2025 Supplemental Draft Integrated General Reevaluation Report (GRR) and Supplemental Environmental Impact Statement (SEIS) for the Mississippi River, Baton Rouge to the Gulf Of Mexico Mississippi River-Gulf Outlet, Louisiana, New Industrial Canal Lock and Connecting Channels Project (also Referenced as "Inner Harbor Navigational Canal Lock")

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Prepared by USACE-MVN ED

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ABBREVIATIONS AND ACRONYMS

ACRA	Abbreviated Cost Risk Analysis
AASHTO	American Association of State Highways and Transportation Officials
ACE-IT	Army Corps of Engineers Information Technology
ADM	Agency Decision Milestone
A/E	Architect-Engineering Firm
AHP	Above Head of Passes
CDF	Confined Disposal Facility
CECC-R	Corps of Engineers, Office of the Chief Counsel
CSRA	Cost and Schedule Risk Analysis
DMMU	Material Management Units
EC	Engineering Circular
ECB	Engineering and Construction Bulletin
EL	Elevation
ER	Engineering Regulation
EM	Engineering Manual
ERDC	Engineer Research and Development Center
ETL	Engineering Technical Letter
FRR	Flood Risk Reduction
FWOP	Future Without Project
FWP	Future With Project
GIWW	Gulf Intracoastal Waterway
GRR	General Reevaluation Report
IHNC	Inner Harbor Navigational Canal
HQUSACE	Headquarters, U.S. Army Corps of Engineers
HSDRRS	Hurricane and Storm Damage Risk Reduction System
HTRW	Hazardous, Toxic, and Radioactive Waste

HUC	Hydrologic Unit Code
IEPR	Independent External Peer Review
INDC	Inland Navigation Design Center
ITR	Independent Technical Review
LADEQ	Louisiana Department of Environmental Quality
LA-DOTD	Louisiana Department of Transportation and Development
LRD	U.S. Army Corps of Engineers Lakes and River Division
MCACES	Microcomputer Aided Cost Engineering System (MII)
MCX	USACE's Mandatory Center of Expertise
MFR	Memorandum for the Record
MVD	Mississippi Valley Division
MVN	Mississippi Valley Division, New Orleans District
MR&T	Mississippi River and Tributaries
MRL	Mississippi River Levees
MRGO	Mississippi River Gulf Outlet
NAVD88	North American Vertical Datum 1988
NOSWB	New Orleans Sewerage and Water Board
OMRR&R	Operation, Maintenance, Repair, Replacement and Rehabilitation
P&S	Plans and Specifications
PDT	Project Delivery Team
PED	Preconstruction Engineering and Design
RSLC	Relative Sea Level Change
RM	River Mile
RNA	Regulated Navigation Area
RP	Recommended Plan
SEIS	Supplemental Environmental Impact Statement
TPCS	Total Project Cost Summary
TSP	Tentative Selected Plan

USACE	United States Army Corps of Engineers
USCG	US Coast Guard
VE	Value Engineering
WRDA	Water Resources Development Act

1 Executive Summary

This Engineering Appendix contains details of preliminary engineering investigations completed to provide input in support of the economic analysis completed for the lock replacement General Re-evaluation Report (GRR). The 2017 draft GRR looked at several alternatives and selected the tentatively selected plan (TSP). Following the period of public review and comment, the TSP became the Recommended Plan (RP) and underwent further analysis for this final integrated GRR and SEIS. That analysis resulted in some revisions from the TSP, (mainly by removing the temporary St. Claude Avenue Bridge). The primary change from the draft 2017 report TSP and 2019 final report RP is construction of a permanent low-level double bascule bridge north of the existing St. Claude Avenue Bridge, with demolition of the existing bridge when construction of the new bridge and its approaches are complete. The revised plan for the replacement of the St. Claude Bridge will require acquisition of a small number of residential properties. This plan for replacement of the St. Claude Bridge eliminates the need for a temporary by-pass bridge. The 2017 draft report TSP and the 2019 final report RP also eliminate the placement of a light rail on the new permanent bridge and its approaches.

This appendix discusses the development of this final GRR study plans. Preliminary investigations included cost estimates, lock filling times, and recommendations for the chamber sill and wall elevations as described herein. It included cost for the RP, updates to the project design and construction schedules, construction sequence, St. Claude Avenue Bridge design, sector gate versus miter gates comparison, cast-in-place versus float-in-construction comparison, structural design, Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) cost discussion, updates to the relocations, among other items.

2 Project Purpose and Description

The study area is located in Orleans and St. Bernard Parishes in southeastern Louisiana. The study is generally bounded by Lake Pontchartrain on the north, the Mississippi River on the south and west, and Lake Borgne, Breton Sound, and the Gulf of Mexico on the east and south. Areas potentially affected by changes in vessel traffic include the navigation channels and related lands in the study area, the inland waterway system on the Gulf Intracoastal Waterway (GIWW), and the Mississippi River.

This Engineering Appendix has been prepared by the New Orleans District, Engineering Division (CEMVN-ED) staff.

All elevations (EL) in this appendix are referenced to NAVD88 (epoch 2004.65) vertical datum, unless otherwise noted.

The purpose of the project is to replace the aging lock, and in doing so, to improve navigation vessel transit times in a manner that is environmentally acceptable and economically justified, without inducing a negative impact to the existing flood risk reduction and hurricane storm damage risk reduction structures and their respective authorized levels of risk reduction. Although the lock is located in a polder perimeter with other flood risk reduction features, the lock itself is not considered a flood risk reduction structure. The new lock will be constructed north of the existing lock, and north of Claiborne Avenue Bridge as shown in Figure 1 below.

Starting on the southwest side of the project, the new floodwall will tie-in into the existing wall on the west of the existing IHNC Lock. The new floodwall will tie to the new lock on the west side. Starting on the southeast side of the project, the new levee will tie into the existing MRL levee on the east side of the existing IHNC Lock and travel to the new lock tie-in wall on the east side. The new floodwall segments under the St. Claude and North Claiborne Ave bridges will tie to the new levees on the east side.

The proposed lock will be a concrete cast-in-place lock with sector gates, a pile foundation and side port culvert filling and emptying system. The dimensions of the new lock chamber for the recommended plan (RP) will be 900 ft long by 110 ft wide. It will be a shallow draft lock with a sill elevation of (-) 22.0 ft.

A box culvert size of 14.5 ft x14.5 ft was selected for the 900-ft long chamber. In addition to the sector gate and lock, a bypass channel, cofferdam, 3 pile dolphins, timber and floating guidewalls, and concrete floodwalls will be constructed at this location.

Although the lock's authorized purpose is not flood risk reduction, the river side gatebay provides a continuous line of risk reduction between upstream and downstream MRL features. The shift in location of the new lock necessitates adjustment of both LPV and MRL project features to tie into the new lock. Phasing of construction activities and temporary risk reduction measures will be utilized to minimize any impacts on the MRL and LPV Projects with respect to the authorized levels of risk reduction during and after construction of the new lock.

The location of the proposed lock was primarily chosen due to its ease of access and little to no obstructions located near the open channel.

The physical features associated with the construction of the new lock structure are:

- Chamber Concrete Monoliths/Pile Foundation
- Sector Gate Monoliths/Pile Foundation
- Steel Sector Gates
- Timber Guidewalls

- Floating Concrete Guidewalls
- End Cell Dolphins
- Cofferdam
- Floodwalls
- Levee
- Maintenance Bulkheads
- Maintenance Bulkhead Storage Platform
- Culvert Roller Gates
- Culvert Bulkheads
- Permanent Mooring Cells

Other major project features include:

- Replacement of the St. Claude Avenue Bridge
- Construction of a Bypass Navigation Channel near the new lock
- Construction of a Temporary Demolition Bypass Channel near the existing lock
- Disposal of Dredge Material
- Demolition of the Existing Lock and St. Claude Avenue Bridge
- Two control houses, a maintenance and administration building, a machinery building, a parking lot, a paint shed, an access road and other associated structures and facilities determined to be necessary during future detailed design.

2.1 Project Location

The existing lock is located along the Inner Harbor Navigation Canal (IHNC), also sometimes referred to as the Industrial Canal in New Orleans, Louisiana in Orleans Parish. The IHNC is a man-made canal, originally constructed by non-Federal interests, that serves as a major navigation artery linking the Mississippi River, the Gulf Intracoastal Waterway (GIWW), and Lake Pontchartrain. The canal commences at Mississippi River Mile (RM) 92.6 Above Head of Passes (AHP) and extends 5.0 miles to the north to its terminus at Lake Pontchartrain. Refer to Figure 1 for project location map.

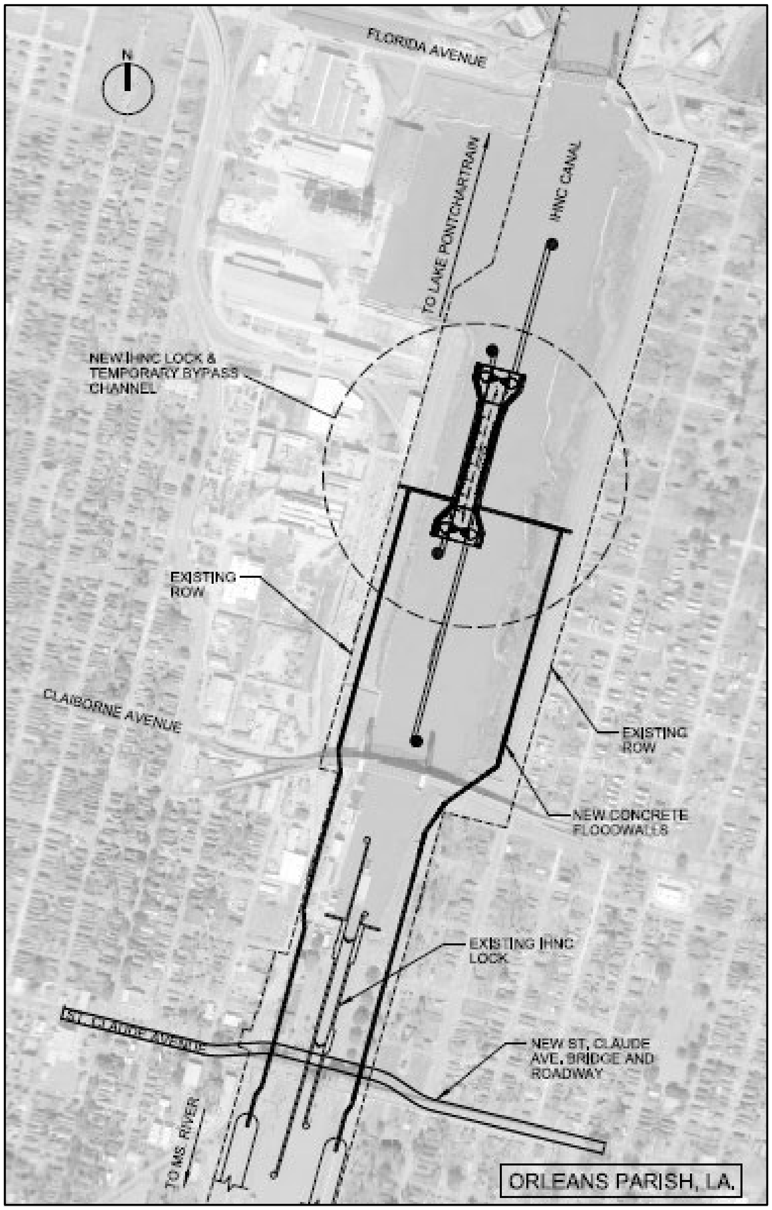

Figure 1: Project Location Map

2.2 Plans for the Study

Preliminary engineering investigations and cost estimates were completed for five alternatives as discussed below in support of the economic analysis performed for the analysis of the final GRR.

Plan 1 - No Action Plan
The existing lock will continue to be operated and be maintained by CEMVN. No detailed engineering investigations were completed for the no action plan; however, chambering times were evaluated for the existing lock as part of the no action plan analysis. Reference Annex 1 for more details on the chambering times.

Plans 2 through 5.

In plans 2 through 5, the shift in location of the new lock necessitates adjustment of both LPV and MRL project features to tie into the new lock.

For the analysis conducted in the draft GRR to select the Tentatively Selected Plan (TSP) five alternatives (referenced as "plans" herein) were analyzed, but cost estimates were developed only for the above-described Plans 2 through 5 as a result of that analysis. Ultimately Plan 3 was selected as the TSP in the Draft Integrated GRR and SEIS that was released for public review and comment in 2017.

For the TSP Phase of this GRR, four (4) different lock sizes were evaluated to develop cost estimates.

2.2.1 Plan 1

This plan is the No-Action. No engineering activities for this plan as part of the study.

2.2.2 Plan 2

This plan would consist of building a new lock (900 ft long x 75 ft wide x (-) 22 ft) north of Claiborne Avenue site.

2.2.3 Plan 3

This plan would consist of building a new lock (900 ft long x 110 ft wide x (-) 22 ft) north of Claiborne Avenue site.

2.2.4 Plan 4

This plan would consist of building a new lock (1,200 ft long x 75 ft wide x (-) 22 ft) north of Claiborne Avenue site.

2.2.5 Plan 5

This plan would consist of building a new lock (1,200 ft long x 110 ft wide x (-) 22 ft) north of Claiborne Avenue site.

3 Recommended Plan

Following the conclusion of the period of public review and comment in 2017 of the Draft Integrated GRR and SEIS, the TSP was adopted as the RP to proceed forward for further analysis in the Final Integrated GRR and SEIS. The analysis for the final report resulted in some revisions, including, but not limited to, the decision to eliminate the plan to construct a temporary bridge at St. Claude Avenue during construction of the new St. Claude Avenue Bridge and approaches. After the Agency Decision Milestone (ADM), which took place after the end of the period of public review and comment on the referenced draft report and SEIS, the Project

Delivery Team (PDT) performed a 35% feasibility level design of the floodwalls and the St. Claude Avenue Bridge for the RP.

The PDT assessments resulted in a decision to eliminate the temporary bridge on St. Claude Avenue, and to construct a permanent bridge north of the existing St. Claude Avenue. By eliminating the temporary bridge, there are reductions to the overall construction schedule, thus reducing the duration of noise and construction impacts to the nearby community. Refer to section 3.6 for additional information regarding St Claude access since the temporary bridge was eliminated. The new location of the St. Claude Avenue Bridge will affect three residential houses located on the northwest quadrant of the existing St. Claude Avenue Bridge.

4 Feasibility Level Design for the Recommended Plan

No additional feasibility level designs were performed on the lock for the RP, except for feasibility level design performed on the floodwalls along the IHNC, new levee sections along the west side of the canal, and on the St. Claude Avenue Bridge. Refer to sections 8 and 12, Annexes 8, 13 and 14 of this Engineering Appendix for details on the feasibility level design of the St. Claude Avenue Bridge. Refer to sections 2, Annex 8, of this Engineering Appendix for details on the feasibility level design of the floodwalls.

The floodwalls for this project will be constructed to El 24.5 to match the required top of wall (TOW) for the lock. Since the floodwalls will be constructed along the IHNC channel, which is subject to high barge traffic, a 500-kip barge impact force will be used for design.

The new IHNC floodwalls will extend from the Mississippi River on the west side of the IHNC channel, and tie into the new lock. The length of floodwall assumed was approximately 5,200 ft on the west side of the IHNC Channel. On the east side, there will be two small segments of floodwall under the St. Claude and North Claiborne Ave. bridges, as well as a small segment from the new lock tying into the levee on the east side. Typical floodwall monoliths will be constructed of cast-in-place concrete and will be roughly 60 feet in length, connected with a joint containing a waterstop. Each monolith will be supported with steel piles driven with batters.

Portions of the existing I-wall and T-wall sections along the IHNC channel require demolition as part of this project. In addition, the existing concrete scour protection will be demolished and replaced with compacted fill embankment. New concrete scour protection will be added after re-grading of this area. The total cubic yards (CY) for demolition of both existing floodwall and scour protection is approximately 13,000 CY.

As part of the new floodwall construction, new T-wall monoliths are required underneath the existing St. Claude Avenue and Claiborne Avenue bridges. The T-walls required under the St. Claude Avenue Bridge will be coordinated with the new bridge construction. To further note, T-walls can be constructed up to the proposed bridge location, prior to removal of the existing bridge. After the existing bridge has been removed, the T-walls can be tied into the new line of protection.

At a property previously owned by the Port of New Orleans and previously occupied by the U.S. Coast Guard located on the west side of the IHNC channel, there are two sites that have been identified through prior HTRW environmental site assessment investigations where contamination is known to exist. Sampling at these two sites indicated that total petroleum hydrocarbons as diesel, total petroleum hydrocarbons as oil, and some polycyclic aromatic hydrocarbons (benz(a)anthracene, benzo(a)pyrene, benzo(k)fluoranthene, indeno(1,2,3-cd)pyrene, and benzo(a)pyrene) remained at elevated concentrations in both areas (including under a diesel aboveground storage tank). The property was acquired in fee by USACE for the lock replacement project in 2001. The Louisiana Department of Environmental Quality (LADEQ) has determined that if these sites will be disturbed during project construction, the contamination must be remediated.

While the actual construction of the IHNC lock replacement facilities will not disturb these areas, the realignment of the MR&T and LPV floodwalls on the west side could possibly disturb the sub-surface contaminated material that is situated beneath approximately 900 ft of existing LPV floodwall located west of and adjacent to the previous U.S. Coast Guard facility. That section of LPV floodwall likely would be removed in order to extend the MR&T to tie-in to the southward face of the replacement lock.

USACE ER 1165-2-132 dated, June 26, 1992, titled, "Hazardous, Toxic and Radioactive Waste (HTRW) Guidance for Civil Works Projects" provides that construction of Civil Works projects in HTRW-contaminated areas should be avoided where practicable. Where HTRW contaminated areas or impacts cannot be avoided, response actions must be acceptable to EPA and applicable state regulatory agencies (i.e., LADEQ). CEMVN has performed a preliminary examination of the physical extent of the HTRW sites as they relate to the potential floodwall realignment. In 2019, CEMVN contracted JESCO to perform additional environmental site assessment investigations for the known HTRW sites located at the prior USCG site. On behalf of CEMVN, JESCO submitted an April 2019 Risk Evaluation/Corrective Action Plan (RECAP) Site Investigation and Interim Action Report to the Louisiana Department of Environmental Quality, Remediation Division (LADEQ-RD). The LADEQ-RD responded to CEMVN by letter dated March 20, 2023, acknowledging receipt of the April 2019 RECAP Report, and requested USACE provide a site investigation work plan to delineate the vertical and horizontal extent of the contamination. CEMVN responded to LADEQ-RD by letter dated July 24, 2023, providing the requested work plan as well as committing to provide annual status updates of implementation of the work plan to LADEQ-RD no later than October 30th of each calendar year. CEMVN also advised LADEQ-RD that implementation of the work plan is contingent upon receipt of federal funding (construction funds) after completion of the lock replacement study. In a letter dated November 20, 2023, LADEQ-RD acknowledged completion of their review of the work plan and concurred with continued coordination both annually as well as upon receipt of federal funds and subsequent implementation of the work plan.

During future engineering and design prior to construction, CEMVN will implement the aforementioned work plan in coordination with LADEQ-RD to determine if there is a practicable way to avoid disturbance of the affected section of LPV floodwall. If it is determined that there is no practicable, cost-effective way to avoid disturbance of the affected section of LPV floodwall, then CEMVN would perform additional coordination with LADEQ-RD and a Corrective Action Plan would be prepared for LADEQ-RD approval to determine the appropriate remediation actions. As it would be the lock replacement project that would require alteration of the existing LPV alignment in order to tie-in the MR&T floodwall to the replacement lock, if alteration of the present LPV floodwall in the vicinity of HTRW materials were required, that cost would be borne by the lock replacement project.

For additional information on HTRW, refer to the GRR, and to Exhibit 5.

The new levees for this project will be constructed to El 24.5 to match the required top of wall (TOW) for the lock. The new levee will extend mainly on the east side from the MRL Levee north to the new lock tie-in floodwall, except for the portions of floodwall beneath the St. Claude and North Claiborne Ave. bridges. The footprint of the levee will extend roughly 300 feet, with a 10-foot crown and 1 foot vertical to 3 feet horizontal side slopes. The new levee will be constructed in two phases, a temporary levee to El. 17.5, and then the final levee to El. 24.5.

