

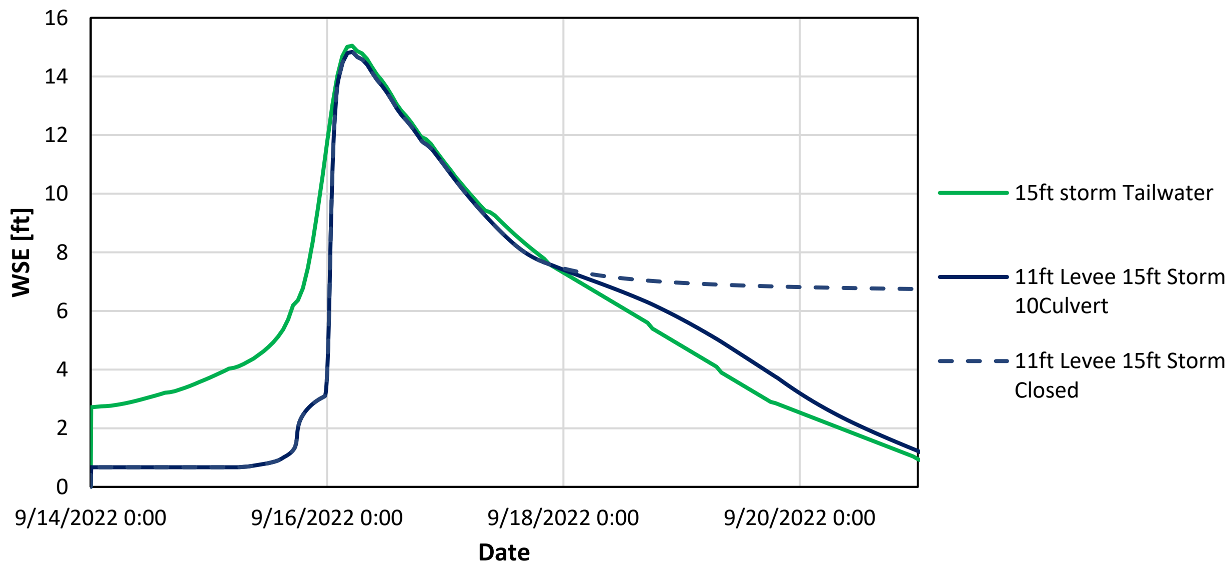

Figure 41: Major Overtopping Event

5.4.4 Resiliency Checks for Higher Precipitation Events

Runs 10, 11, and 12 were additional simulations for 25-, 50-, and 100-year rainfall events under non-overtopping surge conditions and confirmed consistent performance of the culvert system. As expected, both peak elevation and duration above the target level increased with rainfall intensity, with interior water elevations increasing up to +3.5 ft during a 100-year rainfall event. In all cases, the culverts continued to perform effectively once tailwater recedes, demonstrating resiliency across a range of conditions. Results are shown in **Figure 42** and **Table 27**.

