APPENDIX F: MBSD DESIGN AND OPERATIONS INFORMATION

F1: Design Documentation Report (60% Design)

F2: Preliminary Operations Plan

F1: Design Documentation Report (60% Design)

Rev 1

STATE OF LOUISIANA

COASTAL PROTECTION AND RESTORATION AUTHORITY

MID-BARATARIA SEDIMENT DIVERSION (MBSD) PROJECT

STATE PROJECT No. BA-0153

LaGOV NO. 4400020885

Preparation of Engineering and Design

DESIGN DOCUMENTATION REPORT (DDR)
60% FINAL DESIGN SUBMITTAL

for

PRELIMINARY – FOR PERMIT PURPOSES ONLY

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Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACB	Articulated Concrete Blocks
ACI	American Concrete Institute
A/E	Architect and Engineer
AHP	Above Head of Passes
AISC	American Institute of Steel Construction
AREA	American Railway Engineering Association
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ATR	Agency Technical Review
AWS	American Welding Society
BFE	Base Flood Elevation
CCL	Conveyance Channel Levee
CFD	Computational Fluid Dynamics
cfs	Cubic Feet per Second
CIP	Cast-in-Place
CMAR	Construction Manager at Risk
CORS	Continuously Operating Reference Stations
CPRA	Coastal Protection and Restoration Authority
CPT	Cone Penetrometer Test
DAV	Depth Averaged Velocity
DDR	Design Documentation Report
DMPA	Dredged Material Placement Area
DMM	Deep Mixing Method
DT	Design Team (AECOM)
E&D	Engineering and Design
EL	Elevation
EIS	Environmental Impact Statement
EM	Engineering Manual
ESA	Environmental Site Assessment

ETL	Engineering Technical Letters
FN	Froude Number
FWCA	Fish & Wildlife Coordination Act
GEBF	Gulf Environmental Benefit Fund
GPS	Guide to Minimum Standards
Gr	Grade
H&H	Hydrologic and Hydraulic
HSDRRSDG	Hurricane and Storm Damage Risk Reduction System Design Guidelines
HSS	Hydraulic Steel Structure
HSS	Hollow Structural Sections
HVAC	Heating, Ventilation and Air Conditioning
Hwy	Highway
I&C	Instrumentation & Controls
I.C.E.	Independent Cost Estimator
IEL	Interim Earthen Levee
ITR	Independent Technical Review
LaDOTD	Louisiana Department of Transportation and Development
LAPELS	Louisiana Professional Engineering and Land Surveying Board
LCA	Louisiana Coastal Area
LCZ	Louisiana Coastal Zone
LFPDG	Louisiana Flood Protection Design Guidance
MBSD	Mid-Barataria Sediment Diversion
MBrSD	Mid-Breton Sediment Diversion
MDE	Maximum Design Earthquake
MNS	Mass Notification System
MPRSA	Marine Protection Research & Sanctuaries Act
MR	Mississippi River
MRL	Mississippi River Levee
MRMBSDP	Mississippi River Mid-Basin Sediment Diversion Program
NAVD	North American Vertical Datum
NEPA	National Environmental Policy Act
NFL	Non-Federal Levee

NFWF	National Fish Wildlife Foundation
NGS	National Geodetic Survey
NOGC	New Orleans Gulf Coast Railroad
NOV	New Orleans to Venice
NRDA	Natural Resource Damage Assessment
NTP	Notice to Proceed
O&M	Operations and Maintenance
OBE	Operating Basis Earthquake
OMRR&R	Operations, Maintenance Repair, Replacement and Rehabilitation
OTF	Outfall Transition Feature
PCC	Portland Cement Concrete
PLS	Professional Land Surveyor
PMIS	Program Management Information System
PMT	Program Management Team
PIC	Principal-in-Charge
POC	Point of Contact
PPG	Plaquemines Parish Government
PSF	Pounds per square foot
PTZ	Pan Tilt Zoom
PVC	Polyvinyl Chloride
PVD	Prefabricated Vertical Drain
QAQC	Quality Assurance Quality Control
RFI	Request for Information
RM	River Mile
ROE	Right of Entry
ROW	Right-of-Way
R/R	Railroad
SAR	Safety Assurance Review
SCADA	Supervisory Control & Data Acquisition
SIBM	Settlement Induced Bending Moments
SLR	Sea Level Rise
SME	Subject Matter Expert

SOV	Schedule of Values
sTons	Short Tons
SUE	Subsurface Utility Engineering
SWL	Still Water Level
SWR	Sediment to Water Ration
TBD	To be determined
TOS	Top of slab
TOW	Top of Wall
TPC	Third Party Contractor
TWIG	The Water Institute of the Gulf
UPRR	Union Pacific Railroad
USACE	US Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geographic Survey
VE	Value Engineering
WBS	Work Breakdown Structure
WEAP	Wave Equation Analysis for Pile
WRDA	Water Resources Development Act
WSE	Water Surface Elevation

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APPENDICES

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1. PROJECT INFORMATION

The Coastal Protection and Restoration Authority (CPRA) has located the Mid-Barataria Sediment Diversion (MBSD) on the West Bank of the Mississippi River in Plaquemines Parish, Louisiana, at River Mile 60.8 Above Head of Passes (AHP), between the Phillips 66 Alliance Refinery upriver and the Town of Ironton downriver. The upstream portion of the MBSD intersects the Mississippi River Levee (MRL) at Station 1109+58, and the downstream portion intersects the existing and proposed NOV-NF-W-05a.1 levees. See **Figure 1-1.**

The MBSD will reconnect the River to the Barataria Basin, delivering sediment to rebuild the delta marshes with the ultimate goal of improving coastal protection against the effects of sea level rise, subsidence, and storm events.

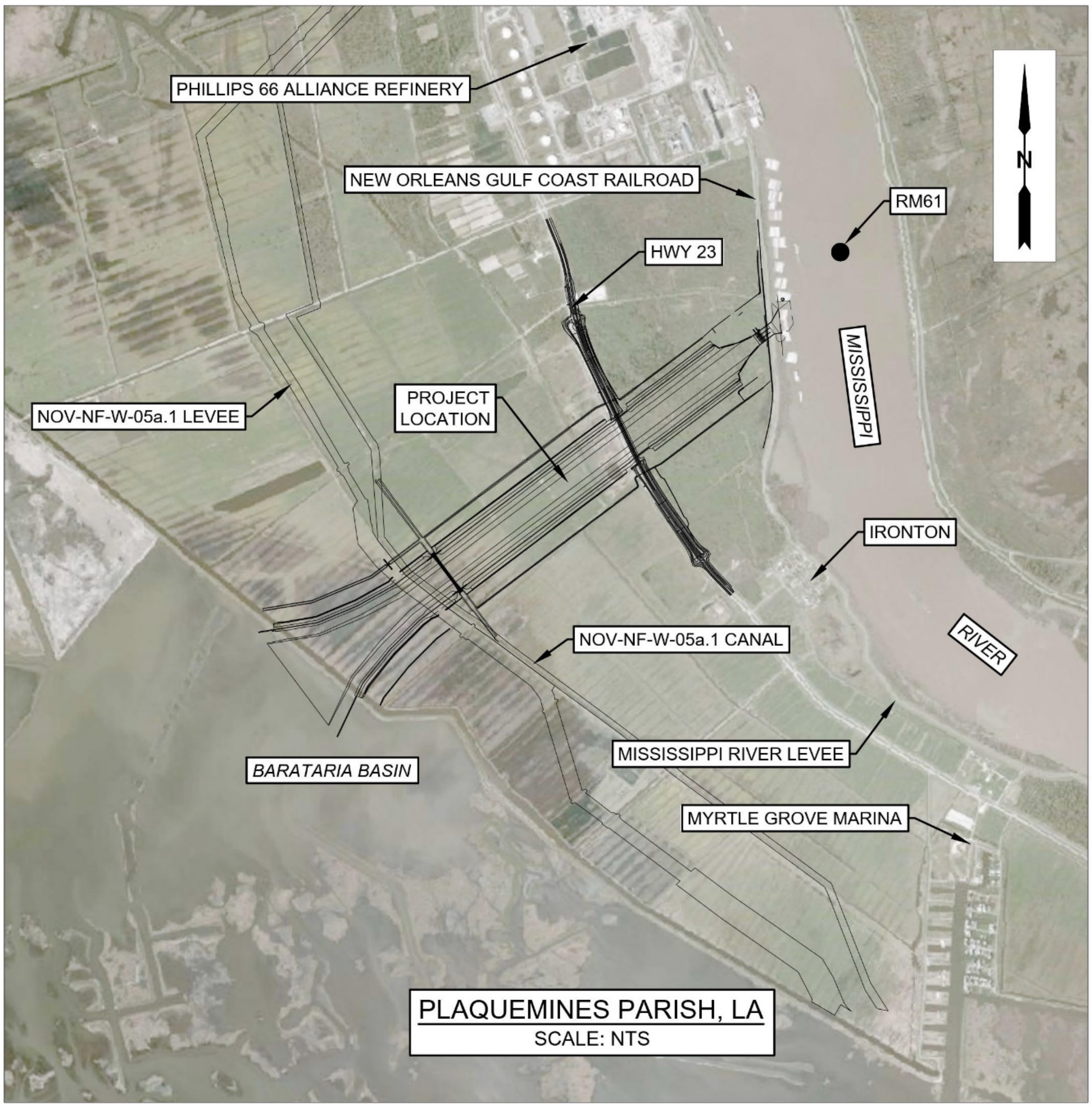

Figure 1-1: Location Map

The MBSD Project is one of two projects which comprise CPRA's Mississippi River Mid-Basin Sediment Diversion Program, the other being the Mid-Breton Sediment Diversion (MBrSD). The MBSD will divert river flow and sediment from the Mississippi River to the Barataria Basin, establishing conditions which will allow the development of a delta area via the transport and deposition of sediment carried downstream by the river during flood events. Goals of the project include:

- Reconnect the Mississippi River to the Barataria Basin
- Establish conditions to allow the development of a delta area open to tidal exchanges
- Deliver 75,000 cubic feet per second (cfs) flow through the Conveyance Channel from the Mississippi River Levee (MRL) to the Barataria Basin by operating gates of the diversion structure. This flow rate was used as a basis to further develop design concepts at the proposed MBSD site. The final diversion flow rates are to be designed to meet the project goals.
- Maintain the current level of flood risk reduction of the MRL and New Orleans to Venice (NOV) levee
- Design the Intake Structure, control structure, channel, and appurtenances to maximize sediment capture and delivery, maximize flow efficiency, and allow for operations adaptability based on monitoring data collected during project operation, while minimizing Operations, Maintenance, Repairs, Replacement, and Rehabilitation (OMRR&R)
- Meet state and federal design criteria and environmental compliance requirements as required to achieve project regulatory approval
- Develop an operational plan for the diversion structure

The MBSD's sediment delivery system is a three-component system which includes sediment intake, conveyance, and discharge. The intake (also referred to as the headworks) consists of an intake structure, diversion gates, and a transition channel. The intake and gate monoliths serve to draw in the required sediment laden water and also serve as a flood protection. The conveyance feature includes an approximate 2-mile conveyance channel and guide levees that parallel the channel. The guide levees that extend from the transition walls to the Corps levee project NOV 5a also serve as flood protection. The parallel levees provide flood protection to the 50-Year (2% AEP) level, future conditions. The discharge component includes an outfall transition feature which ties into the Barataria Basin. Other project components not directly related to sediment conveyance include Hwy 23 bridge and roadway realignment, railroad relocation, interim flood protection measures, an inverted drainage siphon to maintain drainage to Wilkinson Pump Station, a drainage structure in the NOV 5a levee located north of the conveyance channel, utility relocations, and secondary project features such as support buildings and a boat ramp. The siphon connects the north polder drainage area to the existing Wilkinson Pump Station. The drainage structure in the NOV 5a levee drains the impounded area located between the NOV 5a levee and the existing back levee. The siphon and drainage structures include sluice gates added to prevent flooding from hurricane events.