The new bridge is 70 ft wide with two (2), 12 ft wide eastbound lanes and two (2), 12 ft wide westbound lanes. Four (4) foot shoulders are provided on the outside and minimum one-foot shoulders are provided on the inside. A 6-ft wide pedestrian/bicycle lane is provided on the outside edge of the eastbound lanes, separated by traffic with a concrete barrier. A 7-inch reinforced concrete slab/deck was preliminary sized for the bridge approaches. Based on a proposed 7-ft 3-inch spacing between girders and typical 80-ft span between approach pier bents, an AASHTO Type III precast prestressed concrete girder was selected to support the approach decks. Eighteen-inch steel pipe piles were assumed to support the approach piers. Pile capacity curves used for the floodwalls were utilized for the pile tip selection. Initial design and quantities are based on a similar bascule bridge design constructed in another location. The foundation design will be site adapted for this project (pile design, bridge pier design, etc.). Bascule spans were selected to span the existing/future channel alignment and the demolition bypass channel alignment during demolition of the existing lock.

Construction of the new St Claude Avenue Bridge will be phased such that thru traffic along the existing St. Claude Avenue Bridge will be maintained, with the exception of any typical bridge closures to pass navigation, for the entire construction duration. In the event that restriction of thru traffic is required for construction of tie-ins; closures will be minimized to nights and weekends during low traffic volume periods. Additional details regarding traffic control will be developed with the Port of New Orleans, Louisiana Department of Transportation and Development (LA-DOTD) and the City of New Orleans during future detailed design.

Because it is a low-speed urban street, the design speed along St. Claude Avenue is 35 MPH. The speed limit across the proposed bridge is to remain the same. The new bridge is proposed to be placed north of the existing bridge deck, due to the necessity of keeping the existing bridge open during construction. Therefore, a series of horizontal curves will be needed to tie into the existing approach ramps to the proposed bridge deck. From the AASHTO Green Book, the minimum radius of curve at 35 MPH is 419 ft at the centerline, which is the radius of all four horizontal curves.

The proposed bridge deck elevation is (+) 39 ft, whereas the existing bridge deck elevation is approximately (+) 20 ft. However, the approach ramps must tie back to the existing tie-ins along St. Claude Avenue at both Poland Avenue and Reynes Street. The approach ramps are steeper in grade than the existing ramps, but with the addition of longer vertical curves, still suitable for traffic. Three existing homes along the west side will require demolition in order to construct the new St. Claude bridge.

The GRR main report includes identification of resources associated with the project area due to the new bridge construction. Refer to Chapter 6 of the GRR main report, Environmental Consequences (which are the human, natural and cultural environment consequences).

4.1 Cost for the Recommended Plan

In 2019, after ADM, the cost for the RP was updated, using the MII cost estimate. These costs have been updated using further design refinement and a cost and schedule risk analysis to update the contingency (in 2024 dollars):

Table 1: Estimated Cost Breakdown (with Contingency) for RP (2024 dollars)

Estimated Cost (with Contingency) for the RP (Lock is 900 ft x 110 ft x 22 ft sill elevation)		
Structure	**Cost (2024 Dollars)**	**Assumptions**
PED	$616,170,000	Includes spent costs of $139,272,000 through FY 2024.
Lands and Damages	$5,795,000	Provided by Real Estate Division
Relocations	$598,072,000	Includes Demolition and replacement of St. Claude Avenue Bridge
Lock	$2,094,344,000	Incudes demolition of the existing lock and spent costs of $33,418,000 for previously completed demolition.
Channels and Canals	$115,184,000	
Levees and Floodwalls	$615,182,000	
S&A	$366,563,000	

Table 2: Total Estimated Cost (with Contingency) for the RP (2024 dollars)

Total Estimated Cost for the RP	
900 ft x 110 ft x Sill El. (-) 22.0	**2024 Dollars**
Estimated Cost including contingency, PED and S&A	$4,411,312,000
Mitigation (CIMP and TMP)	$276,260,000
Total Estimate Cost with Mitigation	$4,687,572,000

The 2024 project cost estimate was developed in the MCACES MII cost estimating software and used the standard approaches for a feasibility estimate structure regarding labor, equipment, materials, crews, unit prices, production rates, material quotes, sub and prime contractor markups. This philosophy was taken wherever practical within the time constraints. It was supplemented with estimating information from other sources where necessary such as updated information from the 1997 Evaluation Report to the extent that information remained applicable to the RP, material quotes, current published estimating information, historic bid data, and A/E estimates. The intent was to provide or convey a “fair and reasonable” estimate that depicts the local market conditions. The estimate assumes a typical application of tiering subcontractors for certain project construction features. Given the 10+ year span over which this project/program is to be constructed and the unknown economic status during that time, demands from non-governmental civil works projects were not considered to dampen the competition and increase prices.

Contingency for the Recommended Plan

In 2019, after the determination of the RP by the ADM, project contingencies were developed using the USACE Cost and Schedule Risk Analysis (CSRA) to define cost and schedule related risks pertaining to project uncertainties. In 2024, we completed a new CSRA to identify and update all the project risks and develop a new project contingency. The following table lists the major project elements and the corresponding contingency as calculated by the recent CSRA.

Table 3: Contingency for the RP (2024 dollars)

Feature of Work	**Contingency in CSRA (%)**
Lands and Damages	30 See note)
Relocations	80
Locks	80
Channels and Canals	80
Levees and Floodwalls	80
Planning, Engineering and Design	80
Construction Management	80

Note: Contingency furnished by MVN Real Estate Division

Additional information may be found in Annex 3, which contains the Cost Engineering Annex.

4.2 Changes from Alternatives Selection to the Agency Decision Milestone

During the Alternatives Selection milestone, a temporary St. Claude Avenue Bridge was assumed as shown in the 1997 Evaluation Report Addendum. The temporary bridge was removed from the study following the ADM. By eliminating the temporary bridge, there are reductions to the overall construction schedule, reducing the duration of noise and construction impacts to the nearby community. The cost and duration savings comparing the original TSP which consisted of a temporary and permanent St. Claude Avenue Bridge relocation, versus the current RP which consists of a permanent St. Claude Avenue Bridge replacement is approximately $5 million to $20 million in construction cost and approximately 2 years in construction duration. Refer to construction schedule in Annex 4 for the construction schedule for the RP.

Construction of the new St Claude Avenue Bridge will be phased such that thru traffic along the existing St. Claude Avenue Bridge will be maintained, with the exception of any typical bridge closures to pass navigation, for the entire construction duration. In the event that restriction of thru traffic is required for construction of tie-ins; closures will be minimized to nights and weekends during low traffic volume periods. Additional details regarding traffic control will be developed with the Port of New Orleans, Louisiana Department of Transportation and Development (LA-DOTD) and the City of New Orleans during future detailed design.

4.3 Project Schedule for the Recommended Plan

In 2019, the project construction schedule for the RP was revised based on the construction of the individual features of work which included the dredging of a bypass channels, canal excavation, construction of a cofferdam, the new IHNC lock, a new permanent bridge for St. Claude Avenue, earthen levees, and floodwalls. Other activities include the demolition of the existing lock and the existing St. Claude Bridge. This schedule was reviewed and revised to reflect the construction features and assumptions during the 2024 evaluation and updates.

For the construction schedule for the RP, refer to Annex 4 Classic Schedule Layout.

4.3.1 Design Schedule

The schedule layout prepared for the study, assumes that the design phase will last four consecutive years. Design was assumed to start in Year 1 of the project (2029) and be completed in Year 4 of the project (2032). Using the RP schedule, during alternative formulation stage, the PDT decided to assume Year 1 of the project to be 2029. Year 1 is dependent on the assumption that initial funding in FY28 will be provided. Assumed construction funding will follow the Capital Investment Strategy, Scenario 2.

Cost allocations for the RP described in this document are subject to the provisions of Section 102 and 844 of WRDA 1986, as amended by Sec. 1126 of WRDA 2024 (P.L. 118-272). WRDA 1986, as amended, requires twenty-five percent of the Federal costs for the RP to be appropriated from the Inland Waterways Trust Fund and seventy-five percent to be appropriated from the general fund of the Treasury as a part of the USACE appropriated budget.

As agreed with the IEPR team, the project will conduct a pile load test and a noise study during future detailed design.

3.4.2 Construction Schedule

Construction activities associated with the implementation of the lock replacement project features will not negatively impact the ability of the existing flood risk reduction and hurricane storm damage risk reduction projects to provide their respective authorized and constructed levels of risk reduction. The construction sequence shows that, as necessary, the MRL levee/floodwall alignment will be extended from its existing location to the point where the alignment will tie in to the riverward face of the replacement lock in its new location. The construction of the extended levee/floodwall alignment will occur prior to the removal of any existing floodwalls, ensuring a continuous line of risk reduction.

The project construction schedule for the RP was revised in 2024, and was based on the construction of the individual features of work which includes the dredging of a bypass channel near the new lock site, canal excavation, construction of a cofferdam, the construction of the replacement lock in accordance with the dimensions and method of construction identified as the RP , a new permanent double-bascule bridge for St Claude Avenue, and adjustment to the alignment of the MRL and LPV projects, as made necessary by the shift in location of the replacement lock from where the existing lock presently ties in to the MRL and LPV projects. For a detailed breakdown of the construction schedule, including the assumptions used in the cost development, refer to Annex 4, Classic Schedule Layout for the RP.

For the RP, assuming the provision of initial funding in FY28, construction starts in Project Year 4 of the project (year 2033). Construction of the project as a whole is assumed in the RP to be completed on Project Year 14 (year 2047), which is a total of approximately 10 years (this does not include design). This results in a Base Year 2035, and a period of analysis of 50 years, ending in year 2084.

For this analysis, 50 years is used for economic analysis. The minimum project service life of 100 years for major infrastructure projects such as locks, dams, and levees" per ER 1110-2-8159, "Life Cycle Design and Performance”.

Table 4: Durations for Project Features for the RP

Feature	Duration (years)	Notes
Detailed Design	4	Project Year 1 is year 2029. Detailed Design ends in Year 4 (2032).
Construction- Project Year 4 to 14 (2033 to 2047)		
New Lock Channel Excavation and Construction of the Lock	8	New lock channel excavation and construction of the lock. It includes: - New lock channel excavation: 0.5 years (approx. 6 months) - New lock construction: 8 years
Existing (old) Lock Excavation and Demolition	1	- Existing (old) lock demolition: 0.5 years (6 months) - Old lock channel excavation: approx. 6 months
Levees and Floodwalls	6	- Levee and floodwall construction is estimated to occur concurrently with the new lock construction over various durations.
New St. Claude Avenue Permanent Bridge	2	A portion of the floodwall under the St. Claude Avenue Bridge will be constructed during construction of the St. Claude Avenue Permanent Bridge.
Existing St. Claude Avenue Bridge Demolition	1	- Demolition of the existing St Claude Avenue Bridge will occur after the new St Claude Avenue Bridge is open to vehicles.

3.5 Construction Duration of the Lock Structure for the Recommended Plan

Construction of the project will be completed in Year 14 of the project (2047). Since the lock chamber will be completed prior to the completion of the project as a whole, the OMRR&R of the lock chamber structure was assumed to start in Year 10 of the project (2042), which is the year following construction completion of the lock chamber itself, in Year 9 (2041).

3.6 Construction Sequencing for the Recommended Plan

The overall construction sequencing of the new lock structure, etc. is listed below. As part of this GRR, the construction sequencing and cost estimates reflected a cast-in-place (CIP) construction methodology in lieu of float-in-place (FIP) for the lock.

For a detailed breakdown of the construction sequencing and project schedule, including the assumptions used in the cost development, refer to Annex 4.

After the ADM, the RP was taken into feasibility level design, and the construction plan was refined as shown below. After the Ship Simulation was run and the bypass channel was tested by the Navigation Industry, it was determined the proposed bypass channel and bridge near the existing lock would have to be modified. The existing lock chamber would be utilized during the demolition sequence and allow for transit of navigation traffic during nighttime hours. Daytime closures would occur to allow for the removal of the existing lock chamber from the bank and from floating plants. A demolition bypass channel would be required on the east side of the existing lock (after the east wall is demolished) to aid in passing vessels during demolition of the west lock wall and incorporation into the future permanent navigation channel.

The proposed construction sequencing (for the RP) is as follows. Note there are two construction sequences: first on the new lock area and adjustments to the existing respective alignments of the MRL/LPV levees and floodwalls as necessary to address the location of the new replacement lock, and a second for the existing lock area. The existing lock area sequence (second sequence) is to be performed after the construction of the new lock area and associated levee/floodwalls re-alignment is completed.

Table 5: Construction Sequence for the RP

New Lock Area and Floodwalls Sequence
1 – Construct cofferdam (along bypass channel)
2 – Dredge (bypass and new lock)
3 – Construct cofferdam (across channel) and protective dolphins
4 – Unwater, excavate, build lock complex
5 – Place Embankment, Compacted Fill and construct East and West T-walls inside of the cofferdam.
6 – Build portion of guidewalls (in cofferdam)
7 – Remove cofferdam

8 – Build remaining tie-in T-walls, embankment, and permanent levee on east side. Demolish existing east T-walls north of Claiborne Ave. Complete guidewalls, end cells and mooring cells.

Existing Lock Area Sequence (Revised for New Bridge on North Side of St. Claude Avenue):	
All items below are to be performed after the new lock and floodwalls are completed.	
1 – Build west T-walls (All T-walls except those at the existing and new St. Claude Avenue Bridge can be constructed concurrent with new lock area construction) and east levee sections.	
2 – Build bridge piers/protective dolphins (for lift bridges)	Build bridge approaches concurrent with construction sequence number 1 through 3.
3 – Install steel gates leaves for bascule bridge (shutdown vessel traffic) 30-45 days	*Major portion of the steel gates can be constructed in place in the vertical position.
4 – Complete new bridge and make tie-in to existing road. Include T-wall monoliths underneath new bridge	
5 – Demolish existing St. Claude Bridge	
6 – Construct remaining T-wall under footprint of now demolished old St. Claude Avenue Bridge (risk reduction in place)	

7 –Demolish East Lock (walls only) and excavate demolition bypass channel. Navigational traffic to utilize demolition bypass channel after removal of the East Lock wall and excavation. Demolition Bypass Channel to become a part of permanent navigation channel after demolition is completed.	*Navigational traffic to use traffic windows at night, lock demolition activity in main channel in daytime hours
8 – Demolish West lock (walls only)	
9 – Final establishment of channel along existing alignment	

Note:

Construction of the new St Claude Avenue Bridge will be phased such that through vehicular traffic along the existing St. Claude Avenue Bridge will be maintained, with the exception of any typical bridge closures to pass navigation, for the entire construction duration. In the event that restriction of through traffic is required for construction of tie-ins; closures will be minimized to nights and weekends during low traffic volume periods. Additional details regarding vehicular traffic control will be developed with the Port of New Orleans, LA-DOTD and the City of New Orleans during future detailed design.

Refer to Annex 11, Sheets CS-101 to CS-115 for construction sequence of the project.

3.7 Relocations Updates after Agency Milestone

After ADM, the MVN EDD Design Services, Relocations Team performed an investigation of the existing public utilities and facilities located within the proposed project area, while considering the current design requirements for the RP described in this GRR. The limits within the IHNC corridor were from the Florida Ave. bridge, extending south of the St. Claude Ave. bridge where it ties to the existing MR&T features (refer to Annex 11, Plates, Sheet CS-110).

The Relocations Team used U.S. Pipelines and Facilities (IHS, Inc.), Louisiana Department of Natural Resources (LA-DNR), and National Pipeline Mapping System (NPMS) pipeline databases, along with the USACE's permit tracking system called Pipeline Location Observation & Verification Enterprise Repository (PLOVER), to locate utilities within the proposed project area. The PLOVER system was cross-referenced with the other pipeline databases to provide approximate locations of existing utilities and to reveal any new utilities placed since Hurricane

Katrina in 2005. In-house investigations and facility owner notifications were also used to verify the location of current utilities within the IHNC canal.

It was confirmed that several utilities were removed between the North Claiborne Bridge and Florida Ave. Bridge. These utilities were located in the batture area along the eastside of the

IHNC prior to award of construction projects to rebuild and reinforce the HSDRRS features in the area. These utilities were flushed, cut, and capped before the construction of a new floodwall after Hurricane Katrina. The discarded remaining portion of these utilities may still be located within the channel portion of the IHNC. These utilities are now discontinued and no longer in use.

Table 6: Facilities Removed after Hurricane Katrina

Location	Owner	Utility*	Disposition
Florida Ave. Bridge	**Utilities located south of Florida Avenue Bridge**		
	NOSWB	6 - inch waterline	Removed
	ENTERGY GAS	3 - inch gasline	Removed
	ENTERGY GAS	6 - inch gasline	Removed
	NOSWB	6 - inch waterline	Removed
	ENTERGY GAS	4 - inch gasline	Removed
	ENTERGY GAS	2 - inch gasline	Removed
	ENTERGY GAS	3 - inch gasline	Removed
	NOSWB	6 - inch waterline	Removed

*Utilities were removed after Hurricane Katrina, as part of HSDRRS construction.

Facility owners that were identified by the pipeline databases, permits, and construction activities, were contacted by the Relocations Team. The contacted facility owners: New Orleans Sewerage and Water Board (NOSWB), Cox Communications, and Entergy Gas and Entergy Distribution provided drawings or notations of their utilities crossing the IHNC.

The following utilities were identified but appear not to be impacted by the proposed project. Using utility drawings provided by NOSWB and Google Earth, the NOSWB's utilities (Siphon, 54-inch SMF, 48-inch Waterline, and the 66-inch SFM) confirmed that these utilities located just south of Florida Avenue appear not to be impacted by the proposed lock location and mooring cells. However, there is no conclusive evidence that the two 6-inch conduits on the drawings submitted by Entergy Distribution verify their exact locations. Also, a 5-inch cable from Cox Communications, identified via PLOVER, only gives an approximate location within the vicinity of the Florida Ave. Bridge.

The proposed relocation for these impacted utilities will be performed by directional drilling. For feasibility level purposes, it is assumed that no new Right of Way will be needed during the directional drilling operations of the utilities; no temporary construction easement will be needed for directional drill purposes.

Note the area immediately south of the St. Claude Ave. Bridge, where the project ties into the existing levee, does not currently have any utilities in place. Also, the RP design will not impact the New Orleans Public Belt Railroad.

3.7.1 Facility Disposition after ADM

The utilities located just south adjacent to the Florida Avenue Bridge and those just north adjacent to the Claiborne Avenue Bridge will not be impacted. They are listed, marked, and labeled as "Do Not Disturb."

Table 7: Facility Disposition for the RP (2025)

Location	Owner	Utility	Disposition
Florida Avenue Bridge	**Utilities located south of Florida Avenue Bridge**		
	NOSWB	Siphon -Florida Ave. Canal	Do Not Disturb
	NOSWB	54 - inch sewerage force main	Do Not Disturb
	NOSWB	48 - inch water main	Do Not Disturb
	NOSWB	66 - inch sewerage force main	Do Not Disturb
	COX COMMUNICATIONS	5 - inch communication cable	To Be Relocated Concurrent with Construction
	ENTERGY DISTRIBUTION	Approximately 1,110 ft of two (2) – 6-inch conduits with three (3) - 750al mcm cables in each conduit	To Be Relocated Concurrent with Construction
North Claiborne Bridge	**Utilities located south of North Claiborne Avenue Bridge**		
	ENTERGY GAS	Entergy Gas two (2) - 16 - inch natural gas pipelines	To Be Relocated Concurrent with Construction
St. Claude Bridge	**Utilities located north of St. Claude Avenue Bridge**		
	NOSWB	20 - inch waterline	To Be Relocated Concurrent with Construction
	NOSWB	Two (2) - 30 - inch reinforced concrete pipelines	To Be Relocated Concurrent with Construction
	NOSWB	20 - inch C.I. pipeline	To Be Relocated Concurrent with Construction
	ENTERGY DISTRIBUTION	Under existing lock, approximately 400 ft of twelve (12) - 3.5-inch conduits 6 conduits with one (1)- 750al mcm 25kv cables in them	To Be Relocated Concurrent with Construction

Note that since it is anticipated that the utilities that need to be relocated will be directionally drilled, the utility corridors proposed in the 1997 Evaluation Report will no longer be needed. The assumption is that the relocation will occur within existing ROW, so no new ROW will be needed.