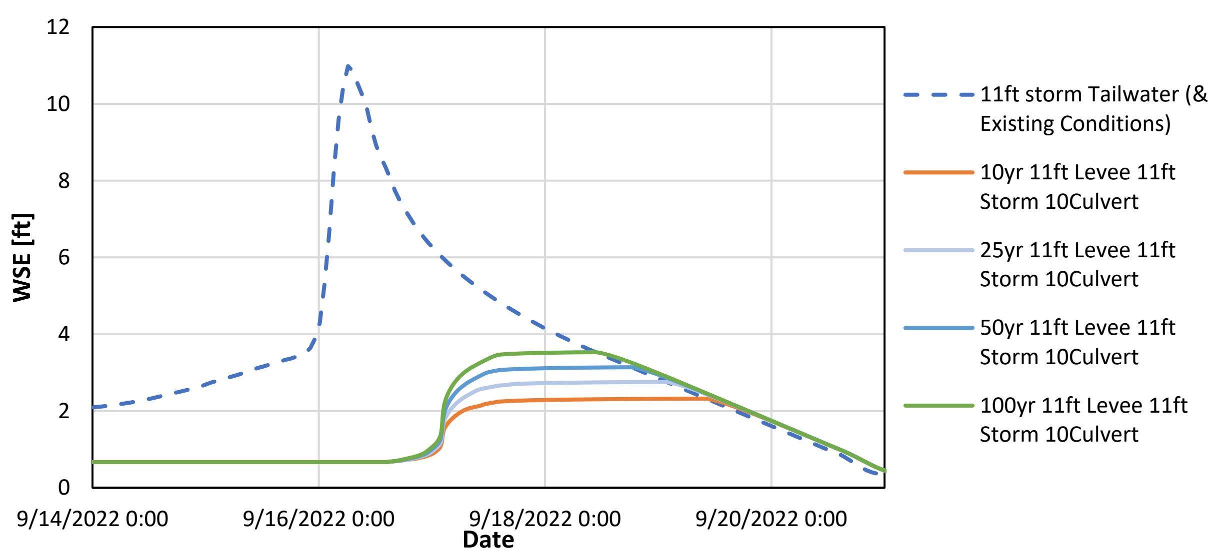

Figure 42: System Resiliency Checks for Higher Rainfall Events

Table 27: Maximum Water Surface Elevation for Resiliency Checks

Simulation	Maximum WSE (ft, NAVD88)
10-yr Rainfall	2.32
25-yr Rainfall	2.76
50-yr Rainfall	3.14
100-yr Rainfall	3.53

5.5 H&H Modeling Summary

The existing Eden Isles system has significant interior storage capacity based on the completed modeling scenarios but leaves the neighborhood vulnerable from tropical storm events accompanied by storm surge. Based on the results of the 2D H&H modeling, the proposed flood protection and interior drainage system for Eden Isles provides substantial reduction in flood risk for storm events with surge levels up to +11.0 ft and provides suitable drainage following storms that do overtop the system.

The analysis confirmed that the combination of a +11.0 ft levee, a closed floodgate at Grand Lagoon, and a culvert system of ten 5-ft-diameter barrels with one-way flow control is capable of maintaining interior water levels below approximately +2.3 ft during the design event, preventing significant interior flooding.

Once exterior storm surge elevations subside, the culvert system allows efficient drainage to near-normal tidal conditions in a timely manner without reliance on pumps. The inclusion of pump stations (220 or 440 cfs total capacity) provides only marginal reductions in peak flood elevation but accelerates post-storm drawdown. Given that drawdown time is not critical for the design event and considering the added cost and complexity, pump stations are not required but may be included to enhance operational flexibility and system recovery. In accordance with guidance from *The Water Institute of the Gulf (TWI, 2024)*, pump sizing for the Eden Isles system should focus on pluvial (rainfall-driven) flooding rather than overtopping events, since overtopping rates exceed what can be feasibly pumped from the system.

Modeling of the overtopping scenarios demonstrated that the proposed culvert system performs effectively following surge overtopping, allowing water to recede once tailwater elevations fall. System performance with culverts is passive and does not rely on human intervention or electrical systems during post-storm conditions.

Resiliency checks for higher-intensity rainfall events (25-, 50-, and 100-year storms) confirmed consistent performance with predictable, proportional increases in ponding and duration above target levels.

In summary, modeling results confirm that a culvert-only configuration with ten 5-ft-diameter barrels and one-way flow control provides suitable interior drainage for non-overtopping conditions. Pump stations are optional enhancements that improve drawdown time but are not essential to achieving the design objectives for flood protection and community resiliency.

6 Project Alternative Summary

This report presents the evaluation of drainage, access, and flood protection alternatives for the Eden Isles community as part of the PO-211 Eden Isles Resilience and Flood Risk Reduction Project. The goal of the study was to identify feasible combinations of structural and hydraulic measures that collectively reduce flood risk while maintaining neighborhood access and drainage function during tropical events.

6.1 Lakeview Drive Alternatives

Four alternatives were developed to provide flood protection along Lakeview Drive to an ultimate design elevation of +11.0 ft, each balancing flood protection, access, and constructability in different ways.

- Alternative A consists of raising Lakeview Drive to +8.5 ft in the first phase and later installing a centerline floodwall (either a sheet-pile foundation or PCC wall and roadway section) to reach the ultimate design elevation. This option provides substantial near-term risk reduction and stays largely within the existing right-of-way. However, it introduces operational challenges: the wall would divide the corridor and restrict cross-street movements, requiring directional U-turn openings spaced approximately every ¼ mile. Openings would not accommodate larger vehicles or longer trailers. Driveway reconstruction and grading adjustments would be extensive. The sheet-pile option is the more expensive of these two. The reinforced concrete version tied into a PCC roadway slab would reduce costs but still entail significant construction impacts.