CPRA has structured the MBSD contract as an Early Involvement contract. The Design Team is performing engineering analysis and designs for the permanent features of the MBSD project, which is currently in the 60% Design Phase. CPRA has already selected a Construction Manager at Risk (CMAR), who provides input to the Design Team regarding constructability and logistics during the design process. The CMAR's team is responsible for the designs associated with any temporary project features, such as the interim Mississippi River Levee and cofferdam. This DDR documents the designs of both the permanent and temporary MBSD project features.

USACE has been involved in several aspects of the project, including permit reviews by the New Orleans District for geotechnical testing, the Risk Management Center's participation in a Semi-Qualitative Risk Analysis for the MBSD guide levees, and various coordination meetings at the request of CPRA. Because the MBSD project will alter a federal project (the MRL and the NOV back levee), CPRA is requesting Section 408 Permission at the completion of 60% Design.

2. SURVEY

2.1 Survey Datum

The survey datum used for horizontal coordinates is NAD 1983 (2011) 2010.00 Epoch and for vertical control NAVD 1988 (2009.55 Epoch) Geoid 12B. All elevations described in this report are in feet.

2.2 Primary Survey Control

The primary survey control benchmarks used for this project are V 393 2006 and N 366 1984. Both benchmarks were established by the National Geodetic Survey (NGS).

2.3 Project Surveys and Imagery

Survey data obtained includes the following:

- Mississippi River Bathymetric and Magnetometer Surveys
- Topographic Survey of project site
- Outfall Bathymetric and Magnetometer Surveys
- High-resolution aerial photography from Mississippi River to Outfall

3. HYDRAULIC DESIGN

3.1 General

The results of the hydrologic and hydraulic (H&H) analyses, and the numerical and physical modeling to support the E&D are described in this section. These analyses were performed for the three major components of the diversion system, namely, the intake headworks, the diversion channel and the outfall transition feature (OTF). The H&H analyses also guided the design of the inverted drainage siphon and inclusion of the existing Wilkinson Pump Station that facilitate the drainage of the polders separated by the diversion channel.

The modeling performed to-date is at 60% E&D level including the internal Value Engineering (VE) analysis. The changes proposed through the VE study are ongoing. Included in this 60% submittal are changes to the intake headworks, Conveyance Channel geometry, and the OTF. The conveyance channel section was revised in the western half of the channel. The channel side berm was eliminated as a cost savings measure which was found to also have hydraulic benefits. This revision eliminated the construction of a 6-foot (average depth) berm resulting in more conveyance area. The second VE revision was the refinement of the OTF; the length of the heavily armored OTF was reduced and the horizontal geometry was improved as described in Para 3.2.3 below. Hydraulic modeling is ongoing on all other VE revisions which will be accomplished in the subsequent 60% E&D Phase, and the DDR will be updated accordingly after modeling is completed. Significant changes under consideration include raising the intake invert elevation from EL -40 to EL -25 and moving the gate monolith closer to the MRL.

3.2 Numerical Modeling of the Major Diversion Components

Two calibrated numerical models were used to estimate water surface elevation, velocity, discharge, and energy loss through the diversion system. The non-hydrostatic, three-dimensional (3D), computational fluid dynamics (CFD) software program FLOW-3D was primarily applied to simulate the near-field, rapidly varied flow hydrodynamics. The hydrostatic, 3D, modeling software Delft3D was used to estimate sediment capture and transported loads through the diversion system. The Delft3D software in a two-dimensional (2D) form was used to simulate hydraulics of the larger domain including the Barataria Basin. All modeling considers conditions as they currently exist and also conditions projected 50 years in the future.

Figure 3.2-1 shows the predicted profiles of the water surface elevation, the depth-averaged velocity (DAV), and the total energy head along the length of the diversion system starting from the river on the right to the end of the Outfall Transition Feature (OTF) to the left. The scenarios plotted in the figure are summarized in **Table 3.2-1**. The profiles are results of a 2D Delft3D model consisting of a 10-mile MR segment, the diversion system, and the Barataria Basin.

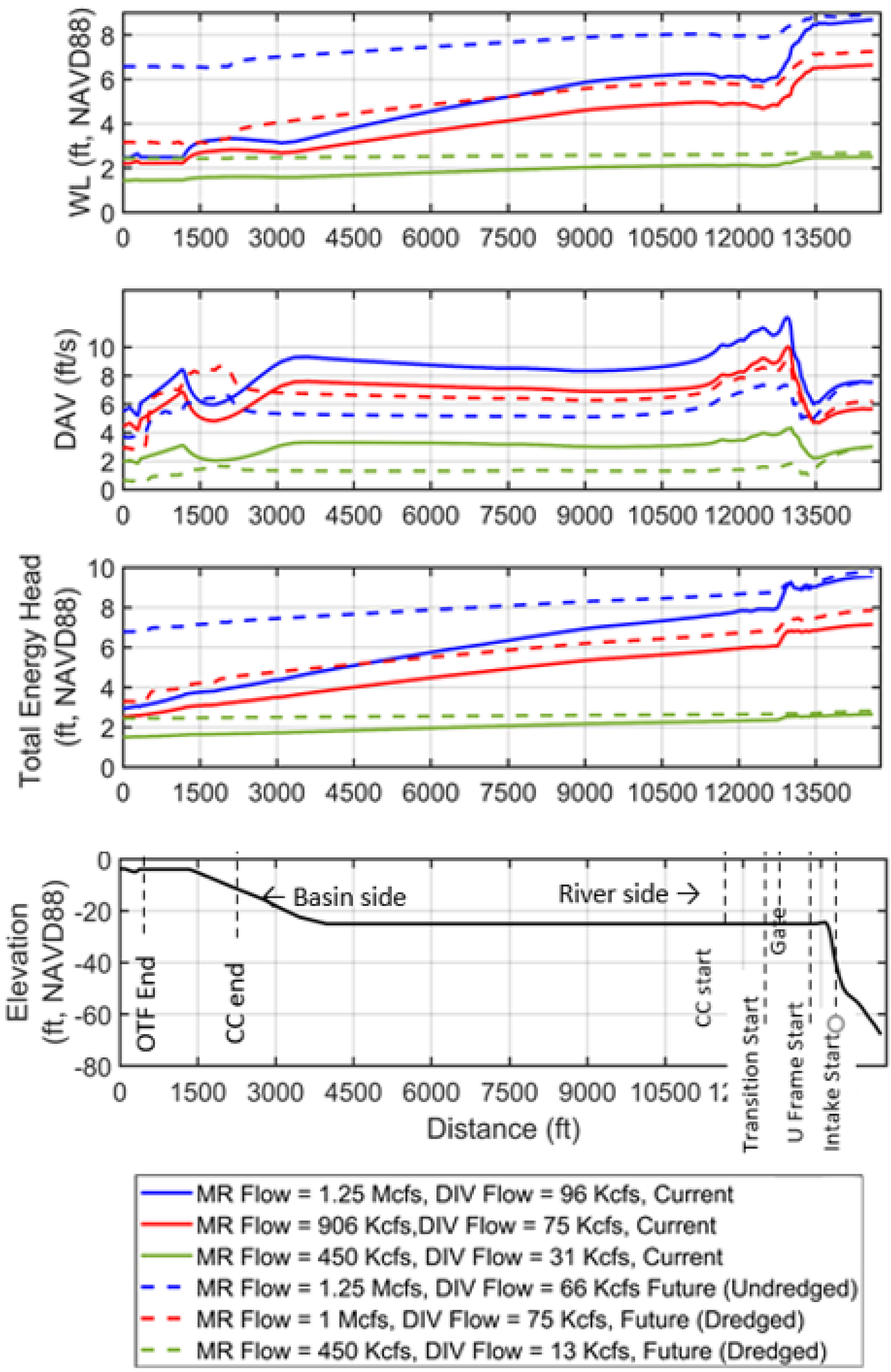

Figure 3.2-1: Predicted Profiles of the Water Surface Elevation, the Depth-Averaged Velocity (DAV) and the Total Energy Head Along the Length of the Diversion System. Flow is from right (river end) to left (outfall end) in the above panels.

Table 3.2-1: Hydraulic Numerical Model Scenarios for Design of Structural Components

No	Upstream MR Inflow[1] (1,000 cfs)	Model Geometry[2]	Estimated Diversion Discharge (1,000 cfs)	Comments
1	1,250	Current	96	This case is considered as **the most likely maximum diversion flow possible at the highest river discharge** at this location under current conditions. This can happen if gates fail to close at high river and is likely to be a very short-term peak condition for design components on the basin-side of the gates. For river-side of the gates this may be taken as most likely ‘regular’ maximum conditions. Suggested useful hydrodynamic information for design from this scenario are: 1. DAV within the conveyance channel and intake transition where short term high flows (period before which the gate operation can be restored) can cause particular damage to riprap. 2. DAV within the river portion of the intake (river-side of U-Frame start) for riprap design. This DAV can be sustained over the same period for which MR flow of 1.25M cfs in river exists.
2	906	Current	75	This case is considered as the **most likely MR flow beyond which diversion gates will be lowered** to restrict the diverted flow to 75,000 cfs. This scenario provides information on design guidance for components on the basin-side of the gates and is a normal occurrence medium term peak operating condition. Suggested useful hydrodynamic information for design from this scenario are: 1. DAV within the intake transition, conveyance channel and OTF for riprap design. These are peak velocities that the diversion is likely to experience every year at design flow of 75,000 cfs. 2. DAV within the intake U-Frame for abrasion design. These are peak velocities that the diversion is likely to experience every year at design flow of 75,000 cfs.
3	450	Current	31	This is the **most likely minimum diverted flow at trigger MR flow** (i.e., when the diversion is opened in rising limb or closed in falling limb**)**. This scenario provides information on design guidance for components on the basin-side as well as river-side of the gates and is a very high occurrence probability medium term lowest operating condition. Suggested useful hydrodynamic information for design from this scenario are: 1. DAV within the U-Frame, Intake Transition, Conveyance Channel and OTF for riprap design.