3.8 Estimated Relocations Costs for the Recommended Plan

In February 2019, and during feasibility level design of the RP, the MII cost estimate was updated to comply with ER requirements for cost estimate not to exceed 2 years in age. In the new MII cost estimate, 900 ft x 110 ft x 22 ft lock, relocations costs were updated based upon revised utility information and quantities prepared and furnished by the Relocations Team. The furnished information included the utility owner, type of utility, size, location, and number of utilities. This information was reviewed and verified for the 2024 update. The total estimated cost for relocations detailed above is $598,073,000 (including $487,106,446 for the demolition of the existing St. Claude Avenue bridge and the construction of the new replacement bridge). A contingency (80%) was applied to all relocations costs.

For the RP, the total estimated construction cost (includes real estate, PED, construction, S&A, and Community impact mitigation) is $6,223,974,000. The estimated cost for facility and utility relocations is $735,628,495. The Relocations represent 11.8 % of the total construction cost. For this analysis, we referred to two memorandums: Corps of Engineers, Office of the Chief Counsel (CECC-R) Bulletin 13-1: Preliminary Attorney's Opinion of Compensability, dated January 14, 2013, and Real Estate Policy Guidance Letter No. 31- Real Estate Support to Civil Works Planning Paradigm (3x3x3), dated January 10, 2019. Since the Relocations represent 11.8 % of the total estimated costs, which is less than 30 % of the estimated total project costs; the above cited guidance allows the District to defer preparation of an Attorney's Opinion of Compensability until final design is obtained during future detailed design. A real estate assessment is included in the Appendix C, Real Estate Plan (REP), in accordance with the cited guidance.

Table 8: Relocation Costs for Facilities and Utilities for the RP (2024 Dollars)

Location	**Owner**	**Utility**	**Relocation Cost ***
Florida Avenue Bridge	**Utilities located south of Florida Avenue Bridge**		
	NOSWB	Siphon -Florida Ave. Canal	**
	NOSWB	54 - inch sewerage force main	**
	NOSWB	48 - inch water main	**

	NOSWB	66 - inch sewerage force main	**
	COX COMMUNICATIONS ***	5 - inch communication cable	$ 2,353,500
	COX COMMUNICATIONS ***	Communication cable	$2,353,500
	ENTERGY DISTRIBUTION ***	Approximately 1,110 ft of two (2) - 6" conduits with three (3)- 750al mcm cables in each conduit	$ 2,574,000
	AT&T	20" cable	$23,931,000
	AT&T	Fiber	$2,353,500
	AT&T	10" fiber	$7,425,000
North Claiborne Bridge	**Utilities located south of North Claiborne Avenue Bridge**		
	ENTERGY GAS	Entergy Gas two (2) - 16 - inch natural gas pipelines	$ 14,355,000
St. Claude Avenue Bridge	**Utilities located north of St. Claude Avenue Bridge**		
	NOSWB	20 - inch waterline	$ 6,210,000
	NOSWB	Two (2)-30 - inch reinforced concrete pipelines	$ 27,036,000
	NOSWB	20 - inch C.I. pipeline	$ 6,210,000
	ENTERGY DISTRIBUTION	Under existing lock, approximately 400 ft of twelve (12) - 3.5-inch conduits 6 conduits with one (1) - 750al mcm 25kv cables in them	$ 11,016,000

	ENTERGY	4-6” conduits	$5,148,000
St. Claude Avenue Bridge	Port of New Orleans	Low level bascule bridge	$487,106,446
		SUB-TOTAL	**$598,071,946**
		S&A and PE&D (23%)	**$137,556,549**
		TOTAL	**$735,628,495**

* 2024 Relocation costs shown are based upon directional drill and include project contingency.

** Insufficient data to determine if the utilities are impacted by new lock construction features. This unknown was addressed through the Cost and Schedule Risk Analysis (CSRA) contingency program as a risk/opportunity event.

*** Insufficient data given to verify exact location. Therefore, relocation costs are included based on possible impact by new lock location and construction features.

4 Hydrology and Hydraulics

4.1 Existing Hydrology Conditions

The project is impacted to the south by water levels on the Mississippi River and to the north by tidal influences and tropical storm/hurricane surges from the Gulf of Mexico (GOM) via Lake Pontchartrain. Before the 2009 closure of the Mississippi River Gulf Outlet (MRGO), the study area was also influenced by tidal influences and tropical storm/hurricane surges from the GOM that propagated downstream of the lock via MRGO.

4.2 Preliminary Hydraulic Investigations

In support of the economic analysis, MVN Hydrology, Hydraulics and Coastal Branch (CEMVN EDH) completed an analysis to determine filling times using a computer program, called LOCKSIM. Filling times were calculated for the five plans. The results are presented below in Table 9. Detailed information concerning the LOCKSIM model are contained in Annex 1 to this Engineering Appendix.

4.3 Filling and Emptying Times from LOCKSIM

In the 1997 Evaluation Report, the emptying and filling times for the proposed plans were calculated using the USACE computer program called H5320, LOCK FILLING AND EMPTYING—SYMMETRICAL SYSTEMS (Hebler and Neilson 1976). Although, H5320 was widely accepted at the time of the study, it had limitations and required improvements. H5320 did not accurately account for the discharge capability of the lock and was only suitable for symmetrical lock systems. The computer program did not take into account the free surface at the upper and lower approaches and within the lock chamber or the pressure and discharge in the emptying and filling manifolds.

For this update to the GRR, the hydraulic modelers used the newer modeling software called LOCKSIM which was developed in 1999 by Dr. Gerald A. Schohl, Tennessee Valley Authority, in collaboration with the USACE ERDC. LOCKSIM corrected the limitations of the previous model. The model uses unsteady pressure-flow equations which are applicable to the conduits within the system in combination with the free-surface equations and allows the user to describe the approach reservoirs, valve wells, and lock chamber. The model computes pressures and flow distributions throughout the entire lock system.

The filling times presented in this report supersede the previous feasibility study. The results from the two models should not be compared because of the difference in hydraulic methodologies.

Table 9 provides the filling times which are an input to the economic model used to select the RP. Reference the GRR Economics Appendix for more details on the RP analysis and selection.

Table 9: LOCKSIM Results

INNER HARBOR NAVIGATIONAL CANAL LOCKSIM MODEL RESULTS							
LOCK SIZE (ft)			FILLING TIME (mins)				
			LIFT (ft)				
PINTLE TO PINTLE DISTANCE (ft)	Length	Width	2	4	6	8	10
	EXISTING LOCK (OBSERVED)						
	675	75	3.00	6.00	9.00	12.00	15.00
	EXISTING LOCK						
	675	75	3.12	4.08	4.88	5.57	6.18
	ALTERNATIVES						
	970	75	3.33	4.03	4.60	5.10	5.55
	970	110	4.08	5.13	6.02	6.80	7.48
	1288	75	4.10	4.90	5.50	5.98	6.42
	1288	110	4.97	6.03	6.88	7.62	8.30

4.4 Lock Authorization and Hydraulic Design Elevations

The lock is authorized by the 1956 Act and subsequent acts as discussed in Chapter 1 of the GRR. It is within the MRL system alignment which is authorized by the MR&T project; the upstream gates face the river and connect to MRL system features. The 1973 Flowline Study provides requirements for design elevations at each river mile along the Mississippi River and freeboard that varies at each river mile. The IHNC lock replacement project is located at RM 92.6 AHP. At this location, the authorized flowline elevation is 17.3 ft with an additional 4.8 ft of freeboard, which results in a required design elevation of 22.1 ft (referenced to NAVD88 2004.65). Freeboard was added to the computed flowline elevation to account for uncertainty and includes 1 ft of freeboard to account for future deterioration, now referred to as subsidence.

4.5 Navigation Concerns in Previous Studies

Ship simulation modeling was performed in 2008 and again in 2023; the initial report is titled Simulation Study for Preferred Construction Method for Proposed 1200-ft Lock on IHNC, Cast-in-Place versus Float-in-Place, dated 23 June 2008. This report was prepared as part of the study for the cast-in-place (CIP) solution with a (-) 40 ft sill. The ship simulation model study recommended use of assistance vessels through the temporary bypass channel that would be required during construction. The pilots had difficulties maneuvering the design vessel on the south end of the cofferdam at the Claiborne Avenue Bridge. The cofferdam was wide and located in close proximity to the bridge. The width of the cofferdam has decreased with the shallower lock sill investigated as part of this GRR. Navigation through the temporary bypass channel should therefore improve, but effects on navigation will not be fully determined until the ship simulation model is updated with design features for this study. In the 2023 ship simulation, pilots from the Navigation Industry raised concerns with the safety of the bypass channel near St. Claude Ave. Bridge. Additional tabletop exercises were held, and the plan was revised near the existing lock to accommodate safety concerns with the bypass channel geometry. This work provided verification of final lock position and channel geometry. The next Ship Simulation will be used to model and refine the construction sequence, determine the length of time to navigate through the construction reach, and recommend safety aids during construction which will be implemented to improve transit times. This model will be completed in the future detailed design phase or as soon as funding is available and documented in the Design Documentation Report (DDR).

4.6 Relative Sea Level Change

Relative Sea Level Change (RSLC) is expected to affect performance of both the existing and the replacement IHNC Locks, in at least five ways, each of which is detailed below in Section 4.8.3. The siting of the lock between the Mississippi River, Lake Borgne, and Lake Pontchartrain subjects the lock to complex interactions from several coastally influenced water bodies under a variety of conditions. The vicinity map below shows the proposed project location and the several surrounding water bodies influencing the project site.

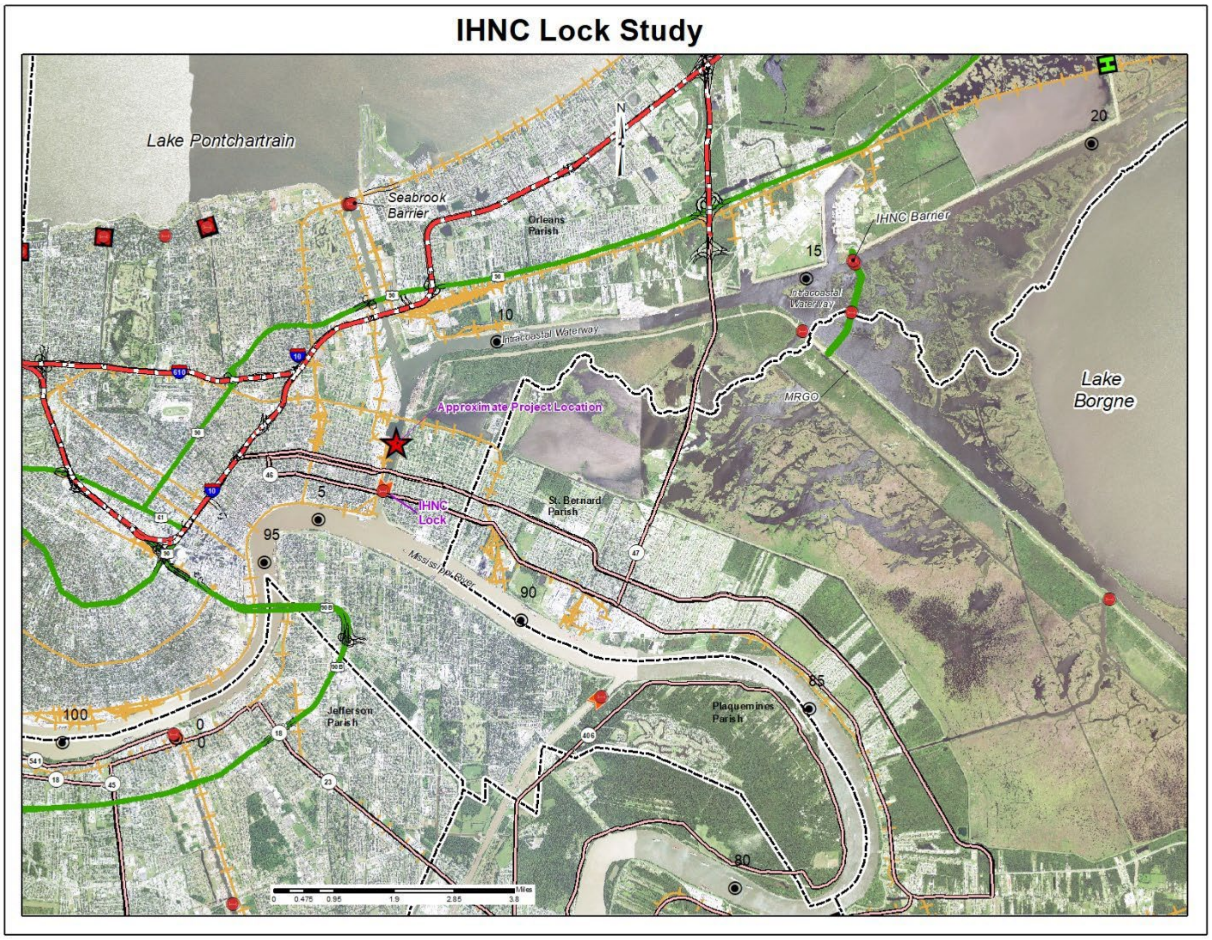

Figure 2: Location of Proposed Project with Surrounding Tidally Influenced Water Bodies

Although their names include the word "lake," both Lake Borgne and Lake Pontchartrain are actually coastal embayments subject to coastal influence including mean sea level change. Lake Borgne is connected to Mississippi Sound and thus to the Gulf of America, while Lake Pontchartrain is connected to Lake Borgne. The Mississippi River, meanwhile, has its mouth approximately 110 miles from the project location, a distance that mutes but does not remove the influence of the sea, especially during periods of low river discharge.

4.7 Relative Sea Level Change Projections

In accordance with Engineering Regulation 1110-2-8162, "Incorporating Sea Level Change in Civil Works Programs," USACE studies in the coastal zone must consider the influence of sea level change on the future with and without project conditions. While sea level change represents an uncertainty in future conditions, projects must be able to perform for their intended design lives despite this uncertainty. The plausible range of future conditions, based on the latest actionable science, is defined by the three USACE sea level scenarios: Low, Intermediate, and High. ER 1110-2-8162 par. 6.d. specifies three potential approaches to considering sea level change; in this case the team elected to apply approach (1), working with a single sea level scenario (Intermediate, in this case) to identify the preferred alternative, and then evaluating the performance of that alternative under the other two scenarios. ER 1110-2-8162 states that this approach "may be most appropriate when local conditions and plan performance are not highly sensitive to the rate of SLC." While local conditions in this area are indeed sensitive to sea level change, the performance of the lock is less sensitive, for several reasons that are detailed below in the sections on sea level impacts.

USACE sea level scenarios are typically projected using the nearest NOAA National Water Level Observation Network (NWLON) gage to the project. In the high-subsidence environment of south Louisiana, this approach is unsatisfactory due to significant local variations in subsidence rates, and therefore relative sea level change rates. The nearest NWLON gage to this project is at Grand Isle, LA, 50 miles away, and the only other NWLON gage in the state is at Sabine Pass on the Texas border. To address this issue, MVN, with the support of the MVD's Mississippi River Geomorphology and Potamology Program published the 2015 Updated Atlas of USACE Historic Daily Tide Data in Coastal Louisiana (Veatch, 2017). This atlas provides sea level change rates for 30 tide gages operated and maintained by MVN, providing better spatial distribution of observed sea level change rates across the district's area of responsibility. Using these gages for generation of sea level scenarios is compliant with policy per ECB 2018-3, "Using Non-NOAA Tide Gauge Records for Computing Relative Sea Level Change."

For this project, three gages are particularly relevant: Mississippi River at IHNC Lock (gage 01340), IHNC at New Orleans (gage 76160) on the canal side of the lock, and at Lake Pontchartrain, West End (gage 85625), which provides an estimate of sea level change in that lake. No long-term gage exists to give a good estimate of sea level change in Lake Borgne (gages exist at Shell Beach and at the Rigolets, but each are some distance from the lock and subject to other influences) but gage 76160 gives a clear picture of the rate of sea level change in lock tailwater. For gages on the Mississippi River, the tidegage atlas provides an estimate of observed change over time for raw data as well as for data after removal of river discharge influence, to separate sea level effects from changes in flood frequency. The rate with river influence removed was used in this study. A summary of pertinent data is shown below in Table 10.

Table 10: Summary of 2015 Updated Relative Sea Level Trends for U. S. Army Corps of Engineers Gages ^

Gage Name	USACE MVN Gage ID Number	Latitude and Longitude	Period of Record Analyzed	2015 Linear Trend for Entire Period of Record (mm/year)
Mississippi River at IHNC Lock	01340	29.964 N, 90.0274 W	Jan 1945 – Dec 2014	**6.8 (tidal only)**
IHNC at New Orleans	76160	29.966 N, 90.0267 W	Jan 1945 – Dec 2014	**10.5**

Lake Pontchartrain at West End	85625	30.022 N, 90.116 W	Jan 1950 – Dec 2014	**8.8**

^ Excerpt of Table 10 from the 2015 Updated Atlas of USACE Historic Daily Tide Data in Coastal Louisiana

As shown in Table 10, the rate of observed rise in the Mississippi River is less than in the IHNC or in Lake Pontchartrain. This is expected, as the effect of sea level rise at the river's mouth tends to decrease with distance upstream (see e.g. Driessen and van Ledden, 2013). However, this difference could also be the result of spatial variation in subsidence or random variations over time. It was decided that as a matter of conservatism, the average rate of land movement from all three gages (0.029 ft/year or 8.7 mm/year) be used to project the sea level scenarios for the site. This rate was entered into the sea level calculator for non-NOAA tidegages (http://corpsmapu.usace.army.mil/rccinfo/slc/slcc_nn_calc.html) as a user-entered rate, resulting in the table ("average rate" in Table 11) and graph (Figure 3) shown below. Assessment of sea level change extends beyond the 50 year economic planning horizon to the end of the project life, which is assumed to be "100 years for major infrastructure projects such as locks, dams and levees" in accordance with ER 1110-2-8159 - Life Cycle Design and Performance. Note the relative changes are shown in ft relative to published mean sea level in 1992, and results in the table are shown for the present year, the project base year (2035), the end of the economic analysis period (2084), and the adaptation horizon (2132).

^^ Base year and period of analysis were revised to year 2035 and 2084 respectively, after this table was prepared. The conclusions presented herein are not materially altered by this change.