- Alternative B follows a similar two-phase concept but raises the roadway to +6.0 ft initially, leaving a lower first-phase profile and constructing the centerline floodwall in a later phase to achieve +11.0 ft. The wider cross-section would allow two lanes on each side of the wall, maintaining continuous access to driveways and reducing the need for directional U-turn openings. This option lessens construction-phase disruption and improves long-term traffic circulation, but the interim embankment provides only modest protection benefits. When the wall is fully built out it may cause aesthetic concerns and be a visual barrier for drivers with its 5 ft height from existing ground. Overall cost for Alternative B is higher than Alternative A.

- Alternative C raises the roadway to +8.0 ft and installs self-deploying passive flood barriers (such as FloodBreak) along straight roadway segments, with permanent floodwalls used on curved or transitional sections (~25% of total length). The barriers remain retracted during normal conditions, preserving open sight lines and driveway access, and rise automatically during flood events. This alternative offers operational simplicity and minimal daily obstruction but carries a substantially higher cost and limited geometric applicability since it cannot be installed along curves and requires slightly larger median widths than the other alternatives.

- Alternative D consists of constructing a levee to a settled elevation of +11.0 ft north of Lakeview Drive. This alternative provides a conventional levee configuration that minimizes roadway operational constraints; however, it would require acquisition of all properties and have major impact to existing infrastructure north of Lakeview Drive to accommodate the levee footprint, side slopes, and construction access. As a result, while Alternative D is technically feasible, it presents significant real estate and stakeholder coordination considerations.

Table 28 provides a summary of considerations for each Alternative.

Table 28: Summary of Considerations for Lakeview Drive Alternative

Alternative	Description	Advantages	Disadvantages	Cost	O&M
Alternative A: 2-Lane Road	**+8.5' Road Raise, +11' Floodwall**	Cost • Most cost-effective option Constructability • Phased Implementation possible	Vehicle Access • Limits ability of emergency vehicle passage if lane is blocked • Traffic congestion due to lane blockage (garbage truck, delivery vehicles, etc.) • Larger vehicles cannot use turn around • Construction sequencing may limit to 1 lane • Potential for greater impacts to traffic during construction • Damage impacts to wall from vehicles Resident Impacts • Limited ability to back up boat/RV in driveway • Landowners may be required to store boat/RV elsewhere • Steeper incline/decline for entering/exiting driveways • Moderate impacts to lake views (2.5' tall, +11.0' elev. wall) Utilities • Extensive Utility Relocations if sheetpile foundation option	$43M - $54M	• Repairing cracking and spalling of concrete floodwalls • Rehab or replacement of exposed metal or connections
Alternative B: 4-Lane Road	**+6.0' Road Raise, +11' Floodwall**	Vehicle Maneuverability • U-turns not required to access all homes • More available roadway to back in boat/RV • Ability to maneuver around stopped vehicles blocking a lane.	Resident Impacts • Expansion of roadway to areas within the ROW currently being utilized by residents • Landowners may be required to store boat/RV elsewhere • Significant impacts to lake views (5.0' tall, +11.0' elev. wall) Utilities • Extensive Utility Relocations if sheetpile foundation option Constructability • Phased implementation possible, but minimal protection prior to floodwall construction.	$63M - $69M	• Repairing cracking and spalling of concrete floodwalls • Rehab or replacement of exposed metal or connections
Alternative C: Passive Flood Barriers	**+8.0' Road Raise, +11' Floodwall**	Functionality • Deploys/Retracts automatically • Protects lakefront view • Barriers can be traffic rated for driving over • Allows turnarounds and cross-traffic turns for majority of alignment Constructability • Phased implementation possible	Cost • Very Expensive Functionality • Dependent on proper deployment for full length • Will still need floodwall in sections (not continuous) • Possible partial deployment from floodside drain • Vehicles or other obstructions on the barriers could prevent proper deployment Resident Impacts • Steeper incline/decline for entering/exiting driveways • Current configuration impacts driveways Utilities • Moderate Utility Relocations	+$143M	• Annual cleanout • Rehab / Replacement of gaskets, seals, hydraulic arms, structural components etc.
Alternative D: Levee	**+11' Levee on north of Lakeview Drive**	Constructability • Limited mainly to earthwork • Existing roadway not affected Utilities • Limited utility relocations Cost • Lowest construction-only cost	Cost • High real estate cost for acquiring lots/homes • Additional cost for demolishing homes, slabs, driveways Resident Impacts • Acquisition and demolition of all structures north of Lakeview Drive Constructability • Phased implementation not practical	$58-59M	• ROW Maintenance / Mowing • Addition of material to maintain levee elevation

While all concepts achieve the desired protection level, each involves trade-offs in cost, access, property impact, constructability, and long-term maintenance. The final selection of a preferred alternative for Lakeview Drive should consider not only the factors discussed here, but also public input and acceptance, as well as the ability of local agencies to operate and maintain the completed system. It should also be noted that the alternatives evaluated were developed for the purpose of this feasibility analysis, and a refined alternative may be developed during subsequent phases of design that incorporates elements of several alternatives to address detailed site-specific constraints.