Table 3.2-1: Hydraulic numerical model scenarios for design of the structural components of the diversion system (Continued)

No	Upstream MR Inflow[1] (1,000 cfs)	Model Geometry[2]	Estimated Diversion Discharge (1,000 cfs)	Comments
4	1,250	Future Undredged	66	This is the **most likely maximum diverted flow in the future if no maintenance dredging is performed** to maintain design flow. This scenario provides information on design guidance for components on the basin-side and river-side of the gates and is a <u>normal occurrence probability short term peak operating condition if operators do not decide to perform maintenance dredging</u>. Based on the possible uncertainties in dredging schedules, it is recommended that design stages of critical flood protection components be at least at the WL mark predicted by this scenario. Suggested useful hydrodynamic information for design from this scenario are: 1. WL in the river, U-Frame, intake transition, conveyance channel and OTF for levee stage/flood protection design. 2. WL in the conveyance channel and OTF for levee stability design.
5	1,000	Future Dredged	75	This is the **design condition in the future (50 years) after basin-side dredging.** This scenario provides information on design guidance for components on the basin-side and river-side of the gates and is a <u>normal occurrence probability medium term peak operating condition if basin-dredging is performed</u>. This scenario is now provided for information only, no particular use of the hydrodynamic data is suggested.
6	450	Future Dredged	13	This is the **most likely minimum diverted flow at trigger MR flow** (i.e., when the diversion is opened in rising limb or closed in falling limb**)** in the future. This scenario provides information on design guidance for components on the basin-side and river-side of the gates and is a <u>high occurrence probability medium term lowest operating condition in the future if basin-dredging is performed</u>. This scenario is now provided for information only, no particular use of the hydrodynamic data is suggested.

Notes:

1. The upstream MR boundary is at RM 66. The downstream MR boundary is at RM 56. The downstream boundary is set at as a stage-boundary relationship based on the past (2008-2018) ten years of data.
2. The future conditions basin bathymetry/ topography is from the CPRA land-building Basin-wide model. The river-side bathymetry is based on the current multibeam high-resolution USACE bathymetries.
3. The MR downstream stage-discharge (Q-H) relation is assumed not to change in the future because of RSLR effects as not enough modeling information exists as of now to justify this from WI Basin-wide modeling exercise.

3.2.1 Intake Headworks

To provide estimates of depth-averaged velocities in the river-portion of the intake, the FLOW-3D model was simulated with 1,250,000 cfs MR flow (the maximum allowable MR flow at this location) with all diversion bays fully open. The diverted discharge is 96,000 cfs.

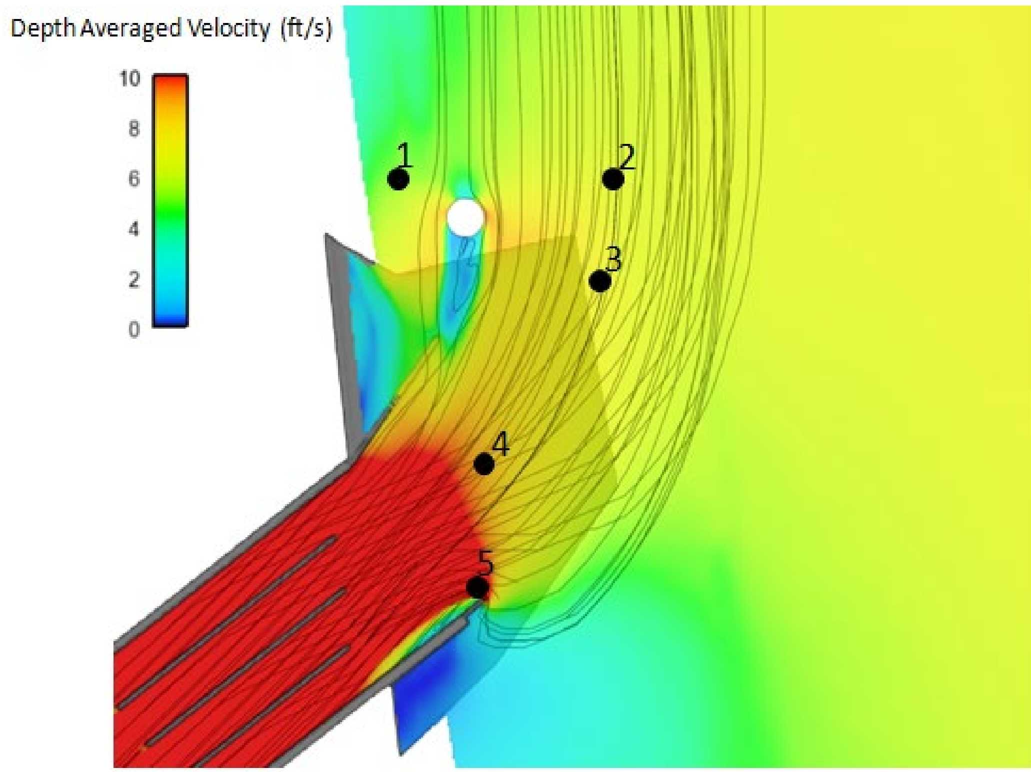

Figure 3.2-2: Depth Averaged Velocity (ft/s)

Figure 3.2-2 shows depth-averaged velocity color-filled contours and streamlines obtained from FLOW-3D model simulation with MR flow of 1,250,000 cfs. Numbered solid circles are locations where values in **Table 3.2-2** are extracted.

Table 3.2-2: Depth averaged velocity, surface velocity and bottom velocity from the five selected points

Point ID	Coordinates Easting, Northing (m, UTM 15N)	Water Depth (ft)	Depth-Averaged Velocity (ft/s)	Velocity at surface (ft/s)	Velocity at about 6 ft above bottom (ft/s)
1	793966, 3285379	10	5.0	6.0	2.6
2	794070, 3285379	38	6.4	7.3	4.3
3	794065, 3285330	34	6.9	7.9	5.3
4	794006, 3285241	34	8.7	9.5	7.3
5	794036, 3285160	30	13.5	15.4	11.7

The 10-Year simulation of the local river morphology near the intake showed degradation in the upstream vicinity (**Figure 3.2-3**).

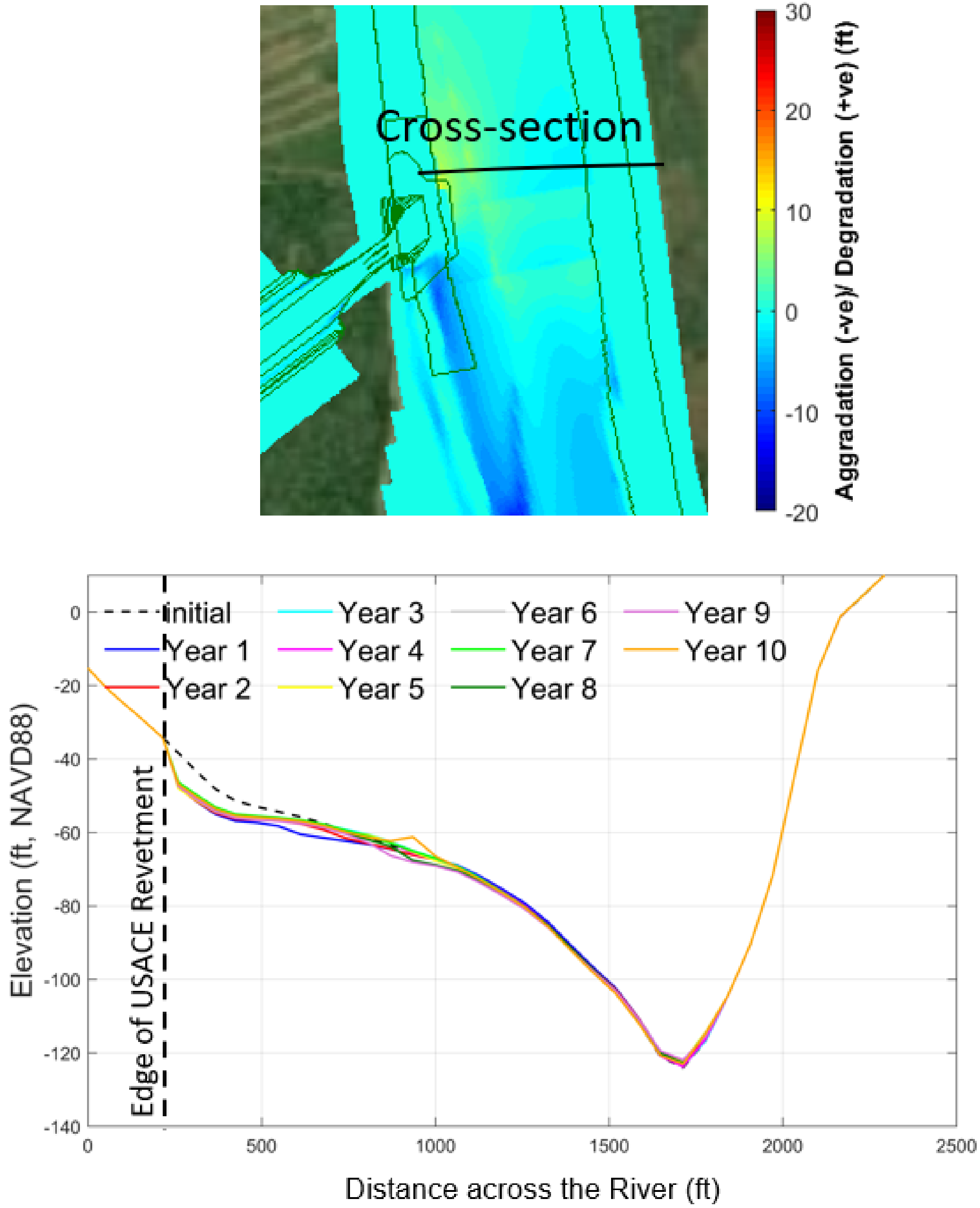

Figure 3.2-3: Morphological evolution of the intake vicinity

Figure 3.2-3 shows the morphological evolution of the intake vicinity over a 10-year (2008-2018) period. Degradation (+ve) and aggradation (-ve) is represented by warm and cool colors, respectively.

The modeling performed to provide guidance for the sizing of the riprap in the transition segment between the U-Frame and the conveyance channel is presented in **Section 6.6.3**.