Table 11: IHNC Lock RSLC Projections (Values are in Feet)

	Mississippi River at IHNC (01340)			Inner Harbor Navigational Canal at New Orleans (76160)			Lake Pontchartrain at West End (85625)			Average of Three Gages		
Year	Low	Intermediate	High	Low	Intermediate	High	Low	Intermediate	High	Low	Intermediate	High
2019	0.6	0.7	0.9	0.9	1.0	1.2	0.8	0.8	1.1	0.8	0.8	1.1
2032	0.9	1.0	1.5	1.4	1.5	2.0	1.2	1.3	1.7	1.2	1.3	1.7
2035	1.0	1.1	1.6	1.5	1.6	2.2	1.2	1.4	1.9	1.2	1.4	1.9
2082	2.0	2.7	5.0	3.1	3.8	6.1	2.6	3.3	5.6	2.6	3.3	5.6
2084	2.1	2.8	5.2	3.2	3.9	6.3	2.7	3.4	5.8	2.7	3.4	5.8
2085	2.1	2.8	5.3	3.2	4.0	6.4	2.7	3.5	5.9	2.7	3.4	5.9
2100	2.4	3.4	6.7	3.7	4.7	8.0	3.1	4.2	7.4	3.1	4.1	7.4
2132	3.1	4.9	10.4	4.8	6.6	12.1	4.0	5.8	11.3	4.0	5.8	11.3

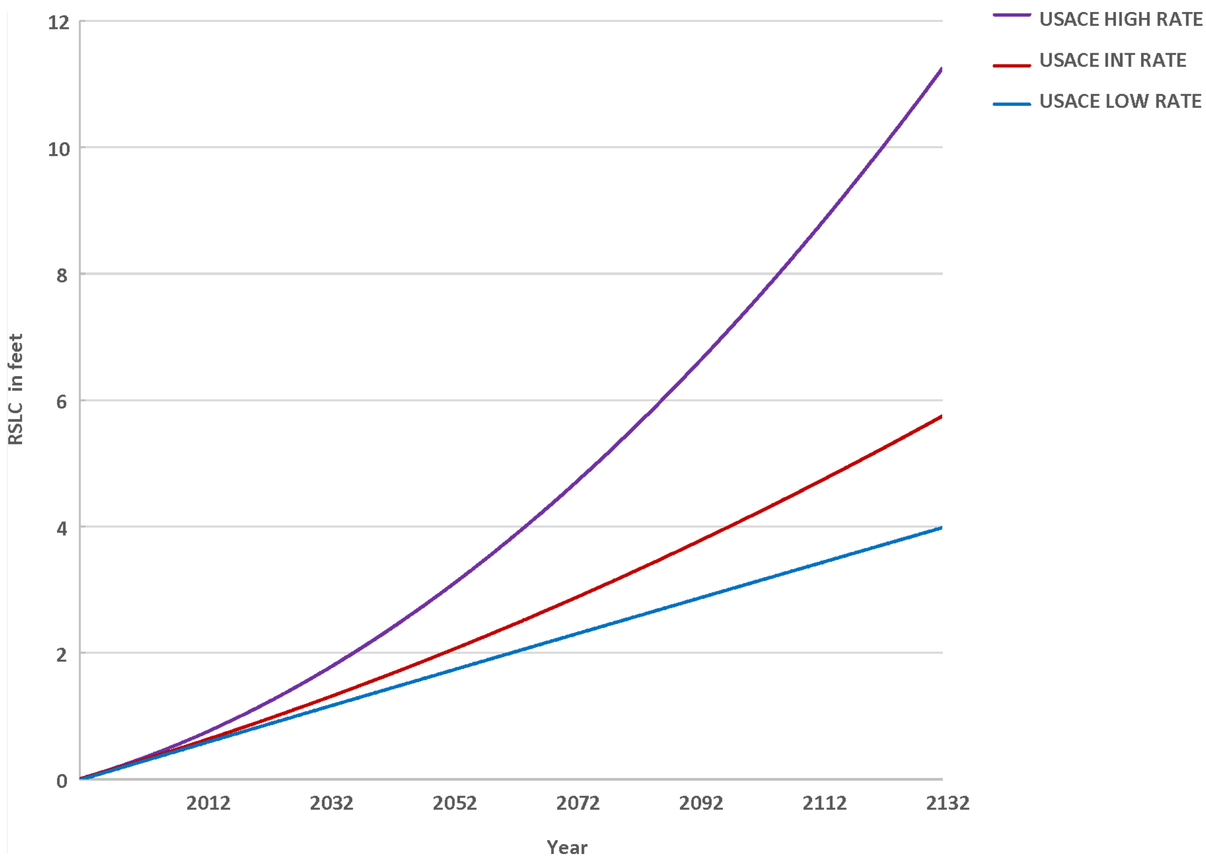

Figure 3: Sea Level Change Projections using the Average Rate for 3 Gages near the IHNC Lock Project Site

4.8 Effects of RSLC on Lock Design

Preliminary investigations were completed to establish the required design heights of the two lock features that are influenced by RSLC, the chamber wall and adjacent relocated floodwalls. These design heights are preliminary and will be refined in future detailed design but can be used for plan selection. These design heights can also be compared to future conditions under sea level change to determine how sea level change can influence project alternative selection. Effects of sea level change on the performance of the selected plan will be addressed in Section 4.8.3.

4.8.1 Chamber Wall and Adjacent Floodwall Height

The IHNC Lock design is based on MR&T hydraulic design elevation. The elevation of the chamber wall h is as follows:

- 17.30 MR&T Flowline elevation referenced to NAVD88 (2004.65)
- Plus 4.80 freeboard (includes uncertainty and 1 ft for future deterioration)

- Plus 2.00 structural superiority[1]

- 24.10 required chamber wall height.

The required chamber wall elevation was rounded to 24.5 ft in accordance with the procedures used in the 1973 MR&T Refined Flowline study. The adjacent wall height is based on the same criteria, with the exception that no structural superiority will be added to the adjacent walls, so it results in 22.10 rounded to 22.5 ft NAVD88 (2004.65).

4.8.2 Consideration of Sea Level Change in Plan Formulation

An alternate design for the chamber wall height is based on the USACE Intermediate Sea level change scenario for the end of the assumed project design life in year 2132, using the average rate of historical change specified in Table 11. Note the 1973 MR&T Project Flood Flowline was computed using the published value of sea level available at that time, which was based on the 1941-1959 national tidal datum epoch, and thus pertains to a midpoint year of 1950. Updating this to the project base year requires adding eustatic sea level change from 1950 to 2032^^. Eustatic sea level is added, without including land subsidence, because the flowline is referenced to a geodetic datum, which does not subside. Once the project is constructed, at that point the built infrastructure will subside, and relative sea level change should be considered from that point onward. The global 20th century average eustatic sea level change rate is 1.7 mm/yr per ER 1110-2-8162, so that rate was used to compute eustatic rise over this period.

- 17.30 MR&T Project Flood Flowline elevation referenced to NAVD88 (2004.65)

- Adjustment for eustatic sea level change from 1950 to 2032^^: 1.7 mm * 82 years = 139.4 mm ~ 0.5 ft

- Plus 5.7 – 1.3 = 4.4 ft of intermediate relative sea level rise between 2032^^ and 2132

- Plus 2.00 ft structural superiority[1]

- 24.2 ft required chamber wall height.

^^ Base year and end of period of analysis were revised to year 2034 and 2084 respectively, after this analysis was performed.

This design is approximately equal to the design based on MR&T criteria and is lower than the MR&T design after the MR&T design is rounded up to the next half-foot. Therefore, the design

[1] Structural superiority is commonly added to ensure structures are resilient over the project service life. It is included in the design of floodwalls and other features that cannot be easily reconstructed in the future to mitigate in uncertainty in model results and sea level rise estimates.

including sea level change does not govern plan selection, and the selected plan can be expected to perform for the standard 100-year design life for major infrastructure, under the assumption of intermediate sea level rise used for the purposes of plan formulation. If future sea level rise actually trends closer to the high scenario, then project performance will be affected as detailed in the following section. However, it should be noted that applying relative sea level change from 2032^^ to 2132 (and eustatic rise from 1950 to 2032^^) is a conservative assumption, because the riverine flowline will not rise as fast as mean sea level. Driessen and van Ledden (2013) found that under high river conditions (such as those that pertain to the project flood flowline), a given relative increase in sea level in the GOM would be reduced by more than 50% at a distance of 150 km up the Mississippi River, less than the distance from the GOM to this project site. Reducing the relative increase from the USACE High scenario from 2032^^ to 2132 by 50% from 9.6 to 4.8 ft, and the eustatic rise between 1950 and 2032^^ by 50% from 0.5 to 0.25 ft again results in an elevation comparable to the MR&T design.

Sea level change can also be expected to affect water levels in Lake Pontchartrain and Lake Borgne, and thus could affect plan selection via impact on the lock tailwater. However, in the absence of a hurricane storm surge, even the 11+ ft of relative rise projected under the high scenario (plus ~0.4 ft to adjust local mean sea level to NAVD88) would not exceed the 12.5 ft elevation used for design on the canal side of the lock. During a hurricane surge, the canal will be closed off from the GOM by the operations of the Lake Borgne Surge Barrier and the Seabrook Complex, which are intended to limit interior water surface elevations to a height of 8.0 ft. Sea level rise will increase the initial water surface before these structures can close and will increase the overtopping volume and ultimate interior water level in the canal during a storm. Nevertheless, interior water surface elevations are limited to the overtopping elevation of the interior walls at elevations of 12.2 to 12.4 ft, so even in this extreme case the interior design is not impacted enough to affect plan selection.

4.8.3 Effect of Sea Level Change on Plan Performance

In contrast to plan selection, sea level change can be expected to affect the performance of the selected plan in at least five ways:

1. During normal hydrometeorological conditions, sea level change will raise both the headwater on the Mississippi River as well as the tailwater in the IHNC. The increase in stage in the IHNC, however, will be faster due to the attenuation effect of the river slope discussed earlier. This effect reduces the head across the lock, reducing load as well as lockage time, so it represents a beneficial effect or opportunity due to climate change.
2. During a river flood, RSLC raises the MR&T Project Flood Flowline and increases the chances of overtopping the lock during a flood. This impact would occur when the existing flowline is elevated by 7.2 ft, from the existing 17.3 ft to the height of the proposed lock at 24.5 ft. This amount of increase would be expected to occur as soon as the year 2114 (0.5 ft of eustatic rise from 1950-2032 plus 6.7 ft of relative rise starting in 2032) under the USACE High Sea level scenario (it does not occur until after the end of the 100-year project life under the other scenarios). While this eventuality does represent a vulnerability to project performance, it should be noted that assuming that the Mississippi River will experience sea level change at the same rate as the sea is quite

conservative. In reality, the effect of sea level change is attenuated with distance upriver by an uncertain degree (estimated at 50% as detailed above). As a result, this impact of sea level can be conservatively considered a potential vulnerability to be addressed via adaptation actions near the end of the project life, but more realistically an issue to be considered in the design of the next IHNC lock replacement after this one.

3. During a hurricane storm surge in Lake Borgne, RSLC increases the total water level in the IHNC. The IHNC Surge Barrier and Seabrook Complex will be closed during major storms, blocking most surge from entering the canal, but with a higher initial water level there will be less interior storage space for barrier overtopping, rainfall, and pumping. The overtopping volume at the Surge Barrier will also be greater due to higher total water levels at the HSDRRS perimeter. This is a threat to the interior IHNC walls, but a relatively minor issue for the lock. It will not overtop before the interior walls and will be stronger than them in terms of both hydrostatic load and vessel impact. If the lock does overtop in this situation, it simply relieves water from the canal to the river.

4. During a hurricane storm surge in the Mississippi River, the IHNC lock could overtop into the IHNC. If the IHNC Lock overtops with surge from the river and allows water into the IHNC interior, the performance of the IHNC interior walls could be threatened. This possibility is explored in detail below.

5. During major maintenance activities, the lock must be unwatered for inspection and repair. RSLC raises the water levels in the water bodies surrounding the lock, putting additional uplift pressure on the lock chamber, and reducing the amount of time when unwatering can be performed safely. The existing IHNC Lock was last unwatered in 2016 for approximately two months. Unwatering while the river stage is above elevation 8.0 ft causes uplift pressures that result in an unwatering factor of safety of 1.2, which is considered unacceptable. Thus, the lock must be re-flooded whenever the river is above elevation 8.0 ft, or whenever the water level in the lock piezometers is above 4.0 ft regardless of river stage. Historically, from 1935 to present the river stage remained below this level for about 75% of the year on average. The lowest stage in the average hydrograph is approximately 2.5 ft, meaning that 5.5 ft of sea level rise (equal to the Intermediate scenario in year 2132) would be sufficient to prevent the lock from being dewatered at any point during an average year. This is within the range of plausible future conditions (although it relies on the conservative assumption of applying sea level change scenarios to river stages without attenuation) according to the analysis in Section 4.7 but would not be expected until after approximately year 50. Possible adaptation options to keep the lock maintainable after that time exist, including a system of relief wells/pumps to reduce uplift pressure. Therefore, this effect is of relatively lower concern, at least until relatively late in the project life.

Of the impacts identified above, impact number 4 is of the greatest concern due to the potential for great economic and life safety impacts to the city of New Orleans should the HSDRRS cease to operate as intended (i.e. if residual flood risk to the project area were to exceed 1% per year). To investigate the risk of excessive overtopping from the river into the

IHNC, future conditions storm surge and wave simulations performed by the State of Louisiana Coastal Protection and Restoration Authority (CPRA, 2017) were used to estimate the effect of sea level change on overtopping rates. The CPRA (2017) analysis represented future conditions using an increase in still water elevation of 1.47 ft and a variable subsidence surface that averages approximately 2 ft for the HSDRRS interior and Mississippi River. Thus, these future conditions were taken to represent about 3.5 ft of relative sea level rise compared to existing conditions. The existing conditions sea level computation is not well explained in the CPRA report, so it was conservatively assumed to refer to the year 1992.5, the midpoint of the most recent National Tidal Datum Epoch, which spans 1983-2001 and represents sea level as presently published.

To assess changes in surge for the Mississippi River at IHNC Lock, a spatial window of area near the confluence of the river and the IHNC was chosen to mask surge results. This window extends from latitude 29.955 N to 29.96 N and from 90.0275 W to 90.028 W and encompasses four model output nodes from the CPRA results. The output for a 1% annual chance surge coincident with a 1% annual chance significant wave at these four nodes is summarized in Table 12, below. In the table, SWL, H_s, and T_s refer to still water level, significant wave height, and significant wave period, respectively. Elevations are in ft and wave periods are in seconds.

Table 12: Present & Future Conditions 1% Annual Chance of Exceedance (ACE) Storm Surge in Mississippi River near IHNC Lock

	Still Water Level (SWL) Present	Still Water Level (SWL) Future	Significant Wave Height (Hs) Present	Significant Wave Height (Hs) Future	Significant Wave Period (Ts) Present	Significant Wave Period (Ts) Future
Min	13.16	14.75	2.63	2.67	3.11	3.12
Max	13.16	14.76	4.44	4.38	3.74	3.7
Avg	13.16	14.758	3.83	3.82	3.53	3.52
Var	0	$1.875 * 10^{-5}$	0.527	0.476	0.066	0.056

Overtopping rates for these conditions were computed using the average values of SWL and H_s by application of the Franco and Franco (1999) equation for the overtopping rate of a linear wall:

$$Q_{wave} = 0.082\sqrt{gH_s^3}\exp\left(-(\mu_b - \sigma_b)\left(\frac{R_c}{H_s}\right)\left(\frac{1}{\gamma_b\gamma_s}\right)\right)$$

Where:

g: acceleration due to gravity
H_s: significant wave height
μ_b: mean b coefficient
σ_b: standard deviation of b coefficient
R_c: freeboard

γ_b: wave obliquity
γ_s: geometry coefficient

Values for the coefficients above were selected to match those in the HSDRRS Design Elevation Report. The resulting overtopping rates, for present and future conditions, were $2.8*10^{-4}$ ft^2/s and $6.2*10^{-3}$ ft^2/s, respectively. These very low rates can be attributed to the fact that the proposed lock chamber wall is quite tall, about 8.5 ft taller than the existing lock.

The 1% annual chance surge and wave results from the CPRA (2017) report were then linearly extrapolated with sea level and used with the Franco and Franco equation as described above to estimate overtopping under future sea level scenarios. These results are shown in Table 13, below.

Table 13: Effect of Relative Sea Level Rise (RSLR) on Overtopping Rate at IHNC Lock During 1% Annual Exceedance Surge/Wave in Mississippi River

RSLR (ft)	Overtopping Rate (ft^2/s)
0	0.0002
3.5 (CPRA Future Conditions)	0.0044
4 (USACE Low Scenario, year 2132)	0.0068
5.7 (USACE Intermediate Scenario, year 2132)	0.0333
11.3 (USACE High Scenario, year 2132)	Free surface flow

The results in Table 13 show that under the High Sea level scenario in the year 2132, still water level elevation in the Mississippi River during a 1% annual chance exceedance storm surge would be higher than the elevation of the lock, resulting in weir flow over the gates during the peak of such a storm. However, it bears repeating that this finding is based on the conservative assumption that future sea level rise in the Mississippi River 110 miles from the sea will be as rapid as at the coastline. This situation would be expected to occur somewhat later in the future.

In accordance with the "when, not if" approach to sea level vulnerability assessment, a threshold is needed for delineating how much sea level change would constitute a performance risk for this project. The Franco and Franco equation is used to estimate an instantaneous peak overtopping rate, so using it to estimate accumulated volume over time is somewhat conservative. According to the IHNC overtopping analysis report by Driessen and van Ledden (2012), the stage-storage relationship for the IHNC basin is approximately 1 ft of stage per 100 million cubic ft of storage. Thus, the expected overtopping rate from the river during a 1% surge and coincident 1% wave event under Intermediate Sea level rise scenario in 2132 would only be expected to raise interior stages by about two one-thousandths of a foot.

After evaluating five potential ways in which sea level change could impact project performance, it appears that performance of the lock is quite insensitive to sea level rise. The selected plan can be expected to perform as intended for its full project life, with the possible exception of a concern over uplift pressures during maintenance unwatering, which requires a more detailed investigation to describe fully. Even if these pressures do present a problem near the end of the intended project life, adaptation measures are available to keep the project performing for 100 years. The robustness of the selected plan is almost certainly the result of two factors: the conservatism of the MR&T design, which by providing 4.8 ft of freeboard allows substantial robustness to changing conditions, and the fact that sea level rise in the river 110 miles upstream of the coast can be expected to proceed more slowly, particularly under high river flow conditions, than at the sea itself. Even under the conservative assumption that sea level rise in the river will be as rapid as in the ocean, serious impacts are not expected until late in the project design life.

4.9 Climate Change Effects due to Altered Inland Hydrology

Engineering and Construction Bulletin 2018-14 ("Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects") requires a qualitative analysis of the effects of climate change on inland hydrology and how these changes could impact the vulnerability of civil works projects. The normal process for this analysis involves a literature review, a statistical analysis to detect nonstationarities in observed discharge data, a projection of future discharge data based on climate projections, and a vulnerability assessment based on several indicators appropriate to the project type. These analyses are not particularly relevant for the present project, so the following analysis is highly abbreviated.

The vulnerability of the IHNC Lock is influenced by the flow frequency of the Mississippi River, which receives hydrologic inputs from a vast area (approximately 40% of the continental United States). The projections for climate change over this area vary widely, with well-documented observed winter warming causing hydrological shifts in the river's headwater region of the Upper Midwest to a more muddled picture near the river's mouth, which lies in a transitional zone between areas that are anticipated to become warmer and wetter and those expected to become warmer and drier. While these uncertainties over the river's future flow patterns make the future flood risk on the river hard to predict, the flood risk to the lock is easy to predict due to its location relative to the Bonnet Carré Spillway (and other upstream floodways). These floodways operate as safety valves for floodwaters contained in the Mississippi River, so that the flood risk to the lock will only change insofar as the likelihood of exceeding the MR&T Project design flood, and therefore the capacity of each floodway, may change. That possibility is even more remote and more uncertain than the already large uncertainty around more "normal" flood events. Nevertheless, the operation of the MR&T floodways removes much of the uncertainty around the future behavior of the river in this area and therefore the potential effects of inland hydrology changes. There is little reason to perform an exhaustive literature search or nonstationarity analysis on such a highly regulated system.

The available tools for projection of future river flows and for vulnerability assessment based on those projections are likewise ill-suited for the unique siting of the lock. These tools operate on the spatial scale of the Hydrologic Unit Code (HUC) 4 watershed, whereas the Mississippi Basin comprises six HUC 2 watersheds, a much vaster scale. Furthermore, the projections would again be of little relevance to a regulated system downstream of several major diversion projects, which serve to mute any potential change or variability. In short, the Jadwin Plan that formed the basis of the MR&T design is by its nature a robust and resilient plan, as it provides "room for the river" that both mutes extremes and allows many opportunities to adapt to changes over time with relatively little negative impact.

4.10 Navigation Studies

In order for the team to verify other lock design elements and variables that will be refined after the Chief's Report: a physical model and a ship simulator model will be completed in the future detailed design phase. These models will not change the hydraulic inputs to the economic model, or the filling times and will be used to refine the design of the RP only.

Physical Model

The physical model will be used to compute Hawser forces, longitudinal and transverse forces acting on the mooring lines. The LOCKSIM model does not accurately compute these forces, therefore, a physical model is required to ensure the Hawser forces are within the acceptable safety range as outlined in the guidance prescribed in EM 1110-2-2602 and EM 1110-2-1604. The physical model will also be used to optimize the culvert configuration, and valve operations to improve hydraulic conditions in the chamber.