6.2 Grand Lagoon Gate Complex

For the Grand Lagoon structure, a barge-gate complex is recommended for advancement into preliminary design. The gate receiving structure consists of a vertical sheet pile wall supported by battered H-piles and a waler system, consistent with similar installations in South Louisiana. A conceptual layout of the gate complex has been developed. System parameters have been selected and sized in accordance with accepted navigation engineering design documents and guidelines, and preliminary geotechnical and structural analyses performed. The barge gate provides operational flexibility and can remain open during normal tidal conditions, closing in advance of storm surge events to prevent backwater flooding into Eden Isles.

The conceptual layout includes a minimum gate opening size of 40 ft and sill elevation of -10 ft NAVD88, which were identified based on representative vessel dimensions within Grand Lagoon and reference elevations from comparable barge-gate installations in south Louisiana. These parameters should be considered preliminary and will be refined during subsequent phases of design as hydraulic performance, navigation requirements, and structural considerations are evaluated in greater detail.

Hydrologic and hydraulic (H&H) modeling was completed to inform selection of drainage features to be utilized during gate closure. The 2D HEC-RAS model results indicate that a system of ten 5-ft-diameter culverts with one-way flow control is sufficient to manage interior rainfall runoff during non-overtopping events. Peak interior water surface elevations remain below +2.3 ft during a 10-year, 24-hour rainfall event, which does not result in significant interior flooding. Pump stations were evaluated as an enhancement to improve post-storm drawdown time; a 220 cfs or 440 cfs pump can reduce drainage time, but pumps are not required for the design-level event. CPRA may consider pump installation to enhance operational flexibility.

6.3 Harbor Drive Alternatives

For Harbor Drive, several design alternatives were evaluated to achieve the desired level of protection. Constraints were less than Lakeview Drive since all the lots are vacant along the road. Alternatives included a phased lift of Harbor Drive with embankment and permanent floodwall similar to Lakeview Drive, a single embankment lift of Harbor Drive to the design elevation of +11.0 ft, embankment adjacent to the road, embankment south of the road in the middle of the vacant properties, and replacement of the bulkhead along the lakefront.

Based on cost, constructability, and impacts to adjacent residential properties, raising Harbor Drive with embankment fill appears to be the most practical and cost-effective alternative. This approach integrates well with the overall system, allows straightforward tie-ins to adjacent grades, and minimizes the need for deep foundations or major utility relocations. A one-way drainage structure has been incorporated near the southeastern extent of the project to facilitate positive drainage to Lake Pontchartrain from the area adjacent to Interstate 10 and Harbor Drive.

6.4 Other Design Considerations

Survey and geotechnical investigations were completed to support feasibility. Detailed utility information was collected, but a complete subsurface utility investigation (SUE) will be required to determine utility depths, sizes, etc. prior to detailed design. If Alternative D is advanced, additional topographic survey data collection will be necessary to facilitate design.

Preliminary soil data indicates subsurface conditions generally consistent with other south Louisiana coastal levee projects, with soft to medium clays underlain by denser material at depth. However, some variability in soil borings were noted along Lakeview Drive. Namely, boring B-1 showed weaker soil strengths than nearby borings previously collected by Eustis Engineers. Also, some variability in borings was observed in Grand Lagoon. Further discussion may be warranted prior to detailed design to determine if additional information may be necessary.

Data collection activities to support environmental and permitting are expected to be initiated during 30% design, or when specific project alternatives are selected and finalized for more detailed design.

Coordination with local stakeholders will be completed in subsequent design phases. Public engagement will be critical to ensure homeowner understanding and acceptance of the selected alternatives.

6.5 Cost

A range of construction cost estimates was developed for each major component of the proposed Eden Isles flood protection system's southern boundary to capture variability in design options. All costs include a 35 percent contingency for construction items, land acquisition costs, and utility relocation costs. Costs are presented in future (2029) costs, representing a future timeframe when project construction could occur.