3.2.2 Conveyance Channel

The purpose of the Channel is to convey the diverted water from the intake at the Mississippi River (MR) to the NOV Levee and into the Basin. The extension from the Intake Structure at the MR to the NOV Levee, which is approximately 2 miles, is necessary to prevent flooding of the infrastructure between the MR and NOV Levees.

One of the key design requirements of the channel is to convey design flow and SWR without any erosion or deposition in the channel. This is primarily a function of the flow speed, which is controlled by the channel's cross-section geometry. The flow speed needs to be sufficiently high such that it can support the sediment load coming though the diversion. A modeling analysis was conducted to determine the flow speed and sediment carrying capacity. A Delft3D model was developed to simulate the diversion flow and loads. The model configuration for the diversion is shown in **Figure 3.2-4**. The domain includes the conveyance channel starting downstream of the intake expansion ramp, the Outfall Transition Feature and the nearfield portion of the Barataria Basin. The appropriate downstream water elevation boundary conditions and basin bathymetry in the nearfield region were developed using data from TWIG's Basin Wide Model simulations. The sediment loads used in the modeling analysis that are associated with the diversion flows are based on the traditional Sand Rating curve and the hysteresis rating curve for fine sediment rating curves developed from measured data in the MR. The modeling process is fully documented in the MBSD Conveyance Channel Modeling Report which was submitted to CPRA in November 2019. The rating curves are documented in the TWIG 2017 deliverable for Task Order 46 (TWIG, 2017).

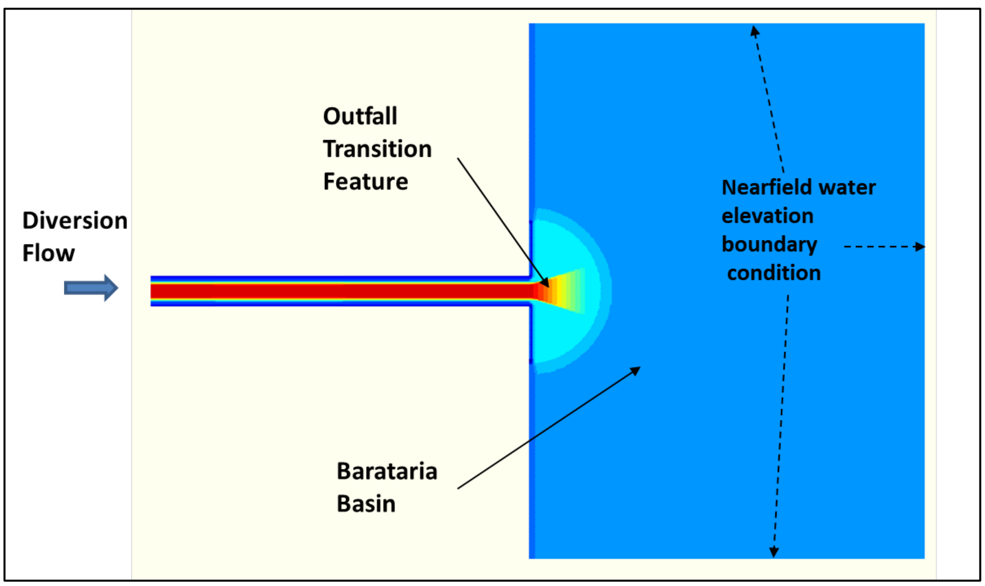

Figure 3.2-4: Delft3D Model Domain for Conveyance Channel Analysis

For the design flow of 75,000 cfs discharge, the cross-section average flow speeds are on the order of 6 fps and were able to support the sediment load passing from the MR through the Intake Structure. The modeling analysis was also completed for a flow at the lower range of expected diversion flows, 40,000 cfs. The results also indicated that the lower flow could transport the sediment load from the MR to the basin without deposition in the channel.

The bottom width of the channel is 300 feet, with 4H:1V side slopes. Armoring for the channel is discussed in **Section 6.6**.

Limiting water quality degradation is considered an operational objective and does not impose design constraints on the Conveyance Channel geometry. Operational strategies for maintaining water quality objectives such as periodic flushing of the channel are being evaluated as design progresses.

3.2.3 Outfall Transition Feature

The hydraulic design of the OTF was improved through a series of iterations where the flare angle, defined as the angle between the conveyance channel axis and the angle of the flare of the OTF, was varied over the length of the OTF in three stages. The design goals were: (1) Reduce eddying effects due to flow separation where the conveyance channel ends and the OTF begins, and (2) Reduce potential for the sediment deposition on the OTF shoulders that would concentrate flow to the middle of the OTF increasing scour hole. The gradual flaring of the OTF in three steps with adequate transition lengths between steps allows for flow stabilization which also reduces a potential scour hole at the OTF edge. Some deposition on the OTF berms near the upstream end of the OTF was found to be unavoidable and should be considered in the geotechnical design of the OTF berms.

Morphology modeling of the Outfall Transition Feature (OTF) indicated that the scour to an elevation of about EL -10 NAVD88 will occur within a short distance downstream of the toe wall (head-cut protection feature) at the end of 3 years of diversion operation (**Figure 3.2-5**). The maximum diversion flow is restricted to 75,000 cfs for the results shown here. The critical shear stress of the native soil is set at 1.5 Pa within the basin. The rate of scouring is the highest in the first year and becomes smaller over the next two years, indicating that the scour depths have almost approached equilibrium after the first year. A sufficient safety factor is suggested in designing the depth of the toe wall in view of the uncertainties in the geotechnical properties and morphology modeling.

For the design of the riprap stability, the peak velocities within the OTF were determined from the scenario simulating the highest possible diverted flow (96,000 cfs) at the highest MR flow (1.25 M cfs) shown in **Figure 3.2-1** previously. Peak velocities of 8 ft/s are possible at the end of the OTF. Note that since the diversion flow is capped at 75,000 cfs by the gate operations, this scenario is a short duration event which may happen if the gates fail to come down at high river.

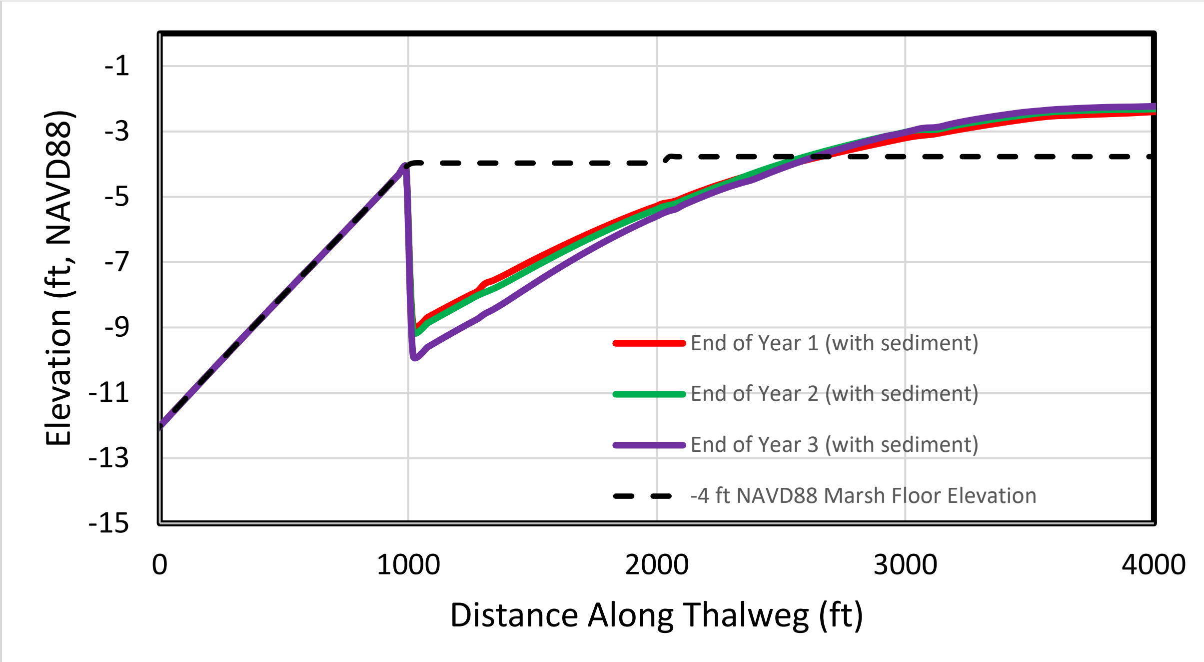

Figure 3.2-5: Predicted scour bed elevations

Figure 3.2-5 shows predicted scour bed elevations from with and without river-sediment runs for the 2,000-foot pulled back OTF. The maximum scour bed elevation at the end of 3 years is EL -10 NAVD88.

The current design did not show flow separation because of the gradual rise in the elevation of the OTF flare and gradual stepped flaring of the OTF. Additional numerical and physical modeling is planned for the 60% E&D phase to further improve the OTF design.

3.3 Physical Modeling

The MBSD is being designed with an extensive numerical and physical modeling program. The two modeling programs complement each other; each modeling approach (numerical and physical) contributes to developing a comprehensive understanding of how the system will perform. Numerical models rely on empirical sediment transport functions and a set of simplified governing equations to describe the movement of water. Therefore, the predictive capability of numerical models is limited by the applicability of the sediment transport functions and the complexity of the hydrodynamic model. Physical models have limitations due to scaling constraints and can be time consuming to construct and operate, possibly resulting in scale induced limitations and fewer tests than what is possible with a numerical model. When combined, the two modeling approaches provided the highest degree of confidence in the development of the design.

A detailed discussion of the conveyance channel and intake modeling is provided in **Appendix B**.

3.3.1 Modeling Approach and Objectives

The total physical model domain includes about 12,500 feet of the Mississippi River, the diversion, and the conveyance channel. The conveyance channel is approximately 10,000 feet in length and diverts at about a right angle to the river. This makes the required model building size extremely large. Therefore, a modeling approach was selected where the domain is split into two models, a river model and a conveyance channel model. The need to maximize model size superseded the desire to have a single model. **Figure 3.3-1** shows the domain of the river model (yellow) and the conveyance channel model (blue).

Figure 3.3-1: Physical Model Domains
(Diversion Intake & Mississippi River model in yellow, Conveyance model in blue)

The river model includes about 12,500 feet of the river, the diversion, and a short (about 1500 feet) reach of the conveyance channel. The primary goals of the 1:65 scale river model tests were to:

- Measure the sediment water ratio through the diversion
- Determine riprap size required in front of the diversion intake
- Determine hydraulic rating curves for the radial gates
- Determine riprap size requirements downstream of the gates
- Determine areas of sediment deposition around the diversion
- Determine if sedimentation will occur downstream of the diversion
- Collect velocity data for numerical model validation
- Develop cofferdam shape
- Develop cofferdam construction sequencing

The prototype depth at the thalweg is about 100 feet, or 1.5 feet in the model. At the diversion, the prototype depth is 45 to 50 feet giving a model depth between about 0.7 and 0.75 feet. In the 1:65 Froude scale model, a river flow of 1,000,000 cfs equates to a laboratory flow of about 30 cfs. The river model is a recirculating live bed model without a sediment feed system. Water and sediment discharged at the downstream end of the river and conveyance channel are both pumped back and introduced to the upstream end of the model.