Ship Simulation Model

The ship simulation model will be used to verify navigability during construction. The model will be used to refine the construction sequence and determine the length of time required to navigate through the proposed construction reach. The ship simulator is manned by a pilot that controls the vessel engine and speed. The pilot maneuvers through the proposed construction scheme and records observations under different wind and speed scenarios. The results of this study will be used to revise the proposed construction bypass channel and recommend safety aids during construction which will be implemented to improve transit times.

Both models will be completed in the future detailed design phase or as soon as funding is available and documented in the Design Documentation Report (DDR).

4.11 Conclusions

Design criteria will be finalized in future detailed design phase. The best available data will be utilized to design the lock. Detailed designs for the replacement lock study will be completed during future detailed design and documented in the DDR that accompanies the Plans and Specifications.

5 Surveying, Mapping, and Other Geospatial Data Requirements

Results from the survey performed by an A/E in October 2016, for the USACE MVN Engineering Division, was used in developing the cross-sections (Annex 11) and quantities (Annex 8) for this GRR. This survey provided current data on the channel depths. The excavation quantities were recalculated using this survey (Annex 8). The survey may be found in MVN ProjectWise document management system under the title "16-130C-IHNC Barge Lock Channel Survey". This survey was performed using epoch 2009.55. It was converted to epoch 2004.65 to match the previous data and analysis performed on the study.

It should be noted that a 2014 survey was used to prepare the cost for the four GRR study plans (before the 2016 survey was available). The 2014 survey areas taken were limited based upon available funding. The cross section created from this survey did not incorporate recent site changes in the field. The specific area of concern was the east side of the channel near the existing water's edge. Additional surveys will be taken during future detailed design.

5.1 Datum

Horizontal Datum

All horizontal data used in the GRR are referenced to the North American Datum (NAD) 83 US Survey Feet.

Vertical Datum

All elevations used in the GRR are referenced to NAVD88 (epoch 2004.65), unless otherwise noted.

The 1997 Evaluation Report used NGVD 29 as the epoch. Therefore, a conversion was made for the current elevations.

NGVD29 (1985) conversion = +0.81:

So, add 0.81 to NAVD88 2004.65 to view data in NGVD 29 (1985).

Or subtract (0.81) to get from NGVD 29 to NAVD88 (2004.65).

6 Geotechnical

The IHNC Lock is undersized for current traffic needs. Recently, there have been several studies looking into different strategies for replacing the lock. These alternatives require a cellular cofferdam (temporary retaining structure) for construction of the cast-in-place lock and sector gates structures. For a project of this size, the cellular cofferdam approach is the best option in terms of cost and constructability. Jet grout will be used at the cofferdam to help maintain stability of the structure during excavation. USACE MVN, Engineering Division's Geotechnical Branch (EDG) performed design analysis on several items to develop ROM cost estimates. Listed below are items analyzed for the GRR, in addition to items that will be further analyzed during future detailed design. For the next design phase, Geotechnical Branch will further evaluate the current designs to further reduce project costs.

Items Designed/Analyzed for GRR:

-MR&T Floodwall stability near the existing lock
-Cofferdam size/type/configuration
-Cofferdam stability at the new lock
-Jet grout limits for cofferdam
-Pile capacity curves for 18-inch OD x ½ inch diameter pipe piles

Items to be Designed/Analyzed:

-T-wall stability analysis (on the west and east of the IHNC channel), both MR&T and LPV.
-New MR&T and LPV floodwall stability analysis (to replace existing floodwalls)
-Levee stability analysis
-Settlement calculations

The Geotechnical Addendum Design Report (Annex 2) takes an already established layout from a previous study and looks at several options for excavation depth and water load cases for an open cell cofferdam design. The geotechnical analysis used the IHNC Lock Replacement Cast-In-Place Cofferdam 95% Feasibility Level Design, USACE Contract No. DACW29-02-D-0008, TO 0002, dated September 2006.

Analyses on the cofferdam investigated failure modes such as sliding, tilting, overturning, bearing, interlock tension, and global stability.

The information in Annex 2 is to be used for cost estimating purposes. A more detailed design is to be performed in the future detailed design phase of the project.

Guidelines from Engineering Manual 1110-2-2503 were followed for design of open-cell cofferdam design. Stability design follows the most current version of the Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines. These analyses are summarized below as they relate specifically to this project.

1) Stability of cofferdam cell on eastern bank with excavation at El (-) 33.0 and the water level at El +5.0.
2) Stability of cofferdam cell on eastern bank with excavation at El (-) 27.5 and the water level at El +5.0.
3) Stability of cofferdam cell on eastern bank with excavation at El (-) 33.0 and the water level at El +3.0 with a 160-kip impact load.
4) Stability of cofferdam cell on eastern bank with excavation at El (-) 27.5 and the water level at El +3.0 with a 160-kip impact load.
5) Stability of cofferdam cell at southern end in the channel with excavation at El (-) 33.0 & water level at El +5.0.
6) Stability of cofferdam cell at southern end in the channel with excavation at El (-) 27.5 & water level at El +5.0.

7) Stability of cofferdam cell at southern end in the channel with excavation at El (-) 33.0 & water level at El +3.0 with a 160-kip impact load.
8) Stability of cofferdam cell at southern end in the channel with excavation at El (-) 27.5 & water level at El +3.0 with a 160-kip impact load.
9) Stability of western bank with excavation at El (-) 33.0.
10) Hand calculations of active and passive pressures for cofferdam cell with diameter of 61 ft and height of 95 ft.
11) Hand calculations for risk of overturning for cofferdam cell with diameter of 61 ft and height of 95 ft.
12) Hand calculation for risk against sliding of cofferdam cell with diameter of 61 ft and height of 95 ft.
13) Hand calculation for risk against bearing capacity of cofferdam cell with diameter of 61 ft and height of 95 ft.
14) Hand calculation for risk against tilting of cofferdam cell with diameter of 61 ft and height of 95 ft.
15) Hand calculation of vertical shear & interlock tension of cofferdam cell with diameter of 61 ft & height of 95 ft.

For more detailed information and analysis, refer to Annex 2.

7 Environmental Engineering

No detailed environmental engineering was performed as part of this appendix. For environmental impacts and other considerations, refer to the GRR Main Report and Environmental Appendix.

8 Civil Design

8.1 Embankment, Compacted Fill

Using the 2016 IHNC Lock survey, a final grade section at the New Lock was developed for cost estimating purposes. For backfill at the new lock, semi-compacted fill (granular sand fill) will be placed in the dry within the cofferdam cell up to 3 ft below El 5.0. After this elevation has been achieved, the granular fill will be capped with 3 ft of clay embankment, compacted fill.

Embankment and Compacted fill at the T-walls were estimated using recent survey sections.

8.2 Excavation Quantities

The quantities of material to be excavated has been calculated, using the results of the 2016 survey. A portion of the excavated material may be stockpiled for use as backfill once construction is complete. The remainder of the material suitable for aquatic disposal may be disposed in the Mississippi River. Material that is not suitable for aquatic disposal will be disposed in an approved solid waste landfill site outside of the project area. Areas were designated depending on the nature of the sediment and were defined as Dredged Material Management Units (DMMU). The following figure shows the designation and location of each DMMU:

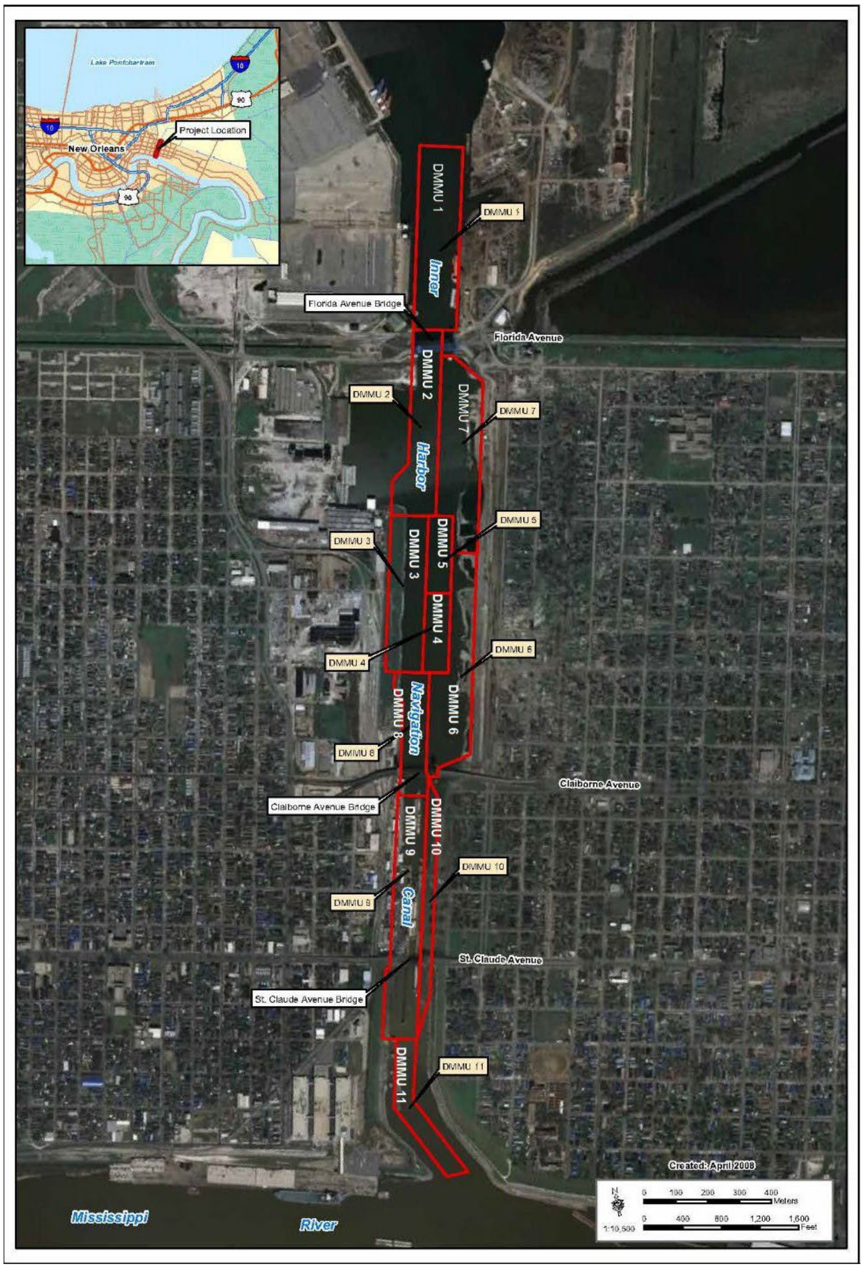

Figure 4: Dredged Material Management Units (DMMU) Designation and Location

Table 14 shows the quantities for excavated material. For additional details on the quantities, refer to Annex 8.

Table 14: Quantities of Excavated Material

Plan Description	New Lock Excavation & Bypass Channel Qty. (Cubic Yards, CY)	New Lock Excavation (Main Channel - DMMU's 3, 4, & 5), CY	Bypass Channel (DMMU's 6 & 7), CY	Excavation @ Existing Lock (DMMU's 9 & 10), CY	Total Excavation @ New & Existing Lock, CY
Plan 1- No Action	0	0	0	0	0
Plan 2 – 900 ft x 75 ft x Sill EL (-) 22.0	427,691	333,094	94,596	265,559	693,250
Plan 3 – 900 ft x 110 ft x Sill EL (-) 22.0 (TSP/ Recommended Plan)	**453,261**	**347,494**	**105,767**	**265,559**	**718,820**
Plan 4 - 1,200ft x 75 ft x Sill EL (-) 22.0	521,524	409,156	112,369	265,559	787,083
Plan 5 - 1,200ft x 110 ft x Sill EL (-) 22.0	553,885	426,207	127,678	265,559	819,444

Note: No calculation of the 1 ft of over dredge quantity for DMMU's 3, 4, & 5 was needed, because excavation will occur within the main channel.

The required excavation depth for the structure within the channel is El (-) 33.0, which includes the extra 1ft for the stabilization slab. The channel will be dredged to El (-) 22.0, and the bypass channel will be dredged to El (-) 17.0.

8.2.1 Excavation Quantities for the Recommended Plan

For the RP of 900 ft x 110 ft x sill El (-) 22.0, the following Table 15 shows the amount of material suitable for aquatic disposal that may be disposed of in the Mississippi River, and material that is not suitable for aquatic disposal that will be disposed in an approved solid waste landfill site outside of the project area:

Table 15: Quantities of Material Suitable for Aquatic Disposal and Material Not suitable for Aquatic Disposal

Material Classification	Quantities (CY)
Contaminated Material (DMMU 5 and 7)	104,909
Material to be Disposed in Mississippi River (DMMU 3, 4, 6, 9 and 10)	613,911

Total Excavation/New Lock/ Existing lock	718,820

8.3 Disposal of Material not Suitable for Aquatic Disposal

In past studies, material that is not suitable for aquatic disposal was analyzed. Due to the required shallow draft lock in lieu of the deep draft as previously studied, the amount of the material to be disposed of has been reduced. Based upon guidance from MVN, Environmental, DMMU 5 and 7 were not suitable for aquatic disposal. Two alternatives were considered for the disposal material not suitable for aquatic disposal: a confined disposal facility (CDF) and an approved solid waste landfill site outside of the project area.

The landfill used for the original analysis was located at:

Environmental Operators, LLC
339 Coast Guard Road, Venice, Louisiana
Tel: (504) 392-4619

Results of the 2016 survey were used to calculate the quantities that will need to be excavated for the plans. A rough order of magnitude (ROM) cost estimate was developed by MVN ED, using these quantities. The cost for the CDF alternative was higher than the approved solid waste landfill alternative. The analysis for the CDF alternative did not include costs related with Real Estate acquisition of the site, and future maintenance of the CDF. The analysis assumed the use of an environmental bucket (see note below) for the landfill alternative. The total amount of material that is not suitable for aquatic disposal from DMMU 5 and 7, for each plan is listed within the Table 16 below.

Note: Environmental dredging techniques aim to achieve a higher concentration of dredged sediment with the lowest possible turbidity. The technique is optimized via the precision with which dredging operations are performed. For instance, accurately removing thin layers of material, so that the least possible quantity of material is dredged, during removal of contamination. Therefore, less overall material is removed, as secure disposal sites for contaminated material can be a challenge to obtain.

The remainder of the material suitable for aquatic disposal may be disposed in the Mississippi River. Material that is not suitable for aquatic disposal will be disposed in an approved solid waste landfill site outside of the project area.

For the 2024 update, the River Birch Landfill was used for the cost estimate as Environmental Operators, LLC is no longer in business. The 2016 relative cost comparison is still valid. Ultimately, the construction contractor will have to select the state-licensed solid waste landfill for disposal meeting the contract requirements.

Table 16: Amount of Material to be Disposed of from DMMU 5 and 7 for Each Plan (2016 Surveys)

Plan	Total Required Material (From DMMU 5 & 7) (CY)
1- No action	0
2- 900 ft x 75 ft	93,013
3- 900 ft x 110 ft	104,909
4- 1200 ft x 75 ft	93,013
5- 1200 ft x 110 ft	104,909

The results of the cost analysis for material not suitable for aquatic disposal, for these two alternatives were as follows:

Table 17: ROM Cost Estimate for Landfill and CDF Alternatives (2016 Surveys)

Disposal of Dredged Material	ROM Cost Estimate ($)	Assumptions
Approved Solid Waste Landfill	$ 34,723,877	DMMU 5 and 7 to be hauled by barge to an approved solid waste landfill. DMMU 3, 4 and 6 to be disposed into Mississippi River.
Confined Disposal Facility (CDF)	$ 45,369,169	DMMU 5 and 7 to be pumped to the CDF. DMMU 3, 4 and 6 to be disposed into Mississippi River.
Cost Difference between Approved Solid Waste Landfill and CDF Alternatives	**$ 10,645,292**	

This cost was prepared using the 2016 survey. The 2024 update used River Birch Landfill for the cost estimate, but this comparison of alternatives is still valid.

8.3.1 Assumptions for the Approved Solid Waste Landfill Alternative

The analysis of the approved solid waste landfill alternative assumed the following:

a. An Environmental Bucket Dredge (see note in previous section) will be used to excavate DMMUs 5 and 7. This material will become the property of the contractor and hauled by barge to an approved solid waste landfill.

b. Material suitable for aquatic disposal (within DMMUs 3, 4, and 6), will be dredged via cutterhead dredge (hydraulic dredge).

c. The DMMU areas near the existing lock will be excavated via the environmental bucket dredge. Due to current construction sequencing, the bucket dredge is assumed to require a mobilization and demobilization twice.

8.3.2 Assumptions for the CDF Alternative

The CDF alternative did not include cost for Real Estate acquisition of the CDF site, or cost for future maintenance of the CDF. Assumptions made to prepare the estimate for the CDF are:

a. The cutterhead (hydraulic dredge) will excavate DMMUs 3, 4, 5, 6 and 7. The material excavated from 5 and 7 will be pumped into the CDF, approximately 5 miles away. Water treatment/handling of effluent is also included in the estimate. In order to dispose in the CDF, the entire CDF site (81 acres) will be cleared and grubbed. Construction of approximately 8,050 LF (linear ft) of retention dikes would be necessary to contain the dredged material and effluent. Treatment of the effluent would be necessary prior to discharge from the CDF site. The location of the CDF location, northeast of the proposed lock structure, is shown on Figure 5.

b. DMMUs 9 and 10, near the existing lock, will be bucket dredged due to the small quantity of material and the complexity/obstructions within the area. The excavation is primary land-based in lieu of open channel.

The detailed results of the cost comparison for disposal of material not suitable for aquatic disposal, may be found in Annex 7.

Figure 5: Confined Disposal Facility (CDF) Location

8.3.3 Recommendation for Disposal of Material not Suitable for Aquatic Disposal

It is the recommendation of MVN Engineering Division, seconded by the PDT, to proceed with the approved solid waste landfill only. The cost of the approved solid waste landfill alternative is over $10 million less than the CDF alternative. In addition, by not considering the CDF alternative any longer, mitigation, real estate, and future maintenance costs are eliminated. Overall, the landfill alternative offers less risks to the Government.

For further discussion regarding the disposal alternatives, refer to the GRR.

8.4 Alignment of St. Claude Avenue Bridge

Because it is a low-speed urban street, the design speed along St. Claude Avenue is 35 MPH. The speed limit across the proposed bridge is to remain the same. The new bridge is proposed to be placed north of the existing bridge deck, due to the necessity of keeping the existing bridge open during construction. Therefore, a series of horizontal curves will be needed to tie into the existing approach ramps to the proposed bridge deck. From the AASHTO Green Book, the minimum radius of curve at 35 MPH is 419 ft at the centerline, which is the radius of all four horizontal curves. See ASHTO 2014 Edition, Chapter 3, Table 3-17b: Super-elevation Runoff L, (ft) for Horizontal Curves, Table 3-13b: Minimum Radii and Superelevation for Low-Speed Urban Streets, and Table 3-34 Design Controls for Crest Vertical Curves Based on Stopping Sight Distance.

The proposed bridge deck elevation is (+) 39 ft, whereas the existing bridge deck elevation is approximately (+) 20 ft. However, the approach ramps must tie back to the existing tie-ins along St. Claude Avenue at both Poland Avenue and Reynes Street. So, the approach ramps are steeper in grade than the existing ramps, but with the addition of longer vertical curves, still suitable for traffic. Refer to Annex 14 for civil calculations of the bridge.

9 Preliminary Sill Elevation Assessment

A technical analysis was conducted during the GRR to determine the sill elevation for the proposed shallow draft lock. This assessment relied heavily on USACE Engineering Manuals (EM), and studies completed between 1997 and 2009.