Four alternatives were evaluated for Lakeview Drive, varying by embankment elevation, protection type, and level of automation. The lowest-cost configuration, a two-phase embankment and Portland Cement Concrete (PCC) floodwall (Alternative A), is estimated at $43 million, followed by the levee alternative (Alternative D) at approximately $58 million. The highest-cost configuration incorporating self-deploying passive flood barriers (Alternative C) exceeds $143 million. Key cost drivers include embankment volume, floodwall and foundation type, real estate acquisition costs, and construction phasing requirements.

Two floodgate concepts were evaluated: a barge-gate and a sector-gate. Based on cost, constructability, and operational considerations, the barge gate was selected for advancement. A planning level analysis estimated a barge-gate option approximately 1/3 the cost of a sector gate, making the barge gate the preferred option. After selection of the barge gate, a detailed cost analysis was prepared for the floodgate complex with costs ranging from $36 - $44 million depending on the gate size. The barge-gate configuration provides a balanced combination of functionality and cost efficiency for the scale of the Eden Isles project.

Several alternatives were assessed for Harbor Drive, ranging from a simple embankment lift to full bulkhead replacement along the lakefront. The lowest-cost option, a levee adjacent to the roadway, totals approximately $1.3 million, while the highest-cost alternative, replacement of the existing bulkhead, exceeds $19 million. Intermediate options such as phased road elevation or mid-parcel levee alignments fall within this range. The recommended alternative of a single-phase construction effort to raise the road to +11.0 ft is estimated at $4.0 million. This alternative minimizes impacts outside of the ROW and reduces O&M costs.

Combining all project components yields a total estimated range of approximately $80 to $206 million, as shown in **Table 29**. These costs are presented in future (2029) dollars and include 35% construction contingency, utility relocation costs, and real estate acquisition costs. The lower bound represents a phased program emphasizing earthen embankments, PCC foundation and floodwall, and a barge-gate at Grand Lagoon, while the upper bound reflects maximum-cost structural alternatives and self-deploying barrier systems. As specific project features are selected and the project advances into next phase, design refinements will further narrow these cost ranges.

Table 29: Summary of Total Project Costs

Project Feature	Costs	
	Low	High
Lakeview Drive	$43,300,000	$143,400,000
Grand Lagoon Gate Complex	$35,900,000	$43,600,000
Harbor Drive	$1,300,000	$19,600,000
TOTAL:	**$80,500,000**	**$206,600,000**

Planning-level O&M costs were developed for each alternative to support comparative evaluation. O&M costs were estimated based on the anticipated maintenance requirements of each feature type, including inspection, vegetation management, structural maintenance, and, where applicable, mechanical and operational components. Costs were escalated to 2031 to represent a possible timeframe of construction completion. Total annualized O&M costs are expected to fall in a range of $255,000 to $975,000, depending on the combination of project features and components selected for final design. These costs would be subject to future escalation over the project life.

6.6 Summary

In summary, this alternatives analysis identifies feasible flood protection strategies for the Eden Isles community. Each Lakeview Drive alternative presents unique challenges and involves trade-offs between cost, community impacts, and the ability to implement in phases. The advantages and disadvantages of each alternative should be considered carefully during the selection of a preferred alternative. Note that elements of several alternatives may be considered during detailed design to address site-specific constraints.

The barge-gate structure at Grand Lagoon is recommended as the preferred closure type and will advance into detailed design. Culverts alone are adequate to provide interior drainage under the closed-gate condition, with pumps optional to enhance drawdown times.

At Harbor Drive, an embankment and road lift to +11.0 ft is recommended as the most practical approach based on cost, constructability, O&M requirements, and impacts to adjacent property.

Final selection of project features should be based on an integrated consideration of cost, constructability, environmental and property impacts, public input and acceptance, and long-term operation and maintenance needs. These decisions will guide the design team as the project transitions from conceptual design to preliminary and final engineering.

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Appendix A

DESIGN CRITERIA

Appendix B

SURVEY DATA

Appendix C

GEOTECHNICAL DATA

Appendix D

COASTAL DATA

Appendix E

WIND AND WAVE LOADING CALCULATIONS

Appendix F

PROJECT ALTERNATIVE DRAWINGS

Appendix G

STABILITY ANALYSES

Appendix H

COST ESTIMATES

Appendix I

H&H MODELING