A separate 1:65 scale model of the conveyance channel and outfall transition was also constructed and tested. The primary goals of the model tests were to:

- Measure head loss in the conveyance channel for clear water conditions
- Determine if sediment accumulates on riprap
- Determine sediment transport characteristics in the flat conveyance channel
- Determine if sediment which deposits during low flow conditions is re-suspended during high flow conditions
- Evaluate sediment deposition and scour in the outfall transition
- Evaluate armoring in the outfall transition
- Evaluate sediment accumulation on the stability berm
- Provide validation data for numerical model validation

The conveyance channel model had a sediment feed system which allows the injection of a specific sediment concentration at the upstream end of the model. A recirculating model was not possible because one objective of the model was to evaluate scour and sedimentation in the outfall transition.

3.3.2 Model Description and Scaling

To obtain the most representative data possible from the physical model, it is necessary to have a physical model that includes both bed load and suspended load. For the purpose of land building, sand size particles were identified as the most important particle size (Ramirez and Allison, 2013). Finer material (wash load) is approximately uniformly distributed in the water column and numerical modeling has shown that the SWR for wash load is independent of diversion design (FTN, 2018). In contrast, numerical models showed that the SWR for sand sized material varies with the diversion design. The physical models are designed to investigate the transport and diversion of sand sized particles. Finer silt and clay sized particles have been shown by others to have approximately a uniformly concentration throughout the river cross section, and the diverted amount of silt and clay is proportional to the amount of water diverted for all diversion designs. Silt and clay sized particles were not included in the models.

The 1:65 scale of the river and conveyance models is based on the required Reynolds number to maintain turbulent flow, analysis of the prototype grain size distribution and available model sediment size and density to match fall velocity. The model Froude number matches that in the prototype and the model sediment is a lightweight plastic material. The particle size distribution in the models was customized to match the prototype as well as possible, and the particle specific gravity in the model was about 1.08.

A detailed discussion of model scaling is included in **Appendix B.**

3.3.3 Instrumentation

Model instrumentation included flow measurement, water surface elevation measurements, water velocity measurements, sediment accumulation quantities (using a laser scanner), suspended sediment samples, and turbidity meters. Each piece of instrumentation provides an essential piece of data for understanding how the sediment diversion system performs.

Model flow was measured using Venturi meters and orifice plates fabricated and installed per ASME guidelines. The accuracy of the flow measurement is estimated at +/- 2%.

Piezometric taps were used to measure the water surface elevation in both models. The piezometric taps were connected to stilling wells where the water surface elevation was measured with a vernier point gauge. The accuracy of the measurements was approximately 0.065 feet for the prototype (0.001 feet in the model). Accuracy of the water level measurements was confirmed by filling the models with water and having zero flow.

Water velocities were measured using either SonTec Micro ADV (Acoustic Doppler Velocity) meters or miniature propeller meters. The MicroADV can operate in a sediment rich environment. For the Micro ADV velocity is measured about 2 inches from the sensor; therefore, the probe can only be used where the water depth exceeds about 3 inches (16 feet prototype at a 1:65 scale). Miniature propeller meters were used to measure water velocity in locations where the MicroADV was not suitable. The miniature propeller meter is a photo-optical interrupt detector. The meter automatically counts the number of revolutions completed in a period of time.

Changes in bed geometry were measured using a Trimble FX 3D scanner. At the beginning of each test, the sediment bed was graded to a uniform thickness and scanned. At the end of a test, flow in the model was gradually reduced in order to preserve bedforms, and then the water was drained from the model. The scanner was used to measure the ending bed elevation. Computer software was used to determine the change in bed elevation.

During each river model test, suspended sediment samples were collected at four locations in the river approximately 500 feet upstream from the locations used by Allison (Allison, 2011). Samples were also collected at three locations in the diversion channel. A one liter (approximate) water sample was collected, and the mass of sediment in the sample was measured to determine the suspended sediment concentration.

3.3.4 Boundary Conditions and Model Control

Flow in the river model was circulated using a laboratory pump. The model does not use a laboratory sump. Flow and sediment leaving the downstream end of the model is conveyed directly to the pump pit where both are pumped to the upstream end of the model. This creates a model where the amount of sediment delivered to the upstream end of the model is equal to the sediment transport capacity of the reach. Because the model does not have a sump, the water level in the model is controlled by changing the total volume of water in the model.

The conveyance channel model was constructed with a sediment feed system to continuously supply the necessary sediment. Use of a sediment feed system made it possible to quantify how the channel and outfall basin evolve with time. Any sediment that exits the model was removed with a filter system such that only feed sediment was entering the model. The feed rate was determined from prototype sediment concentration data. The water level at the downstream end of the model was controlled with bottom hinged tip gates that maintained the water level in the discharge basin. The water level was based on numerical modeling results provided by FTN.

The physical model was constructed and is being operated based on Froude similitude because the flow physics are dominated by gravitational and inertial forces. The Froude number (Fr), representing the ratio of inertial to gravitational forces, is defined for rivers as,

$$\mathrm{Fr} = \frac{\mathrm{V}}{\sqrt{\mathrm{gd}}} \tag{1}$$

where
V = depth average river velocity
g = gravitational acceleration

d = river depth

The model Reynolds number exceeds a minimal value of about 2,000 to maintain turbulent flow conditions (Gill and Pugh, 2009).

Relevant model scale ratios are given in **Table 3.3-1**. A detailed discussion of scaling is provided in **Appendix B.**

Table 3.3-1: Model Scale Ratios for 1:65 Scale Model

Scaled Item	Scaling as function of length scale	Model with Froude Similitude	Typical Prototype value	Model value
Length Scale $L_r = L_m / L_p$		1/65	65 ft	1 ft
Velocity Scale $V_r = V_m / V_p$	$V_r = L_r^{1/2}$	1/8.06	3 ft/s	0.372 ft/s
Flow Scale $Q_r = Q_m / Q_p$	$Q_r = u_r A_r = L_r^{5/2}$	1/34,063.04	1,000,000 cfs	29.36 cfs
Time Scale $T_r = T_m / T_p$	$T_r = \frac{L_r}{u_r} = L_r^{1/2}$	1/8.06	1 hour	7.44 min

3.3.5 Validation

The river model was validated by comparing model and prototype velocity measurements. The prototype data was collected by Allison (2011).

Velocity Validation

The physical model geometry accurately represents the prototype model geometry. Therefore, if the model velocity profiles match the prototype measurements near the upstream end of the model then downstream velocities should also match. Alden validated the physical model at a river flow of 617,000 cfs and 1,060,000 cfs. Adjustments were made to the headbox and flow distributor such that the depth averaged velocity profiles matched within about +/- 15%. During validation, testing the diversion geometry was included in the model but no flow was withdrawn, thus it should not impact the model velocities. A more detailed discussion of model validation is provided in Alden, 2020.

Suspended Sediment Concentration Validation

Validation of suspended sediment concentration profiles is considerably more difficult than validation of the water velocity. Suspended sediment concentration varies significantly at the same flow depending on which part of the hydrograph the sample was collected on. Suspended sediment concentration in the model and prototype was compared at three locations, MGup2, MGup3, and MGup4 (Allison, 2011).

The suspended sediment concentrations in the model agree well with prototype values at flows less than about 800,000 cfs. However, at larger flows, the model consistently has a higher depth averaged sediment concentration than the prototype. A detailed analysis of the model and prototype data shows that the model over represents the suspended sediment concentration towards the center of the river at flows of 900,000 cfs and 1,000,000 cfs. The model sediment concentration toward the right bank more closely matches prototype conditions. A detailed discussion of suspended sediment concentration is included in Alden, 2021.

3.3.6 Data Collection and Testing Program

River Model Data Collection

During short and long duration model testing the following data was collected.

- The sediment concentration profile in the river was measured at four locations across the river at three time intervals during the test. During Test 1, samples were collected at 20, 50 and 80% of the river depth. During subsequent tests samples were nominally collected at 20, 35, 50, 60, 70, and 80% of river depth (0% is the water surface). The number of samples was increased to improve the granularity of the measured sediment concentration profile. One sample was collected at each depth at three times during the test program. River samples were collected at approximately the same locations used by Allison (Allison, 2011), though not all of the points used by Allison were sampled.
- The sediment concentration in the conveyance channel was measured at three locations across the channel at three time intervals during the testing. During Test 1, samples were collected at 20, 50 and 80% of the conveyance channel depth at the end of the model portion of the conveyance channel. During subsequent tests samples were nominally collected at 20, 40, 60 and 80% of water depth at the end of the gated structure and beginning of transition into the conveyance channel.
- The entire model was laser scanned before and after each test. The scans provide insight on the bedform size and geometry and also show general areas of scour and deposition in the river.
- Fifteen velocity measurements were made during each test for validation of the CFD model completed by FTN.
- Dye and video were used during each test to document flow patterns.
- Photographs were used after each test to document changes in the bed geometry.
- Water surface elevation was measured at eight locations in the river and 11 locations in the conveyance channel about 12 times during the short-duration tests and about 48 times during the long-duration tests.
- River flow and diversion flow was monitored continuously throughout each test.

Figure 3.3-2 shows the location where data was collected.