9.1 Summary of Technical Considerations

Adequate Clearance for Design Vessel:

- Sill and chamber depths are set to prevent tows from striking the lock floor and permit reasonable entrance and exit times.
- Per EM 1110-2-1604, a sill depth that is 2 times the design vessel draft is required to allow for a safe entrance into and exit from the lock. The design draft for a fully loaded liquid tank barge is 11 ft.
- According to the 1997 Evaluation Report, a preliminary sill elevation was set at (-) 22.0, which will provide a water depth that is equal to or greater than 2 times the draft depth 95% of the time. This preliminary sill elevation will provide a water depth that is equal to or greater than 1.82 times the draft depth 100% of the time.
- Port of New Orleans personnel and representatives of the Gulf Intracoastal Canal Association (GICA) have stated that the expected maximum draft of vessels locking through the new structure is 12 ft, therefore this is the design draft. The preliminary sill elevation set at (-) 22.0 will provide a water depth that is equal to or greater than 1.67 times the maximum draft depth 100 % of the time. This satisfies the requirements of EM 1110-2-1604 and allows for safe operation of the lock.

Limiting Hawser Forces on Vessels during Lockage:

- Hawser forces are the stresses in the mooring lines during lockage. Per EM 1110-2-1604, this force cannot exceed 5 tons for a barge. Hawser forces can be reduced by increasing the distance between the bottom of the vessel and the lock floor or by increasing filling and emptying times.
- With a preliminary sill elevation set at (-) 22.0 ft, Hawser forces on the vessels can be limited to the allowable range and provide acceptable filling and emptying times.

Constructability:

The existing IHNC channel is maintained to El (-) 32.0. By setting the sill of the new structure at El (-) 22.0, minimal excavation or backfill will be required for construction of the new lock.

Recommendation:

Based on the findings of previous studies and requirements from USACE EMs, ED recommends proceeding with a sill elevation of (-) 22.0 ft in the GRR. This recommendation must be validated by a physical model during future detailed design.

This discussion was summarized in the memorandum from MVN Engineering Division, Structures Branch (USACE-MVN-EDS), dated April 2015 (Annex 10).

The option of a sill elevation of (-) 16.5 was not considered and was not fully analyzed to the same level of details as plans 1 to 5. Detailed cost for this option may be found in the following section. Details and assumptions may be found in Annex 3.

9.2 Shallower Sill Cost Analysis

The team completed a quantitative cost comparison (ROM level cost) between a sill elevation of EL (-) 22.0 and EL (-) 16.5 to validate the cost findings of the 1997 Evaluation Report, and to analyze cost savings for a shallower lock. Reference Annex 3 for cost analysis.

The comparison is provided for information only. The engineering recommendation is for a sill elevation at (-) 22.0. A shallower sill elevation may not satisfy the safety requirements for Hawser forces. The PDT consulted with the Inland Navigation Design Center (INDC), USACE's Mandatory Center of Expertise (MCX) for inland navigation, for concurrence on the sill elevation. The INDC representatives also recommended a sill elevation of (-) 22.0.

As shown in Table 18, the difference in costs between the two options is minimal.

Table 18: Cost Comparison for 1200 ft x 110 ft Lock with Sill Elevation (-) 22.0 and **(-) 16.5 (2016 Dollars)**

Sill Elevation	Lock Dimensions	Construction Cost with 45% Contingency (2016 dollars)
(-) 22.0 (Plan 1)	1200 ft x 110 ft	$ 908,174,826
(-)16.5	1200 ft x 110 ft	$ 891,933,400
Difference		**$ 16,241,426**

10 Cast-In-Place versus Float-In-Construction of the Lock

10.1 Design Summary

Float-In-Place Design

The preliminary design for a float-in-place lock structure was developed by URS Group, Inc. between 2003 and 2009. The design consisted of five (5) precast, pre-tensioned concrete monoliths which would be built at an offsite graving site. These modules would be floated through the IHNC to the proposed location of the replacement lock. The monoliths would be set down on 48-inch diameter steel pipe piles with sand ballast and tremie concrete.

Cast-In-Place Design

The proposed design for a cast-in-place lock structure was developed by the USACE New Orleans District between 2004 and 2007. The design consisted of seven (7) lock monoliths founded on 24-inch square precast concrete piles. Conventional construction techniques, including sheet pile cofferdams, unwatering systems and cast-in-place concrete would be utilized.

10.2 Comparison of Technical Approaches

Design Quality

Design of the float-in-place structure will require marine design experience and will rely heavily on lessons learned from previous USACE projects. The use of innovation and new technologies associated with float-in design invariably involves a steep learning curve and appropriate model development and testing, which are associated with high costs.

Construction Execution

- The float-in-place structure will require two separate construction sites – one at the new lock structure and one at the graving site. Concurrent construction will be possible at the two sites.

  Transportation and set down will be affected by weather and water levels in the canal.

  Float-in-place design may receive a high level of scrutiny from USACE Headquarters. Per the Olmsted Locks and Dam Project - Frequently Asked Questions, "*Experience has shown that the current in-the-wet construction method is more expensive and time consuming than originally envisioned and Corps' headquarters has directed us to build a team of regional and national experts to consider alternative construction techniques such as building cofferdams for in-the-dry construction*."

- The cast-in-place structure requires one construction site, but the size of this site is constrained by existing structures. Construction activities at the site are linear, with limited opportunity for concurrent work.

Cofferdam overtopping is possible during storm events.

Navigation Issues

- The float-in-place design requires a bypass channel to maintain navigation on the canal. Helper boats will be required to assist traffic through the bypass. The IHNC will need to be completely shut down during transportation of the five lock modules.

- The cast-in-place design requires a bypass channel to maintain navigation on the canal. Due to the increased footprint of the cofferdam, the bypass channel will be narrower under this approach. Additional vessel simulation is required to confirm alignment is navigable.

Contractor Expertise

- Transportation, alignment and set down of floating units will require a marine specialist. The local market has little experience with "in-the wet" construction techniques, which will increase the cost of a float-in-place design.

- There is a large local and national pool of contractors capable of performing the construction work associated with the cast-in-place design.

Biddability

- Historically, "in-the-wet" construction projects have generated limited interest from contractors due to the significant risks associated with this construction technique. The Olmsted Locks and Dam project received no bids when advertised as a firm fixed price contract. In order to be awarded, the contract was changed to a cost-plus contract, placing all risks on the Government, and ultimately resulting in significant cost overruns.

- The Harvey Canal Floodgate project was redesigned and awarded as a cast-in-place structure based on feedback from potential bidders regarding the float-in-place structure.

- There is a large local and national pool of contractors capable of performing the construction work associated with the cast-in-place design.

Cost

- A significant economic advantage with the innovative float-in-place construction method comes from its speedy project delivery, assuming that the project will be fully funded. If the contract is underfunded, any perceived cost savings associated with innovative construction will be eliminated. Specialized construction equipment is often needed for the innovative construction technique, further raising costs. A cost comparison performed by USACE Lakes and River Division (LRD) in 2007 showed that the float-in-place was more expensive to construct than the cast-in-place design for a deep draft lock, with decreases in the high-cost items for the cast-in-place design.

- The cast-in-place design features more standardized construction methodologies, providing greater confidence in cost estimates due to less unknowns and risk in construction.

Community/Environmental Impacts:

- The float-in-place design reduces the impact to the community by eliminating cofferdam construction in the channel and a significant portion of the cast-in-place concrete through the offsite construction of the float-in modules. The graving site will require mitigation of wetlands and additional environmental impacts.

- The cast-in-place design features more standardized construction methodologies, providing greater confidence in cost estimates due to less unknowns and risk in construction.

10.3 Recommendation

Based on this assessment and previous studies, it is recommended that only the cast-in-place design is further investigated in the GRR. Once completed, the cast-in-place and float-in-place designs would function identically. However, as shown in this section, when compared to the float-in, the cast-in-place design presents less chance for cost escalation and schedule delays due to unforeseen design and construction challenges.

This recommendation is based on the USACE experiences with Olmsted Locks and Dam and the Harvey Canal Floodgate. During discussions with the INDC, they have expressed the same concern with in-the-wet construction. For a more detailed discussion on the comparison between float-in-place and cast-in-place designs, refer to the 2007 Letter Report prepared by LRD.

11 Sector Gates versus Miter Gates

Considerations that set the two gate types apart are:

- The existence of a reverse head at various times of the year
- Durability
- Gate geometry
- Gate bay monolith geometry
- Construction time
- Culvert maintenance
- Overall costs

11.1 Reverse Head

Miter gates are not designed to operate against a reverse head. To deal with this condition, either a second set of gates must be installed, or the lock must be shut down for the duration of the reverse head. Since the latter is not an option, a second set of gates must be included along with the appurtenant machinery. To accommodate the second set of gates, the gate bay monolith must be lengthened accordingly.

A single set of sector gates, by design, can handle both a direct head and a reverse head without the need for more gates or more machinery.

11.2 Durability

Miter gates do not stand up to damage as well as sector gates. Additionally, if a miter gate leaf is damaged such that there is a flow of water into the chamber, flooding of the downstream side could occur and/or the undamaged gates cannot be operated. Consequently, an estimated $1.4 million emergency crane must be provided to lift the emergency bulkheads into place to stem the flow.

Sector gates, on the other hand, have no problem operating against a flow, and can thus temporarily stem the flow until the emergency bulkheads can be placed. Since placement of the emergency bulkheads is no longer imperative, the emergency crane can be eliminated.

11.3 Gate Geometry

Miter gates are routinely built for the rough channel geometry of 110-ft wide by 46.5-ft tall. Sector gates have also been recently constructed for similar heights and widths as part of the HSDRRS.

Additionally, the larger couple (distance between pintle and hinge) distance greatly reduces thrust on the hinge and pintle and large main chords, minimizing deflection. Gate deflection due to dead load is mostly cambered out during fabrication. Wheels and flotation tanks were considered in the preliminary design but discounted.

11.4 Gate Bay Monolith Geometry

The miter gates' gate bay monolith is rectangular in shape, roughly 180-ft wide by 436-ft long. The sector gates' gate bay monolith is 218 ft, 8-inch long by 320-ft wide. The additional 70 ft of width of the sector gate bay monolith affects the width of the cofferdam and the navigability of the bypass channel. However, the original clearance between the limits of the bypass channel and the edge of the gate bay concrete of 110 ft has been reduced to 40 ft. This distance is more than sufficient, so no adjustment to the bypass channel would be necessary.

11.5 Construction Time

The construction duration for the sector gate is included within Annex 3 of this appendix. No design and construction duration for the miter gate was included within the GRR.

11.6 Culvert Maintenance

Miter gates cannot operate without culverts to fill and empty the chamber. Sector gates, on the other hand are capable of filling and emptying the chamber in a sufficient time frame when there is a small head differential. Although the sector gate was shown to need the culverts for high head differentials, the gates are capable of operating without them during periods of low differentials. Being able to end-fill during those times would enable the culverts to be unwatered for maintenance without closing the lock to marine traffic.

11.7 Costs

Excluding spare gates for either option, the sector gate option is roughly $12 million less than the miter gate option.

11.8 Conclusions

A detailed analysis of the sector versus miter gate is presented in Annex 5. The analysis includes details on a 1997 physical model study, structural, naval, chamber and foundation designs, and cost comparison.

When assessing the sector gates versus miter gates, the costs turned out to be roughly comparable. In light of its preference for a sector gate structure, and the data presented in Annex 5, USACE New Orleans District, Operations Division (CEMVN-OD) elected to pursue the sector gate option. In addition, CEMVN-OD authorized the elimination of the emergency bulkhead crane.

12 Structural Requirements

12.1 Design of Lock Structure

No detail design for the lock was completed at this stage of the study. Lock quantities were prorated using the existing quantities and design data as presented within the 1997 Evaluation Report with limited pile foundation design for the gate bays and T-walls along with limited feasibility design of the St. Claude Avenue Bridge. Detail design of the lock structure will be performed during future detailed design phase.

A map of the proposed new lock is included in Figure 6. For the 900-ft lock plan (the RP), the lock would be located approximately 2,400 ft north of the existing lock structure. The proposed location is the same as shown in previous reports. A box culvert size of 14.5 ft x14.5 ft was selected for the 900-ft long chambers. In addition to the sector gate and lock, a bypass channel, cofferdam, 3 pile dolphins, timber and floating guide walls, and concrete floodwalls will be constructed at this location. The new floodwalls will tie into the existing walls on the west and east of the IHNC channel. This location was primarily chosen due to its ease of access and little to no obstructions located near the open channel. The physical features associated with the construction of the new lock structure are:

The associated lock features are:

- Chamber Concrete Monoliths/Pile Foundation
- Sector Gate Monoliths/Pile Foundation
- Steel Sector Gates
- Timber Guide walls
- Floating Concrete Guide walls
- End Cell Dolphins
- Cofferdam
- Floodwalls
- Maintenance Bulkheads
- Maintenance Bulkhead Storage Platform
- Culvert Roller Gates
- Culvert Bulkheads
- Permanent Mooring Cells

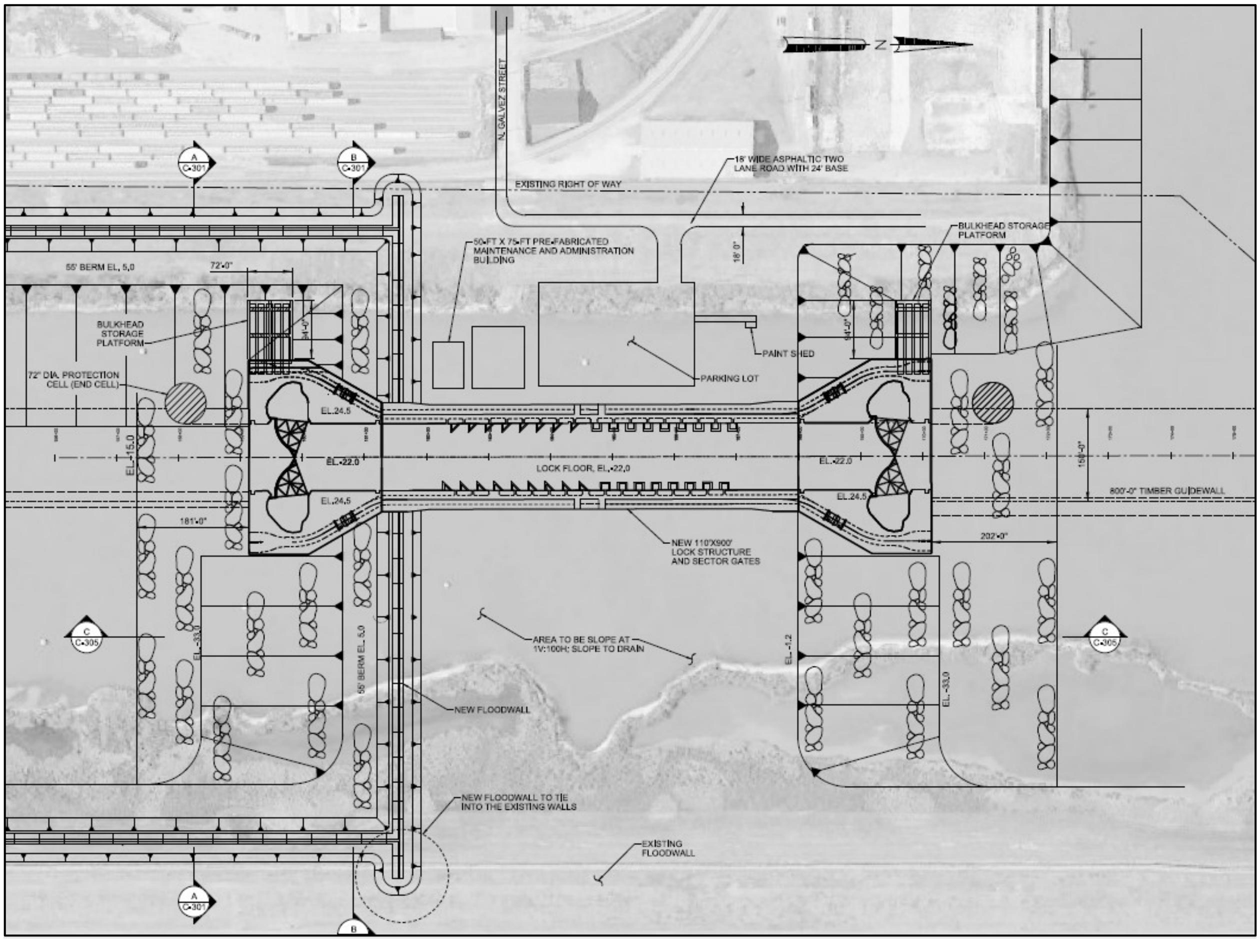

Figure 6: New Lock Plan View

All design is in accordance with applicable USACE engineering guidance and applicable industry standards.

12.1.1 Technical Publications

- American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary (ACI 318-14).
- American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, 9th Edition.

12.1.2 USACE Publications

- Hurricane and Storm Damage Reduction System Design Guidelines, New Orleans District, 12 June 2008.
- EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures Change 2 March 2001.

• EM 1110-2-2104 Strength Design Criteria for Reinforced Concrete Hydraulic Structures, November 2016.
• ETL 1110-2-584 Design of Hydraulic Steel Structures, June 2014.
• EM 1110-2-2503 Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures, September 1989.
• EM 1110-2-2703 Lock gates and Operating Equipment, June 1994.
• EM 1110-2-2906 Design of Pile Foundations, January 1991.
• ER 1110-2-8152 Planning and Design of Temporary Cofferdams and Braced Excavation, August 1994.

12.2 Chamber Monoliths

The layout and geometry for the lock chamber monoliths were taken from previous 2007 Report. Similar to the sector gates, the lock sill elevation was raised to El (-) 22.0. All costs related to the lock chamber were determined using sizes from the 2007 report and prorating based on based on new hydraulic conditions. The lock chamber monoliths will be cast-in-place concrete in lieu of float-in-place concrete. This will facilitate an easier method of construction within the 3-sided cofferdam as discussed within Section 12.8. The lock will utilize a 10-ft thick base slab. Wall thickness will be 4 and 6 ft at the top and bottom respectfully.

The pile foundation for the new lock chamber will utilize 24-inch prestressed concrete piles. No detailed design analysis was performed for this GRR. The lengths of the piles were determined using the weight of concrete and weight of water (vertical and uplift). Loads under the culverts and chamber walls were evenly distributed amongst the 10 piles underneath those sections of the chamber monoliths and resulting tip elevation of EL (-) 126 selected thereafter. A factor of safety (FOS) of 2.0 was used in determining the pile loads and tip elevation. For detailed quantity calculations, refer Annex 8. During future detailed design, alternative pile types and construction methodologies for pile driving will be investigated to mitigate noise impacts in the adjacent communities.

Table 19 shows the pile quantities for each plan:

Table 19: Pile Quantities – 24-inch Prestressed Concrete Piles

Plan	Area	Number of Piles	Total Pile Quantity
5 (1200 ft x 110 ft)	Wall	792	79,200
	Chamber	900	94,050
2 (900 ft x 75 ft)	Wall	552	55,200
	Chamber	414	43,470
4 (1200 ft x 75 ft)	Wall	792	79,200
	Chamber	594	62,370
3 (900 ft x 110 ft)	Wall	552	55,200
	Chamber	690	65,550

Table 20 provides the total quantity of concrete for each plan:

Table 20: Concrete Quantity- Lock Structure	
Plan	**Total Quantity (CY)**
5 (1200 ft x 110 ft)	98,442
2 (900 ft x 75 ft)	57,301
4 (1200 ft x 75 ft)	85,669
3 (900 ft x 110 ft)	66,185

12.3 Sector Gate Monoliths

Sector gate monoliths will be constructed at the river side and lake side of the new lock structure. Both sector gate monoliths will serve as the mainline protection and for flow control between the Mississippi River and GIWW. The sector gates will be constructed within the same cofferdam as the chamber monoliths.

The sector gate pile foundation was analyzed using Case Pile Group Analysis (CPGA). Similar to the 2006 CIP vs. FIP Evaluation Report, 24-inch square concrete piles were used in the foundation. A base slab thickness of 13 ft was used in developing pile loads for the sector gate (a thinner base slab will be evaluated during future detailed design. The load cases analyzed to develop pile loads are shown below in Table 21. Note only usual and unusual loads were investigated during this GRR. Extreme load conditions will be investigated during future detailed design. The pile lengths and quantities for the three (3) separate lock configurations are as shown below in Table 21:

Table 21: Load Cases for Sector Gate Monolith Pile Foundation	
1a	Operation, Maximum Direct Head – Gates Closed
1b	Operation, Maximum Direct Head – Gates Open
2a	Unusual Operation, Maximum Direct Head Plus Freeboard - Gates Closed
2b	Unusual Operation, Maximum Direct Head Plus Freeboard - Gates Open
3a	Unusual Hurricane Plus Freeboard – Gates Open
3b	Unusual Hurricane Plus Freeboard – Gates Closed
4	Maintenance Unwatering
5	Unusual Maintenance
7a	Normal Operation - Gates Closed
7b	Normal Operation - Gates Open

Refer to Annex 12 for preliminary design load cases.

Based upon the varying lock sizes and geometry, three (3) separate CPGA analysis were performed for the sector gate pile foundations:

Case 1: Revise Sill Elev. from (-) 40 to (-) 22.0. (All other attributes are the same).
Case 2: Revise Sill Elev. from (-) 40 to (-) 16.5 with box culverts sized at 14.5 ft x 14.5 ft.
Case 3: Revise Sill Elev. from (-) 40 to (-) 22.0; 110 ft Gate Bay to 75 ft; Box Culverts remains the same 18 ft x 14.5 ft.

Table 22: Pile Quantity for Sector Gates				
Case	**Number of Piles**	**Length (LF)**	**Total Length per Monolith (LF)**	**Total for both for Both Monoliths (LF)**
1	933	120	111,960	223,920
2	933	120	111,960	223,920
3	817	120	110,295	220,590

12.4 Steel Sector Gates

The steel sector gate quantity was prorated using the previous total gate weight, 2,128,000 lbs, taken from the 2006 CIP vs. FIP Evaluation Report. The gate opening had a width of 110 ft with a top of wall (TOW) El 23.0 and Sill El (-) 40. The sill elevation for the new shallow draft is El (-) 22.0 with a TOW El 24.5. The quantity was prorated using the total hydrostatic head acting on the gates using the Maximum Direct Head Load Case of El 17.3 R/S and El (-) 0.8 L/S. The prorated gate weights are as shown in Table 23:

Table 23: Steel Gate Weights
110 ft x Sill El (-) 22.0 = 1,335,000 lbs
110 ft x Sill El (-) 16.5 = 1,068,000 lbs
75ft x Sill El (-) 22.0 = 908,000 lbs

12.5 Timber Guide Walls

The cost for the timber guide walls were prorated using the quantities from the 2007 Report. Previously, the 800-ft long layout and configuration for the timber guide walls was shown within the drawings from the 2007 Report. They were to b e placed only on the lakeside of the structure. Timber guide walls were previously selected because of the minimal pool differential on the lakeside. The material is currently used along the lakeside guide walls of the existing lock and have performed satisfactorily. Since the previous report, EM 1110-2-3402, Barge Impact Forces for Hydraulic Structures has provided a shift in criteria and guidance for impact loads. Due to this change, the timber guide walls were removed from the plan and replaced with fixed concrete guide walls as described in Paragraph 12.6. For a detailed breakdown of the revised quantity, refer to Annex 8.

12.6 Concrete Guide Walls

The cost for the floating concrete guide walls were prorated using the quantities from the 2007 Report and from recently awarded projects. The 1,260-ft long layout and configuration for the floating guide wall will be the same as shown within the drawings from the 2007 Report. The floating guide wall will only be required on the river side, along the east wall of the IHNC channel due to the large pool differential associated with the Mississippi River. A similar floating guide wall has been in service on the southeast side of the existing lock and performed satisfactorily. An 800-foot fixed concrete guide wall will be utilized on the lakeside of the structure. An end cell will be used at the end of the guide wall to prevent direct impact at the nose of the structure. During future design, further refinement of the guide wall material type and design will be performed to optimize construction and life cycle costs. For a detailed breakdown of the revised quantity, refer to Annex 8.

12.7 End Cell and Bullnose Dolphins

The cost for the end cell dolphins were based on the quantities developed from the 1997 Evaluation Report. The end cells consist of a cellular sheet pile in-filled with sand and capped with concrete. A total of four (4) end cells will be constructed, one at the end of each guide wall on the four corners of the lock structure. The concrete bullnose dolphins are necessary to mitigate barge impact to the bridge piers, lock structure, and guide walls. There will be a total of 4 bullnose dolphins at the new St. Claude Bridge, and a total of 4 bullnose structures at the new lock. The location of the bullnose structures is shown on Plan Sheets C-101 through C-103. The existing northeast end cell dolphin and 60-foot-long timber guide wall will be demolished as part of the existing lock demo and replaced with the bullnose structures. The bullnose dolphins are U-shaped with six-foot thick parapet walls and six-foot-thick pile founded slabs. The new dolphins will be standalone structures with vertical pipe piles (26 per structure) such that interference with the adjacent IHNC Lock structure, forebay, and other construction is mitigated. Based on historic hydrograph data at IHNC Lock, the structures will be constructed one foot below the lowest still water elevation, which will mitigate corrosion in the steel pipe piles. The proposed dolphin on the northeast corner will approximately match the top of sheet pile elevation of the existing end cell dolphin at EL 13.0 and will be made accessible by stairs connected to the forebay of the IHNC lock. The southwest dolphin will have a top of concrete at EL 24.5.

12.8 Cofferdam

While constructing the cast-in-place lock structure, a 3-sided cofferdam will be required to excavate the existing IHNC channel to required grade El (-) 33. The layout for the cellular cofferdam layout was taken from the 2006 CIP vs. FIP Evaluation Report. The east-to-west section of cofferdam cells will tie-into the existing levee on the west side of the channel. Upon completion of the lock and sector gate, the inside of the cofferdam will be backfilled, and the cofferdam removed.

For the GRR, MVN Geotechnical Branch performed an analysis to develop the size and length (61-ft diameter, 95-ft long PS-31 sheet pile) for the cellular cofferdam.

Jet grout columns will be required at the bottom of the cellular cofferdam to provide adequate stability of the structure. Approximately, a total of thirty-four (34) circular cofferdam cells will be used for this project.

12.9 Floodwalls

The floodwalls for this project will be constructed to El 24.5 to match the required top of wall (TOW) for the lock. Since the floodwalls will be constructed along the IHNC channel, which is subject to high barge traffic, a 500-kip barge impact force will be used for design.

The new IHNC floodwalls will extend from the Mississippi River on both east and west sides of the IHNC channel, and tie into the new lock. The length of floodwall assumed for the GRR was approximately 6,480 LF (162 floodwall monoliths, at 40-ft lengths).

Table 24: Total Number of Floodwalls
West Side of IHNC Channel
Total Length of Floodwalls = 4,520 LF Total Number of 40-ft Floodwall Monoliths = 113
East Side of IHNC Channel
Total Length of Floodwalls = 1,960 LF Total Number of 40-ft Floodwall Monoliths = 49

Note: Floodwall lengths were measured from a CAD file.

For the RP, a site-specific T-wall design was performed. For the detailed calculations for the T-wall design, refer to Annex 13.

Portions of the existing I-wall and T-wall sections along the IHNC channel require demolition as part of this project. In addition, the existing concrete scour protection will be demolished and replaced with compacted fill embankment. New concrete scour protection will be added after re-grading of this area. The total cubic yards (CY) for demolition of both existing floodwall and scour protection is approximately 13,000 CY.

As part of the new floodwall construction, new T-wall monoliths are required underneath the existing St. Claude Avenue and Claiborne Avenue bridges.

The T-walls required under the St. Claude Avenue Bridge will be coordinated with the new bridge construction. To further note, T-walls can be constructed up to the proposed bridge location, prior to removal of the existing bridge. After the existing bridge has been removed, the T-walls can be tied into the new line of protection.

12.10 Maintenance Bulkheads

For the El (-) 40 sill alternative studied within the 2005/2006 Feasibility Report, the steel bulkheads utilized seven (7) lower bulkheads, and four (4) upper bulkheads. The steel bulkhead structures were prorated based upon the difference in head elevation for the new shallow draft requirements. Based on the higher sill elevation, the total number of bulkheads were revised to three (3) lower bulkheads, and four (4) upper bulkheads. No structural design calculations were performed for developing costs for this GRR. More detailed designs for total gate weights will be provided during future detailed design.

12.11 Maintenance Bulkhead Storage Platform

No structural design calculations were performed for developing costs for this GRR. Quantities were pro-rated from previous designs. More detailed designs (for total gate weights) will be provided during future detailed design. For a detailed breakdown of the revised quantity, refer to Annex 8.

12.12 Culvert Roller Gates and Bulkheads

No structural design calculations were performed for developing costs for this GRR. Quantities from the 1997 Evaluation Report were used for the culvert roller gates and bulkheads.

12.13 Permanent Mooring Cells

No structural design calculations were performed for developing costs for this GRR. Quantities from the 1997 Evaluation Report were used for the culvert roller gates and bulkheads.

12.14 Electrical and Mechanical Requirements for Lock Structure

At this stage, no new electrical and mechanical components will be developed for the sector gates and the lock. For cost purposes, the electrical and mechanical were prorated using the cost per each from the 2023 Colorado-Brazos Locks Feasibility Study. Several sector gates structures have been designed and/or constructed within the HSDRRS. For cost estimating purposes, typical electrical and mechanical components from those designs were utilized for this report.

The mechanical and electrical service will be sized to support the structure loads including power for gate machinery, lighting, controls, and any other miscellaneous loads.

12.15 St. Claude Avenue Bridge Replacement

In 2019, a limited feasibility design was performed for the St. Claude Avenue Bridge Replacement feature of the RP. As previously mentioned, the temporary bridge was eliminated following the ADM; and a permanent bridge was proposed north of the existing St. Claude Avenue Bridge. The horizontal and vertical curvature of the bridge was designed for a posted speed limit of 35 mph. The curvature was adjusted to minimize impacts to the nearby community. Refer to Section 8 for further details on the geometrical layout of the bridge.

The new bridge is 70 ft wide with two (2), 12 ft wide eastbound lanes and two (2), 12 ft wide westbound lanes. Four (4) foot shoulders are provided on the outside and minimum one-foot shoulders are provided on the inside. A 6-ft wide pedestrian/bicycle lane is provided on the outside edge of the eastbound lanes, separated by traffic with a concrete barrier. A 7-inch reinforced concrete slab/deck was preliminary sized for the bridge approaches. Based on a proposed 7-ft 3-inch spacing between girders and typical 80-ft span between approach pier bents, an AASHTO Type III precast prestressed concrete girder was selected to support the approach decks. A STAAD model was developed to size the pier cap, pier columns, pile cap and pile tips. Eighteen-inch steel pipe piles were assumed to support the approach piers. Pile capacity curves used for the floodwalls were utilized for the pile tip selection. For the actual bascule bridge steel spans and the bascule concrete piers, quantities were pro-rated based on recent designs for those components. Bascule spans were selected to span the existing/future channel alignment and the temporary bypass channel alignment during demolition of the existing lock. For the detailed calculations for the T-wall design, refer to Annex 13. The bascule bridge type was selected based on previous studies performed. During future detailed design, additional engineering will be performed to ensure the bridge that combines cost and schedule efficiency with minimized impacts to the surrounding community is constructed.

12.16 Control House, Maintenance & Admin Building, Machinery Building w/ Generators, Paint Shed

During the future detailed design, USACE ED will design and layout in further detail the various buildings to be constructed as part of the new lock. ED will work with MVN-OD to determine the logistics and suitable locations for the proposed structures. The structures and buildings include:

1) Control House
2) Maintenance & Administration Building
3) Machinery Building with Emergency Generators
4) Paint Shed

These buildings (and additional associated structures and facilities to be determined necessary during future detailed design are required to maintain operation and functionality of the lock. In addition, site access roads will be constructed for ingress and egress to these structures. The maintenance and administration building, parking lot, and access roadway is shown on Annex 11, Plates Sheet C-103.

13 Hazardous and Toxic Materials

At property previously owned by the Port of New Orleans and previously occupied by the U.S. Coast Guard located on the west side of the IHNC, there are two sites that have been identified through prior HTRW environmental site assessment investigations where contamination is known to exist. Sampling at these two sites indicated that total petroleum hydrocarbons as diesel, total petroleum hydrocarbons as oil, and some polycyclic aromatic hydrocarbons (benz(a)anthracene, benzo(a)pyrene, benzo(k)fluoranthene, indeno(1,2,3-cd)pyrene, and benzo(a)pyrene) remained at elevated concentrations in both areas (including under a diesel aboveground storage tank). The property was acquired in fee by USACE for the lock replacement project in 2001. The Louisiana Department of Environmental Quality (LADEQ) has determined that these sites must be remediated.

While the actual construction of the IHNC lock replacement facilities will not disturb these areas, the realignment of the MR&T and LPV floodwalls possibly could disturb the sub-surface contaminated material that is situated beneath approximately 900 ft of existing LPV floodwall located west of and adjacent to the previous U.S. Coast Guard facility. That section of LPV floodwall likely would be removed in order to extend the MR&T to tie-in to the southward face of the replacement lock.

USACE ER 1165-2-132 dated, June 26, 1992, titled, “Hazardous, Toxic and Radioactive Waste (HTRW) Guidance for Civil Works Projects” provides that construction of Civil Works projects in HTRW-contaminated areas should be avoided where practicable. Where HTRW contaminated areas or impacts cannot be avoided, response actions must be acceptable to EPA and applicable state regulatory agencies (i.e., LADEQ). CEMVN has performed a preliminary examination of the physical extent of the HTRW sites as they relate to the potential floodwall realignment. In 2019, CEMVN contracted JESCO to perform additional environmental site assessment investigations for the known HTRW sites located at the prior USCG site. On behalf of CEMVN, JESCO submitted an April 2019 Risk Evaluation/Corrective Action Plan (RECAP) Site Investigation and Interim Action Report to the Louisiana Department of Environmental Quality, Remediation Division (LADEQ-RD). The LADEQ-RD responded to CEMVN by letter dated March 20, 2023, acknowledging receipt of the April 2019 RECAP Report, and requested USACE provide a site investigation work plan to delineate the vertical and horizontal extent of the contamination. CEMVN responded to LADEQ-RD by letter dated July 24, 2023, providing the requested work plan as well as committing to provide annual status updates of implementation of the work plan to LADEQ-RD no later than October 30th of each calendar year. CEMVN also advised LADEQ-RD that implementation of the work plan is contingent upon receipt of federal funding (construction funds) after completion of the lock replacement study. In a letter dated November 20, 2023, LADEQ-RD acknowledged completion of their review of the work plan and concurred with continued coordination both annually as well as upon receipt of federal funds and subsequent implementation of the work plan.

During future engineering and design prior to construction, CEMVN will implement the aforementioned work plan in coordination with LADEQ-RD to determine if there is a practicable way to avoid disturbance of the affected section of LPV floodwall. If it is determined that there is no practicable, cost-effective way to avoid disturbance of the affected section of LPV floodwall, then CEMVN would perform additional coordination with LADEQ-RD and a Corrective Action Plan would be prepared for LADEQ-RD approval to determine the appropriate remediation actions. As it would be the lock replacement project that would require alteration of the existing LPV alignment in order to tie-in the MR&T floodwall to the replacement lock, if alteration of the present LPV floodwall in the vicinity of HTRW materials were required, that cost would be borne by the lock replacement project.

For additional information on HTRW, refer to the GRR, and to Exhibit 5.

14 Operation, Maintenance, Repair, Replacement and Rehabilitation

Cost and closure schedules were prepared by the USACE Louisville District (CELRL-ED-D-S) in 2015 in collaboration with MVN-OD.

For purposes of this Engineering Appendix, the provisions and definitions of ER 1110-2-401, Section 5.l, 30 September 1994, serve to limit and define the terms “repair”, “rehabilitation” and “replacement” as used herein. The regulation provides, in pertinent part as follows:

“…Repair is considered to entail those activities of a routine nature that maintain the project in a well-kept condition. Replacement covers those activities taken when a worn-out element or portion thereof is replaced. Rehabilitation refers to a set of activities as necessary to bring a deteriorated project back to its original condition...”

14.1 Overview of Work

The cost and closure schedules detail the anticipated maintenance and repair demands for all IHNC Lock Replacement Project plans during the fifty-year study period. The schedules were developed based upon key indicators including historical performance at the project, MVN’s current maintenance program, as well as multiple large-scale investment strategies from other USACE inventory of projects.

14.2 Cost and Closure Matrix Guidelines

The IHNC Lock GRR was used to support the framework of the schedules. The GRR outlined the screening process and provided five (5) project plans.

- Plan 3 : Concrete U-frame Chamber 900 ft x 110 ft
- Plan 5 : Concrete U-frame Chamber 1,200 ft x 110 ft
- Plan 2 : Concrete U-frame Chamber 900 ft x 75 ft
- Plan 4: Concrete U-frame Chamber 1,200 ft x 75 ft

The sill elevation of (-) 16.5 was not considered during the operation and maintenance analysis.

Plan 1 is the "future without project" plan.

There are two scenarios considered when projecting future cost and closure schedules for each plan. The first scenario is a "Fix as Fails" plan where the historical maintenance pattern is projected into the future and then increased maintenance is required as the structure continues to age. This pattern of increasing maintenance requirements is supporting historical data about the structure. The second scenario will be the "Advanced Maintenance" schedule following either a major rehabilitation or large-scale improvement (i.e., new lock chamber). Once a major rehabilitation or large-scale capacity improvement is made it is assumed that the chamber returns to a much-improved maintenance scenario since all the unreliable features have been replaced as necessary.

Fix as Fails (FAF) Schedule

One of the primary purposes and outputs of this analysis is the long-term investments in infrastructure required of the system to sustain a safe and reliable transportation link. The more age and operating cycles that the infrastructure sees, the more cumulative wear and tear occurs to the structure. Review of the last 20 years of maintenance costs and closures within the USACE inventory of lock structures, supports the premise that older facilities require more frequent maintenance closures than the newer facilities.

Advanced Maintenance (AM) Scenario

An improved maintenance scenario for this analysis means that either a major rehabilitation or large-scale capacity improvement (i.e., new lock chamber) has taken place and the deteriorated lock that was rehabilitated or replaced no longer needs additional closures beyond what history has shown to be "typical" frequencies and durations for reliable locks. The AM scenario contains much less maintenance closures than those associated with an aged and deteriorated project. This is because it is assumed all unreliable features would be replaced or substantially improved such that the lock can be returned to a maintenance schedule that is more reflective of a "reliable" lock. The other categories of maintenance closures are included in the advanced maintenance scenario following a major rehabilitation or new lock chamber. All maintenance up to the date of the completed work uses the FAF maintenance scenario within the reliability analysis.

For Future without Project, the following work items and frequency were used for the analysis:

Work Item	Frequency
Unwatering & Monitoring / Major Repairs / Gates	10 years
Rehabilitation of Chamber Guide wall (W & E)	20 years
Rehabilitation of Guide wall (NW & SW)	20 years
Rehabilitation of Guide wall (NE & SE)	20 years
Rehabilitation of Dolphin (NE, NW, SE, SW)	15 years
Minor Hurricane Damage	5 years
Major Hurricane Damage	25 years
Rewiring and Machinery Rehabilitation	20 years
Maintenance by Hired Labor Units	Annually
Rehabilitation of Chamber Guide wall Armoring (W & E)	12 years
Rehabilitation of Guide wall Face Timber (NW & SW)	12 years
Rehabilitation of Guide wall Face Timber (NE & SE)	12 years
PLC System Upgrade	5 years
Routine Maintenance	Annually
Periodic Inspection (PI) Program	5 years
A/E Instrumentation (Pre-PI)	5 years
Security Maintenance Contract w/ ACE-IT^^^	Annually
ED Instrumentation Cost	Annually

^^^ ACE-IT: Army Corps of Engineers Information Technology

For the RP (and the study plans except future without project), the following work items and frequency were used for the analysis:

Work Item	Frequency
Unwatering & Monitoring / Major Repairs / Gates	10 years
Rehabilitation of Chamber Guide wall (W & E)	50 years
Rehabilitation of Guide wall & Dolphin (NW & SW)	35 years
Rehabilitation of Guide wall & Dolphin (NE & SE)	35 years
Minor Hurricane Damage	5 years
Major Hurricane Damage	25 years
Rewiring and Machinery Rehabilitation	30 years
Maintenance by Hired Labor Units	3 years
Rehabilitation of Chamber Guide wall Armoring (W & E)	25 years
Rehabilitation of Guide wall Face Timber (NW & SW)	15 years
Rehabilitation of Guide wall Face Timber (NE & SE)	15 years
PLC System Upgrade	5 years
Routine Maintenance	Annually
Periodic Inspection (PI) Program	5 years
A/E Instrumentation (Pre-PI)	5 years
Security Maintenance Contract w/ ACE-IT ^^^	Annually
ED Instrumentation Cost	Annually

14.3 Cost and Closure Schedules

The individual work items and related repairs schedules were determined by a multitude of factors:

- Patterns in historical operations and maintenance records at the IHNC
- Conversations with IHNC Lock project personnel, such as Lockmaster, Project Manager, Project Engineers, etc. regarding current maintenance schedules, recent work items and costs
- Historical operations and maintenance performance data of similar MVN projects (Bayou Sorrel and Calcasieu C&C Schedules were completed in 2011)
- Historical operations and maintenance performance data of similar lock projects within the USACE inventory

For the purpose of this analysis, chamber closures were broken down into three primary categories based on cost of work and impact to navigation: No Impact to Navigation, Minor Repair and Major Repair.