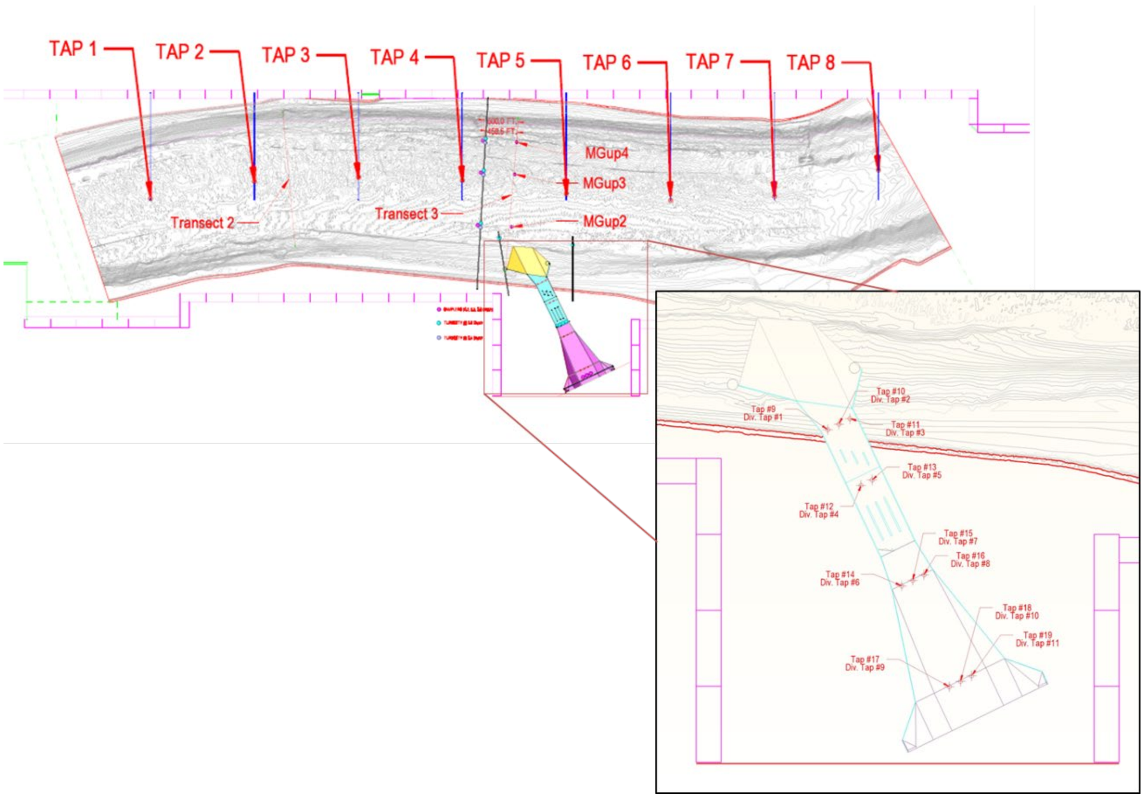

Figure 3.3-1: Locations where model data was collected in the river model.

<u>River Model Invert EL -40 Testing Program</u>

The initial river model testing program with the invert EL -40 gate design involved 10 tests to evaluate the system performance at a range of Mississippi River flows. The diversion flow in the model was set based on the numerical modeling results. The 10 model tests are summarized in **Table 3.3-2** and **Table 3.3-3**.

Table 3.3-2: River Model Test Conditions

Test ID	Start Date	Target Values				Gates Open	Diversion Invert
		Duration (hrs)	River Flow (cfs)	Diversion Flow (cfs)	Tap 2 WSE (ft)		
1	9/13/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
2	10/17/19	12	750,000	69,000	5.79	4 @ 45 ft	-40 ft
3	10/25/19	12	1,000,000	75,000	7.33	4 @ 34 ft	-40 ft
4	11/7/19	12	600,000	46,750	4.11	4 @ 40 ft	-40 ft
5	11/19/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
6	12/3/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
7	12/11/19	48	900,000	75,000	6.72	4 @ 40 ft	-40 ft
8	1/9/20	12	600,000	46,750	-2.50	4 @ full open	-40 ft
9	1/27/20	48	750,000	69,000	5.79	4 @ 45 ft	-40 ft
10	2/26/20	12	900,000	60,000	6.72	4 @ full open	-25 ft

Table 3.3-3: River Model Test Goals and Notes

Test ID	Duration (hrs)	River Flow (cfs)	Notes
1	12	900,000	Model validation to Allison (2011) data
2	12	750,000	Determine sediment diversion ratio
3	12	1,000,000	Determine sediment diversion ratio
4	12	600,000	Determine sediment diversion ratio
5	12	900,000	Repeat of test 1 with additional measurements
6	12	900,000	Change in intake armoring approach alignment
7	48	900,000	48-hour test with same conditions used in Test 1 and 5.
8	12	600,000	Repeat of Test 4 but with lower water level to test ability manipulate suspended sediment concentration with small changes in water level.
9	48	750,000	48-hour test with same conditions as test 2
10	12	900,000	Diversion invert was changed to -25 ft without adjusting width.

After Test 1, Alden spent multiple days running the model but without resetting the bed between tests. The testing was exploratory to refine the data collection systems used in Test 1. The changes that were adopted were used on all subsequent tests and Test 1 was repeated. During testing it was noted that bedforms evolve quickly after a test is started. Qualitatively, bedforms appeared to reach full size within one hour. The largest bedforms tended to be larger than prototypical bedforms.

The river model was also used to determine the head loss through the gates for 100 operating conditions. The test conditions are given in Alden, 2020. Riprap stability downstream of the gates was evaluated with a series of tests that involved a high water level in the Mississippi River and a low water level downstream of the gates to simulate opening of the gates with a large head differential. This condition creates the highest velocities under the gate and the highest potential for mobilizing the riprap. Five tests were used to evaluate these conditions.

<u>Conveyance Channel Model</u>

During conveyance channel model testing the following data was collected:

- Sediment concentration was measured at two times during the test and at three locations along the conveyance channel. At each location, nine samples were collected. The samples were collected at approximately 20, 50 and 80% of the channel depth on the channel center line and on the left and right side of the channel (**Figure 3.3-3**).
- The entire model is laser scanned before and after each test to quantify general trends in aggradation or degradation of the channel bed and to quantify the size of ending bed forms.
- During each test, velocity measurements were made at three locations along the length of the channel. At each location 14 measurements were made (**Figure 3.3-4).**
- Dye and video were used during each different channel flow to document flow patterns. Time lapse video was used to document the movement of bedforms for each test.
- Photographs were used after each test to document changes in the bed geometry.
- Water surface elevation was measured at 10 locations along the length of the model and measured three times during the test (**Figure 3.3-5**).

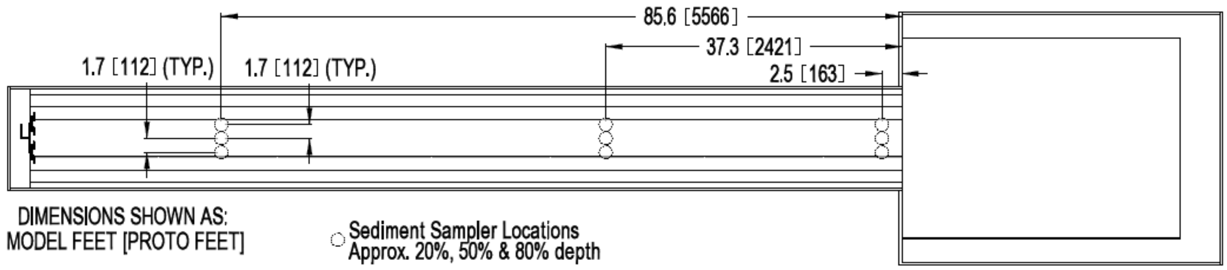

Figure 3.3-2: Location of suspended sediment sampling in conveyance channel.

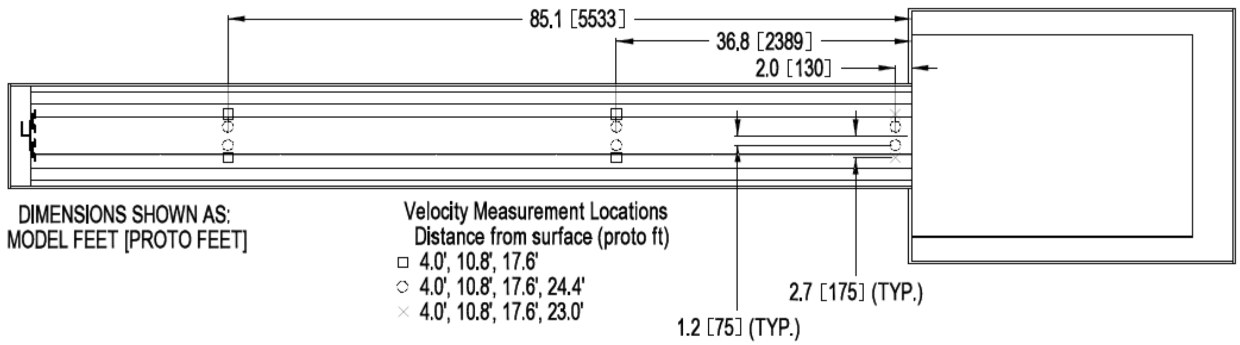

Figure 3.3-3: Location of velocity measurements in conveyance channel.

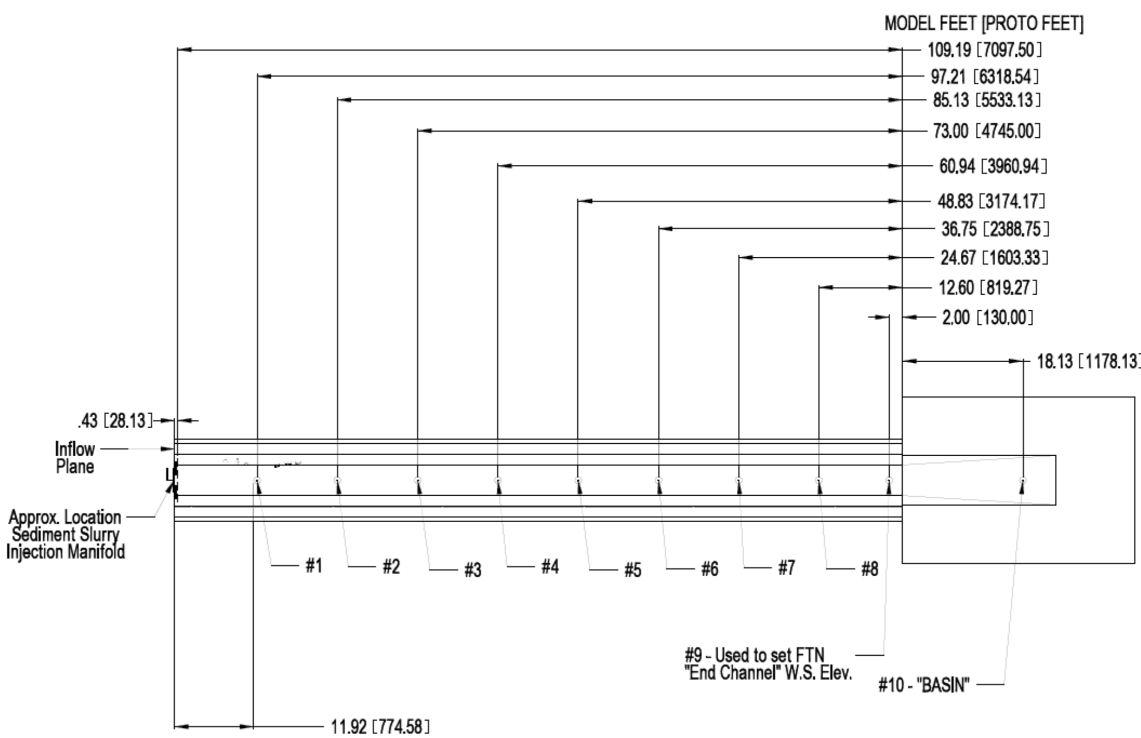

Figure 3.3-4: Location of water level measurements in conveyance channel.