No Impact to Navigation

These columns are dedicated to all work items that are considered routine maintenance and result in the project not needing to shut down operation more than two (2) hours in order to accomplish the tasks. Examples of work include basic routine maintenance, instrumentation calibration or computer system upgrades where implementation of work can occur either during or in-between normal lock operations.

Minor Repair

Minor repair items are closures that are essentially independent of routine maintenance work. These involve down time due to instances that are considered unavoidable. Lock chambers are sometimes closed for unforeseen occurrences regardless of historical level of maintenance.

These columns are dedicated to work items that require the project to shut down operation for entire 12-hour shifts up to 175 cumulative hours and carry a cost of less than $1,000,000. Examples of work include guide wall rehabilitation where a floating plant crew must occupy the chamber to perform the work or a critical piece of machinery (i.e., gate sector gear or valve hydraulic cylinder) must be removed for repair or rehabilitation, thus rendering the chamber inoperable.

Major Repair

Many features of a lock chamber deteriorate with time and usage. As can be expected, the older and high-use projects are closed more often for maintenance. These features require more frequent maintenance to keep them operational. Most of these features (gates, operating machinery, and others) require the lock chamber to be closed in order to perform maintenance.

These columns are dedicated to typically large-scale work items that result in the project to shut down operation for entire 12-hour shifts greater than 175 cumulative hours and carry a cost greater than $1,000,000. Examples of work include routine unwatering and emergency repairs such as barge impacts to guide walls and dolphins or hurricane damage. Chamber closure is a result of emergency failure resulting in direct inoperability and/or extensive repair work with crews occupying the chamber.

14.4 Cost and Closure Projection Results

In all scenarios considered for the IHNC Lock Replacement Project, the ‘No Impact to Navigation’ component is broken out separately and holds an equal value in each case. Given the nature of the items (routine maintenance, security contracts, periodic inspections, etc.), this kind of work is considered constant and unlikely to vary in cost or frequency throughout the duration of the study period.

The first schedule compiled was the ‘Without Project’ condition. This scenario assumes no replacement lock chamber will be constructed and the IHNC Lock will be the sole chamber for the entire duration of the 50-year study period. The ‘Without Project’ condition also assumes typical maintenance (FAF) will continue, and no heightened levels of preventative maintenance will be implemented above what is already in place.

With the original lock chamber construction completed in 1923, the project will be 161 years old by the end of the period being analyzed in this GRR (period of analysis ends in 2084). Much degradation has already occurred in all facets from concrete to machinery and with the assumed lack of preventative maintenance in the future, the degradation of the chamber and all its components will continue and grow more severe throughout the study period. The frequency of repairs is greatest within this scenario and is adequately captured within the schedule.

Plans 2 to 5 quantify each replacement lock chamber configuration based on the screening process from the IHNC Lock GRR. Construction of the replacement chamber is scheduled to begin in 2024 and will undergo a 6-year construction schedule. From the initiation of the GRR to the completion of construction of the chamber (2029), the existing chamber will continue to operate under a Fix-as-Fails maintenance schedule. After completion of the replacement chamber, to the end of the period of analysis (2084) the new chamber will operate under an Advanced Maintenance schedule.

The primary difference between the FAF and AM schedules is the proactive approach to quality control on critical components. For example, sector gate and culvert valve machinery universally hold a 30-year life expectancy, but instead of waiting for the component to fail (FAF), it may be rehabilitated or repaired during a routine unwatering a few years ahead of its expected lifespan (AM). The result is a small increase in cost and closure for that particular year for the added work, but the gains are seen in the less frequent failures and need for closures throughout the study period.

The screened lock replacement plans are as follows. The sill depth for all plans is (-) 22 ft:

- Plan 3: 900 ft x 110 ft concrete U-frame chamber
- Plan 5: 1200 ft x 110 ft concrete U-frame chamber
- Plan 2: 900 ft x 75 ft concrete U-frame chamber
- Plan 4: 1200 ft x 75 ft concrete U-frame chamber

The existing IHNC Lock chamber has 5 sets of miter gates. With the closure of the MRGO, the need for a deep draft-chamber is no longer required, so every plan is for a shallow-draft chamber. Subsequently, with a shallow-draft chamber, the need for miter gates is no longer required, so every plan will consist of 2 sets of sector gates. This uniformity in design makes the matrices for all plans very similar, with slight variation in costs coming from the geometrical differences (i.e., rehabilitation on a 1,200 ft guide wall will cost more than a 900 ft guide wall and similar with a 110 ft sector gate versus a 75 ft gate). All closure breakdowns are included as a separate 'Closure Breakdown' tab as well as individual breakdowns included in each plan (called Alternative) tab.

14.5 OMRR&R Cost Summary for Study Plans

The cost summary for the OMRR&R plans used in Alternatives Analysis is as follows. For more details on the Alternative-level OMRR&R plans, refer to Annex 6: OMRR&R Analysis.

Plan 3: Concrete U-frame Chamber 900 ft x 110 ft

Plan 5: Concrete U-frame Chamber 1,200 ft x 110 ft

Plan 2: Concrete U-frame Chamber 900 ft x 75 ft

Plan 4: Concrete U-frame Chamber 1,200 ft x 75 ft

Table 25: OMRR&R Total Costs for Each Plan (2015 Dollars)

	Costs (2015 Dollars)		
	Minor Repairs	**Major Repairs**	**Total**
Plan 1- With No Project	$ 36,500,000	$ 106,000,000	$ 142,500,000
Plan 3- - 900 ft x 110 ft	$ 23,875,000	$ 65,500,000	$ 89,375,000
Plan 5- 1200 ft x 110 ft	$ 24,175,000	$ 67,500,000	$ 91,675,000
Plan 2- 900 ft x 75 ft	$ 23,875,000	$ 62,500,000	$ 86,375,000
Plan 4- 1200 ft x 75 ft	$ 24,175,000	$ 64,500,000	$ 88,675,000

	Closure Durations (Hours)		
	Minor Repairs	**Major Repairs**	**Total**
Plan 1- With No Project	9,800	24,750	34,550
Plan 3- - 900 ft x 110 ft	5,725	15,715	21,440
Plan 5- 1200 ft x 110 ft	5,725	15,715	21,440
Plan 2- 900 ft x 75 ft	5,725	15,715	21,440
Plan 4- 1200 ft x 75 ft	5,725	15,715	21,440

The OMRR&R costs above were used during the Alternative Selection milestone. To convert from 2015 dollars to 2019 dollars, use escalation factor of 1.08. The analysis for the RP used 2019 dollars.

Table 26: OMRR&R Total Costs for Each Plan (2019 Dollars)

	Cost (2019 Dollars)
Plan 1- With No Project	$153,900,000
Plan 3- - 900 ft x 110 ft	$96,525,000
Plan 5- 1200 ft x 110 ft	$99,009,000
Plan 2- 900 ft x 75 ft	$93,285,000
Plan 4- 1200 ft x 75 ft	$95,769,000

Following the determination of Plan 3 as the Recommended Plan during the Alternative Selection milestone, the OMRR&R Plans for the Recommended Plan (Plan 3) and Future Without Project (Plan 1) were further refined, both in level of detail and in costs used.

14.6 OMRR&R Cost Summary for the Recommended Plan

The cost and closure schedules detail the anticipated maintenance and repair demands for all IHNC Lock Replacement Project plans during the 50-year period of analysis. The schedules were developed based upon key indicators including historical performance at the project, MVN's current maintenance program, as well as multiple large-scale investment strategies from other USACE inventory of projects.

Construction of the replacement chamber is scheduled to begin in 2033 and will undergo an 8-year construction schedule. The chamber will be completed in year 2041. The OMRR&R of the new chamber is assumed to start in year 2042 (the year after chamber construction is complete). The construction completion for all components of the project (for example, demolition of the old lock, construction of the St. Claude Bridge and demolition of the old St. Claude Bridge) will be completed in year 2047. The period of analysis of 50 years starts in year 2048 and ends in year 2096. Refer to Annex 4 for construction schedule. The schedule for the RP (ADM) was used to develop the OMRRR&R per year in 2025 dollars:

Table 27: OMRR&R for the RP (2025 Dollars)

Lock 110' x 900' (-) 22 Draft					
FY Year	OMRR&R Cost (2025 dollars)	FY Year	OMRR&R Cost (2025 dollars)	FY Year	OMRR&R Cost (2025 dollars)
2029	$2,902,278	2052	$ 4,436,781	2074	$ 4,147,114
2030	$4,733,210	2053	$ 7,647,520	2075	$ 6,188,512
2031	$3,998,745	2054	$ 2,540,966	2076	$ 2,251,434
2032	$4,144,768	2055	$ 4,727,646	2077	$ 7,648,950
2033	$7,648,932	2056	$ 4,147,156	2078	$ 4,731,166
2034	$2,249,144	2057	$ 1,811,037	2079	$ 1,808,593
2035	$4,728,476	2058	$ 9,544,313	2080	$ 1,956,092
2036	$4,726,374	2059	$ 3,997,261	2081	$ 5,821,806
2037	$1,810,451	2060	$ 6,917,478	2082	$ 9,544,327
2038	$9,253,447	2061	$ 1,805,064	2083	$ 1,808,375
2039	$1,809,732	2062	$ 1,956,582	2084	$ 4,728,649
2040	$4,439,873	2063	$ 8,743,204	2085	$ 6,187,860
2041	$1,811,739	2064	$ 4,436,169	2086	$ 1,957,007
2042	$4,729,751	2065	$ 1,809,632	2087	$ 2,244,858
2043	$7,648,020	2066	$ 2,535,589	2088	$ 8,741,382
2044	$1,081,493	2067	$ 4,728,660	2089	$ 4,729,281
2045	$3,999,095	2068	$ 9,253,950	2090	$ 1,805,027
2046	$1,082,146	2069	$ 2,904,199	2091	$ 1,957,728
2047	$4,729,606	2070	$ 5,164,828	2092	$ 1,812,290
2048	$9,836,625	2071	$ 1,810,317	2093	$ 1,954,944
2049	$2,906,162	2072	$ 6,918,448	2094	$ 5,826,238
2050	$4,145,754	2073	$ 7,648,480	2095	$ 9,545,907
2051	$2,901,260	2074	$ 4,147,114	2096	$ 1,811,738
				Total OMRR&R	$ 302,011,642

14.7 OMRR&R Cost Summary for a Future Without Project

The cost and closure schedules are a series of spreadsheet matrices that detail the anticipated maintenance and repair demands for all IHNC Lock Replacement Project plans during the 50-year period of analysis. The schedules were developed based upon key indicators including historical performance at the project, MVN's current maintenance program, as well as multiple large-scale investment strategies from other USACE inventory of projects.

Assuming that the lock is not replaced, the OMRR&R was calculated using the same years (2029 to 2096) as in the analysis of the RP in previous section. The schedule for the FAF plan was used to develop the OMRR&R per year in 2025 dollars for the existing lock:

Table 28: OMRR&R for Future Without Project (2025 Dollars)

Future Without Project- Predicted					
FY Year	OMRR&R Cost (2025 dollars)	FY Year	OMRR&R Cost (2025 dollars)	FY Year	OMRR&R Cost (2025 dollars)
2029	$2,905,042	2052	$1,080,982	2074	$ 2,066,356
2030	$4,729,616	2053	$2,066,524	2075	$ 1,082,170
2031	$3,999,036	2054	$1,081,433	2076	$ 8,375,161
2032	$4,145,607	2055	$1,082,366	2077	$ 2,065,250
2033	$7,646,572	2056	$2,065,546	2078	$ 2,026,996
2034	$2,247,150	2057	$1,808,110	2079	$ 1,080,747
2035	$4,730,355	2058	$1,448,094	2080	$ 6,446,684
2036	$4,726,329	2059	$2,065,018	2081	$ 2,029,674
2037	$1,809,406	2060	$1,080,505	2082	$ 6,914,935
2038	$9,253,720	2061	$6,914,350	2083	$ 2,067,148
2039	$1,808,180	2062	$2,065,485	2084	$ 1,446,116
2040	$4,440,154	2063	$2,028,632	2085	$ 1,807,510
2041	$1,810,405	2064	$1,079,719	2086	$ 2,066,817
2042	$1,081,583	2065	$2,070,039	2087	$ 2,172,793
2043	$1,078,403	2066	$2,031,620	2088	$ 8,014,304
2044	$1,080,022	2067	$1,080,731	2089	$ 2,067,077
2045	$1,079,435	2068	$2,064,426	2090	$ 1,080,309
2046	$1,081,115	2069	$8,743,188	2091	$ 6,918,240
2047	$1,078,574	2070	$1,809,739	2092	$ 1,079,765
2048	$1,079,443	2071	$7,903,783	2093	$ 6,441,333
2049	$1,076,543	2072	$2,171,554	2094	$ 2,028,341
2050	$2,065,017	2073	$6,551,473	2095	$ 6,915,906
2051	$6,916,785	2074	$2,066,356	2096	$ 2,065,970
				Total OMRR&R	$210,421,408

The total OMRR&R cost for the RP is $ 302,012,000. The total OMRR&R cost for a future without project is $210,422,000. The difference between the predicted OMRR&R for the existing lock and the OMRR&R of the new lock is $91,590,000.

15 Engineering Assessment of Existing Lock

In 2017, MVN ED prepared an analysis with the details of the current condition and deficiencies of the existing IHNC Lock structures and components. During PI No. 11, engineers performed a close visual examination of the lock. A total of thirty-three (33) deficiencies were found, requiring a remedial action. Five (5) years later during PI No. 12, engineers identified a total of fifty-three (53) deficiencies, indicating that the condition of the lock continues to deteriorate and the required maintenance to maintain navigation continues to increase.

According to the survey from the most recent PI Report, the existing miter gates are at elevation 15.70 referenced to NAVD88 epoch 2004.65; the authorized flowline elevation is 17.30 referenced to the same datum and epoch. This results in a deficiency of 1.60 ft at the structure because the miter gates are lower than the authorized flowline elevation. The deficiency is not a result of changes based on earlier flowline studies before 1973. The previous studies were not used to establish authorized water surface for the MR&T Project or construct the existing lock. The lock was designed in the early 1900's before the authorized flowline was published. Different design standards and hydrologic conditions were considered to construct the lock. For this reason, the deficiency is not solely due to settlement or environmental factors; it's also due to the difference in hydrology assumptions and criteria. The existing lock was unwatered in 2016. During this event, the machinery, electrical, and several miter gates were replaced. This unwatering will give the lock additional years of operation but does not address the main structural deficiency which is spalling of the concrete in various locations, and age of steel reinforcement, which could lead to problems in the future.

Additionally, it should be noted that the existing lock is not designed for an unwatering load case. An unwatering event results in unacceptable safety factors for flexure and flotation of the chamber. The lock was designed using now obsolete codes, and the reinforcement is inadequate for modern concrete design. There is a significant wear of the concrete and steel reinforcement due to the age of the structure. Continued deterioration of the concrete and steel reinforcement may preclude future maintenance unwatering of the chamber. Maintenance unwatering will eventually be unsafe without extensive retrofit of the existing structure. The retrofit of the existing lock would be costly and create additional delays to navigation. Based upon the number of deficiencies, replacement of the lock is recommended.

For more information, details, and pictures of the damage, refer to USACE-MVN-EDS, memorandum, revised 21 June 2017 (original memo dated 14 October 2016). The memorandum may be found in Annex 9.

16 Maintenance Issues with Existing Lock

In 2019, additional maintenance issues with the current lock were discovered. These issues included degradation of the lock valves and water infiltration into and out of the lock through the concrete. Investigations revealed degradation of the concrete through construction joints at the lock floor, leading to water running into the manways and machinery pits of the locks and causing large sinkholes to form adjacent to the lock walls.

In particular, on the northern end of the structure, two (2) large sink holes formed along the east and west lock walls. These two sink holes align with each other, indicating a potential continuous seepage connection beneath lock base slab from the east side of the lock to the west side.

The IHNC Lock was unwatered from August 2020 to November 2020 to address the formation of sinkholes along the lock and install new sluice valves and operating equipment. Once unwatered, water leakage through the structure was observed in multiple deteriorated construction joints. The worst leakage observed was in the vicinity of the large sinkholes on the west side of the lock near gates 7 and 8 that had begun forming in November of 2018. All construction joints with significant leakage were injected with polyurethane to stop the water flow. Additionally, cores were drilled through the lock slab to determine whether voids had formed beneath the lock structure. Cores revealed a maximum of 1-2 ft of voids beneath the lock structure. Concrete was injected beneath the lock structure to fill in these voids.

While the unwatering accomplished the intent to identify the source of the major sinkholes and repair the structure as needed, only a small fraction of the structure's original construction joints was repaired during this unwatering. Environmental conditions such as higher frequency of rain events have resulted in increased periods of high river, which will continue to place greater pressures on the aging construction joints that were not repaired during the unwatering. It should be noted that the flow through the construction joints noted in the 2020 unwatering was not occurring during the 2016 unwatering. The deterioration of the construction joints that caused the formation of sinkholes largely occurred due to the high river events in 2018 through 2020. While unwaterings can be undertaken to fix construction joints in the future, it can reasonably be expected that the frequency of these unwaterings will increase as the structure continues to age, resulting in increases in delays to navigation.

Based on the vast number of documented deficiencies, replacement of the lock is technically merited.

17 Future Without Project (FWOP) for the Existing Lock

The HSDRRS is a redundant system with exterior or perimeter flood risk reduction measures, with the TOW elevation of the Inner Harbor Navigation Canal Surge Barrier of 26 ft. Once the Inner Harbor Navigation Canal Surge Barrier is closed, the lower pool (Lake Pontchartrain) is not exposed to the Gulf of America and IHNC Lock becomes an interior feature of HSDRRS.

On the upper pool (Mississippi River), the existing navigation lock continues to operate during high water events. However, locking procedures are slightly altered for safety reasons. For instance, tows are put in the queue (or 'on turn') at a greater distance in the river from the forebay of the lock and tows, depending on size and other variables, are allowed to break in the forebay rather than in the river. Both instances are due to increased flow velocities in the river and/or to avoid impacting the MRL. The primary concern with the lock and high water in the river are the machine rooms in the lock chamber that could be overtopped when the riverside gate is opened. Ultimately, the result is an increased total time for a tow to lock through the existing navigation lock. The lock is closed for navigation once the stage in the canal reaches 15 ft.

At the high RSLC rates, FWOP could result in high water and storm events exceeding the existing lock elevation. The existing lock could be retrofitted to account for these height deficiencies, but it would require extensive retrofits that would be very costly and not economically feasible or justifiable. Alternatively, a sector gate out in front of the existing lock structure could be investigated to provide the required level of risk reduction for an extreme event coupled with the high rate of RSLC.

18 Plates, Figures, and Drawings

Refer to Annex 11 for plates.

19 Data Management

During the GRR phase, the PDT continued to use ProjectWise as the engineering data management system for the study, to store all design work and all information developed for the study.

20 References

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Note: For structural references, see Section 12 of this appendix.