Conveyance Channel Model Testing Program

Four tests were completed to determine the roughness and head loss characteristics of the riprapped channel. The test conditions are described in **Table 3.3-4**.

Table 3.3-4: Clear Water Test Conditions

Test ID	Test Date	Flow (cfs)	Tap #9 WSE (NAVD88 ft)
C1	3/1/19	40,000	1.84
C2	3/6/16	57,500	2.03
C3	2/26/19	75,000	2.16

Thirteen tests were completed with the sediment feed system active to evaluate sediment movement and channel roughness. The tests are described in **Table 3.3-5** and **Table 3.3-6**.

Table 3.3-5: Conveyance Channel Sediment Test Conditions

Test ID	*Start Date*	*Duration (hrs)*	*Flow (cfs)*	*Tap #9 WSE (NAVD88 ft)*	*Feed Conc. (mg/l)*
1	*7/5/19*	*8*	*40,000*	*1.84*	*19*
2	*7/16/19*	*8*	*40,000*	*1.84*	*19*
3	*9/17/19*	*8*	*40,000*	*1.84*	*73.5*
4	*9/30/19*	*8*	*57,500*	*2.02*	*32*
5	*10/3/19*	*8*	*57,500*	*2.02*	*73.5*
6	*10/10/19*	*8*	*75,000*	*2.14*	*51.5*
7	*10/25/19*	*8*	*75,000*	*2.14*	*73.5*
8	*11/11/19*	*48*	*75,000*	*2.14*	*51.5*
9	*1/21/20*	*8*	*75,000*	*2.14*	*51.5*
10	*2/12/20*	*8*	*75,000*	*2.14*	*265*
11	*2/28/20*	*8*	*100,000*	*12.00*	*73.5*
12	*3/10/20*	*8*	*75,000*	*2.14*	*0*
13	*3/17/20*	*8*	*75,000*	*2.14*	*165*

Table 3.3-6: Conveyance Model Test Goals and Notes

Test ID	Duration (hrs)	Flow (cfs)	Notes
1	8	40,000	Shake down test with low inflow concentration
2	8	40,000	Repeat of Test 1with low inflow concentration
3	8	40,000	High inflow concentration (river flow of 1,000,000 cfs).
4	8	57,500	Low inflow sediment concentration
5	8	57,500	High inflow concentration (river flow of 1,000,000 cfs).
6	8	75,000	Low Inflow concentration (river flow of 900,000 cfs).
7	8	75,000	High inflow concentration (river flow of 1,000,000 cfs).
8	48	75,000	Long duration test to evaluate erosion in transition feature, low inflowing sediment concentration.
9	8	75,000	New outfall transition feature geometry. Low inflowing sediment concentration to evaluate transition erosion.
10	8	75,000	Very high sediment feed concentration (265 mg/l) to support Mid Breton testing. Data is included in this report as extreme condition.
11	8	100,000	High water level test to evaluate sediment accumulation on stability berm if stability berm was lowered
12	8	75,000	Riprap disturbance testing
13	8	75,000	High sediment feed concentration (165 mg/l) to support Mid Breton testing. Data is included in this report as extreme condition

River Model Invert EL -25 Fixed Bed Testing

Testing with the invert EL -25 was underway during the writing of this document. The model was reconfigured with a fixed bed for a series of tests that did not require sediment and then reverted to a live bed model for three sediment tests.

To ensure that the diversion can deliver 75,000 cfs when the river is at 1,000,000 cfs, the gate head loss was measured for a range of operating conditions with the river at 1,000,000 cfs. For each test, the river flow was set at 1,000,000 cfs and the diversion flow was at 40,000, 58,000 or 75,000 cfs. Gates were 10, 15, 20, 25, 30 ft or fully open. Tests were run with the open gates equally open as follows:

- All gates operational
- Gate 1 closed, gates 2 and 3 with variable opening
- Gate 2 closed, gates 1 and 3 with variable opening
- Gate 3 closed, gates 1 and 2 with variable opening
- Gates 1 and 2 closed, gate 3 fully open
- Gates 1 and 3 closed, gate 2 fully open
- Gates 2 and 3 closed, gate 1 fully open

During fixed bed model testing the following data was collected.

- Water surface elevation was measured at eight locations in the river and 12 locations in the conveyance channel about 12 times during the tests.
- River flow and diversion flow was monitored continuously throughout each test.
- Water velocity profiles were measured in the river.

Flow pattern testing with dye injection was used to determine the flow patterns associated with changes in the upstream and downstream guide wall angle. **Figure 3.3-6** shows the diversion and the position of the guide walls during testing. Each guide wall was rotate +/- 5 degrees (blue lines) and +/- 10 degrees (red lines) from the baseline location (green lines) shown in **Figure 3.3-6**. All guide wall position combinations (25 combinations) were tested at four river flows, 600,000, 750,000, 900,000, and 1,000,000 cfs.

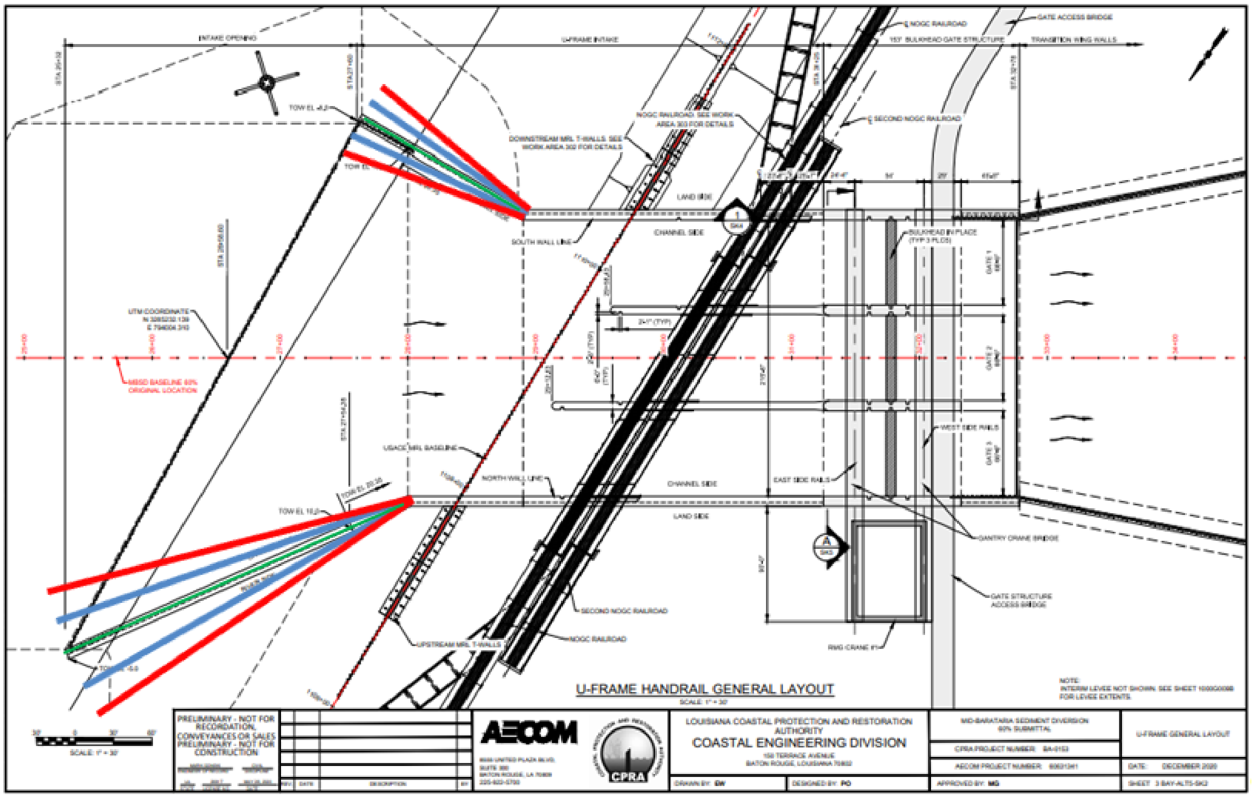

Figure 3.3-5: Guide wall locations during testing.

For each test, the river flow, water level, and the diversion flow were established. The guide wall locations were then adjusted. For each guide wall location, the flow field was allowed to stabilize and then about 2 minutes of video was collected from four overhead cameras. Each video recording was used to collect one representative image showing the flow patterns. The flow pattern images were combined into four files, one file for each river flow. In addition, a digital scale was added to each image to allow the comparison of flow patterns for various guide wall configurations where the river flow is the same. **Figure 3.3-7** shows an example of the dye testing. A detailed presentation of the dye testing is included in Alden, 2021.

Figure 3.3-6: Camera 1, 1,000,000 cfs, baseline guide wall positions.

The test matrix for flow patterns is shown in **Table 3.3-7**.

Table 3.3-7: Flow Pattern Test Matrix

Test Series	Nominal River Flow (cfs)	Nominal Diversion Flow (cfs)	Nominal River level (ft)
1 to 25	1,000,000	75,000	7.33
26 to 50	900,000	75,000	6.72
51 to 75	750,000	69,000	5.79
76 to 100	600,000	46,750	4.11

Cofferdam testing was primarily completed at 1,000,000 cfs river flow. The CMAR provided drawings of the cofferdam, and Alden fabricated the coffer cells for installation in the model. The coffer cells were made of sheet metal and could be pushed into the riprap. The large coffer cells are 63 feet in diameter and the center to center distance between adjacent cells is 81 feet. The CMAR was primarily responsible for documenting the data collected by Alden.

Cofferdam testing data collection was primarily qualitative in the form of video and still photos of dye injected into the model. In addition, Alden made velocity measurements near the cofferdam and at the location of the proposed trestle; data was provided to the CMAR. After the testing with the CMAR was complete, Alden measured the average water velocity across the width of the river with and without the cofferdam in place.

Cofferdam installation testing was completed at 1,000,000 cfs river flow. The impact of the cofferdam on the river water velocity was measured at a river flow of 600,000, 750,000, and 1,000,000 cfs.

River Model Invert EL -25 Live Bed Testing

Live bed testing of the model with the invert EL -25 was used to evaluate system performance for sediment diversion. Three additional sediment tests were completed in the invert EL -25 design testing with three gates. The tests are described in **Table 3.3-8** and **Table 3.3-9** where they are shown as tests 11, 12, and 13 and are shown in conjunction with the previous testing. The previous testing informed the testing program for the revised design and is therefore shown.

Table 3.3-8: River Model Test Conditions

Test ID	Start Date	Target Values				Gates Open	Diversion Invert
		Duration (hrs)	River Flow (cfs)	Diversion Flow (cfs)	Tap 2 WSE (ft)		
1	9/13/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
2	10/17/19	12	750,000	69,000	5.79	4 @ 45 ft	-40 ft
3	10/25/19	12	1,000,000	75,000	7.33	4 @ 34 ft	-40 ft
4	11/7/19	12	600,000	46,750	4.11	4 @ 40 ft	-40 ft
5	11/19/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
6	12/3/19	12	900,000	75,000	6.72	4 @ 40 ft	-40 ft
7	12/11/19	48	900,000	75,000	6.72	4 @ 40 ft	-40 ft
8	1/9/20	12	600,000	46,750	-2.50	4 @ full open	-40 ft
9	1/27/20	48	750,000	69,000	5.79	4 @ 45 ft	-40 ft
10	2/26/20	12	900,000	60,000	6.72	4 @ full open	-25 ft
Revised Design Testing with Three Gates and Invert EL -25							
11	5/12/21	12	900,000	75,000	6.72	3 @ full open	-25 ft
12	5/20/21	12	1,000,000	75,000	7.33	3 @ full open	-25 ft
13	6/1/21	12	750,000	69,000	5.79	3 @ full open	-25 ft

Table 3.3-9: River Model Test Goals and Notes

Test ID	Duration (hrs)	River Flow (cfs)	Notes
1	12	900,000	Model validation to Allison (2011) data
2	12	750,000	Determine sediment diversion ratio
3	12	1,000,000	Determine sediment diversion ratio
4	12	600,000	Determine sediment diversion ratio
5	12	900,000	Repeat of test 1 with additional measurements
6	12	900,000	Change in intake armoring approach alignment
7	48	900,000	48-hour test with same conditions used in Test 1 and 5.
8	12	600,000	Repeat of Test 4 but with lower water level to test ability manipulate suspended sediment concentration with small changes in water level.
9	48	750,000	48-hour test with same conditions as test 2
10	12	900,000	Diversion invert was changed to -25 ft without adjusting width.
Revised Design Testing with Three Gates and Invert EL -25			
11	12	900,000	Determine sediment diversion ratio
12	12	1,000,000	Determine sediment diversion ratio
13	12	750,000	Determine sediment diversion ratio

3.3.7 Results

Select model results are presented in the following section. A detailed presentation of all model results is given in Alden, 2020 and Alden 2021.

Gate Head Loss

Gate head loss results are only given for the invert EL -25 diversion. For all testing, the river flow was about 1,000,000 cfs with an average water level between EL 7.38 and EL 8.40 NAVD 88. The diversion flow was set using the pump at the downstream end of the diversion channel. The head loss was then measured for a range of gate openings.

The upstream water surface elevation was taken as the water surface elevation at Tap #4 (**Figure 3.3-2**). The water velocity profile was measured at Tap #4 to define the velocity head and compute the total head from the static and dynamic components. The total head downstream of the gates was determined at the downstream end of the conveyance channel expansion.

Head loss for four operating conditions was calculated, and **Figure 3.3-8** shows results for the all gates open condition. To assess the repeatability of the testing, **Figure 3.3-8** shows the results of three repeat tests at diversion flows of 40,000, 57,500, and 75,000 cfs with all gates open. Reproducibility is about within 0.1 feet (prototype). The calculated measurement of uncertainty of the head loss can be estimated from the uncertainty in the water level measurement and velocity measurement. The estimated experimental uncertainty in the head loss testing is less than 0.15 feet. Additional results are in Alden, 2021.

Gate head loss testing shows there will be up to 2.2 feet of head loss through the structure when the river flow is 1,000,000 cfs. This is an increase of 0.75 feet from the invert EL -40 design.

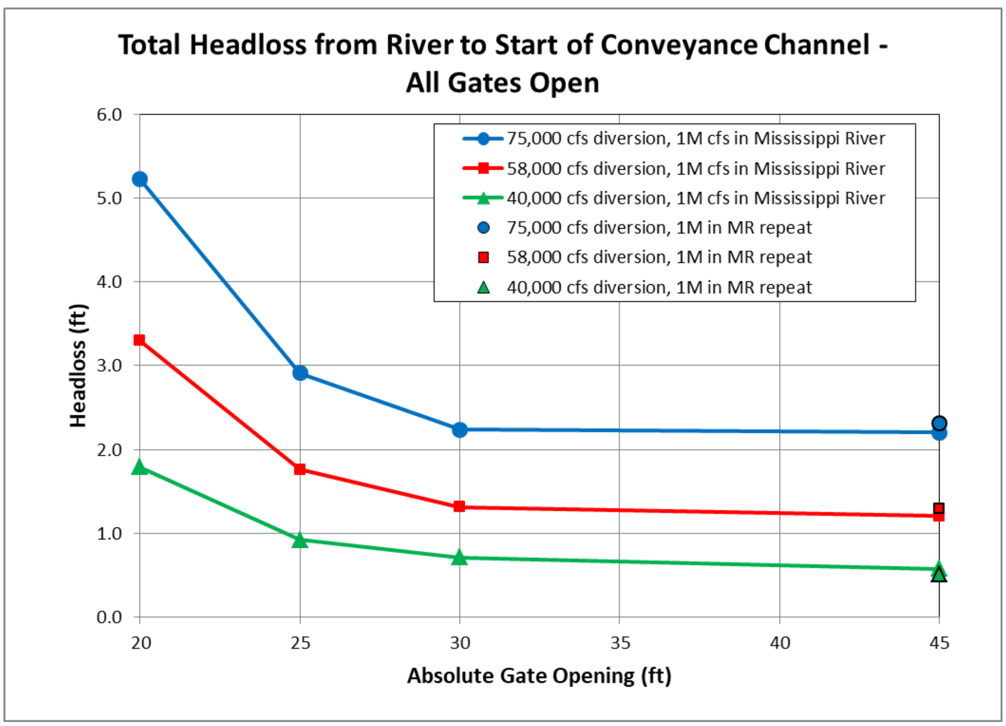

Figure 3.3-7: Total head loss through the diversion with all gates open.

Riprap Stability

Throughout the head loss testing, observations of riprap movement were made. For all of the physically possible test conditions, the riprap did not move. Additional riprap stability testing is ongoing to determine riprap stability when the riprap is disturbed.

Flow Pattern Testing

Guide wall angle testing was completed for 25 combinations of upstream and downstream guide wall locations as shown in **Figure 3.3-6**. Each guide wall has a hinge point and baseline location as shown in **Figure 3.3-6**. The guide wall was deflected +/- 5 degrees and +/- 10 degrees from the baseline location. During each test, four video cameras recorded the tests. Based on the video, the best guide wall alignment was identified for each flow condition from each camera. **Table 3.3-9** shows the best performing upstream guide wall alignment for a given downstream guide wall alignment. **Table 3.3-10** shows the best performing downstream guide wall alignment for a given upstream guide wall alignment. The most favorable flow conditions for a given flow and guide wall alignment is shown in black for camera one, red for camera two, and blue for camera three. The three cameras have a different view of the intake and the photos are captured at different times. Based on the summary data shown in **Table 3.3-9** and **Table 3.3-10**, the most favorable flow patterns may not occur for the same guide wall configuration depending on which camera is evaluated.

Table 3.3-10 shows that an upstream angle of zero is best for most river flows and downstream angles. **Table 3.3-11** shows significant variability of which downstream angle is best for a given upstream angle. This occurred because the significance of the downstream wall angle on the diversion performance is negligible. Based on these observations, the baseline guide wall alignment is likely the best alignment.

Table 3.3-10: Most Favorable Flow Patterns Relative to Variable US Angle

Alignment (US/DS)	River Flow (cfs)			
	600,000	750,000	900,000	1,000,000
-10/-10			Best	
-5/-10	Best, Best	Best, Best	Best	
0/-10	Best		Best	Best
5/-10		Best		Best, Best
10/-10				
-10/-5				
-5/-5				
0/-5	Best, Best	Best, Best	Best, Best, Best	Best, Best, Best
5/-5				
10/-5	Best			
-10/0	Best			
-5/0				
0/0	Best, Best	Best, Best	Best, Best, Best	Best
5/0		Best		Best, Best
10/0				
-10/5	Best			
-5/5		Best		Best, Best
0/5	Best, Best	Best	Best	
5/5		Best		Best
10/5			Best, Best	
-10/10				
-5/10	Best	Best, Best	Best	Best
0/10	Best	Best		Best, Best
5/10			Best, Best	
10/10	Best			

Camera 1
Camera 2
Camera 3

Table 3.3-11: Most Favorable Flow Patterns Relative to Variable DS Angle

Alignment (DS/US)	River Flow (cfs)			
	600,000	750,000	900,000	1,000,000
10/10		Best	Best, Best	
5/10				
0/10	Best, Best, Best		Best	
-5/10		Best, Best		Best, Best, Best
-10/10				
10/5	Best	Best, Best	Best	
5/5				
0/5				
-5/5	Best, Best	Best		Best
-10/5			Best, Best	Best, Best
10/0				
5/0	Best	Best, Best	Best, Best	Best, Best
0/0		Best	Best	
-5/0				
-10/0	Best, Best			Best
10/-5	Best			
5/-5	Best	Best, Best		
0/-5				Best
-5/-5				Best, Best
-10/-5	Best	Best	Best, Best, Best	
10/-10				
5/-10	Best			
0/-10			Best, Best	
-5/-10		Best	Best	
-10/-10	Best, Best	Best, Best		Best, Best, Best

Camera 1
Camera 2
Camera 3

For each test, the head loss through the gate structure was also determined. Additional data about the head loss results is presented in Alden 2021.

Velocity Data

For seven of the 100 guide wall alignments and each of the three live bed tests, velocity measurements were made at 12 locations near the intake. Measurements were made using the micro-ADV. The measurements were taken at 60% depth (water surface is at 0%), which is about equal to a depth-averaged velocity. The 12 measurement locations approximately correspond to velocity measurement locations sampled by FTN in their numerical model simulation to within about 50 prototype feet.

Velocity measurements were made for Mississippi river flows of 750,000 cfs, 900,000 cfs, and 1,000,000 cfs. For the 750,000 cfs test the diversion flow was about 69,000 cfs and for the 900,000 and 1,000,000 cfs river flow the diversion flow was approximately 75,000 cfs. Measured values are presented in Alden, 2021.

Cofferdam Testing

Construction of the diversion will require the construction of a cofferdam. **Figure 3.3-9** shows the cofferdam layout relative to the diversion. The testing was guided by the CMARs representatives (Joe Schwenk, Jim Gardner, and Jim Beckerle) who were on site for three days during the testing. There were two components of the testing: 1) Local velocities around the coffer cells as construction proceeds, and 2) the water velocity across the river width when the cofferdam is complete.

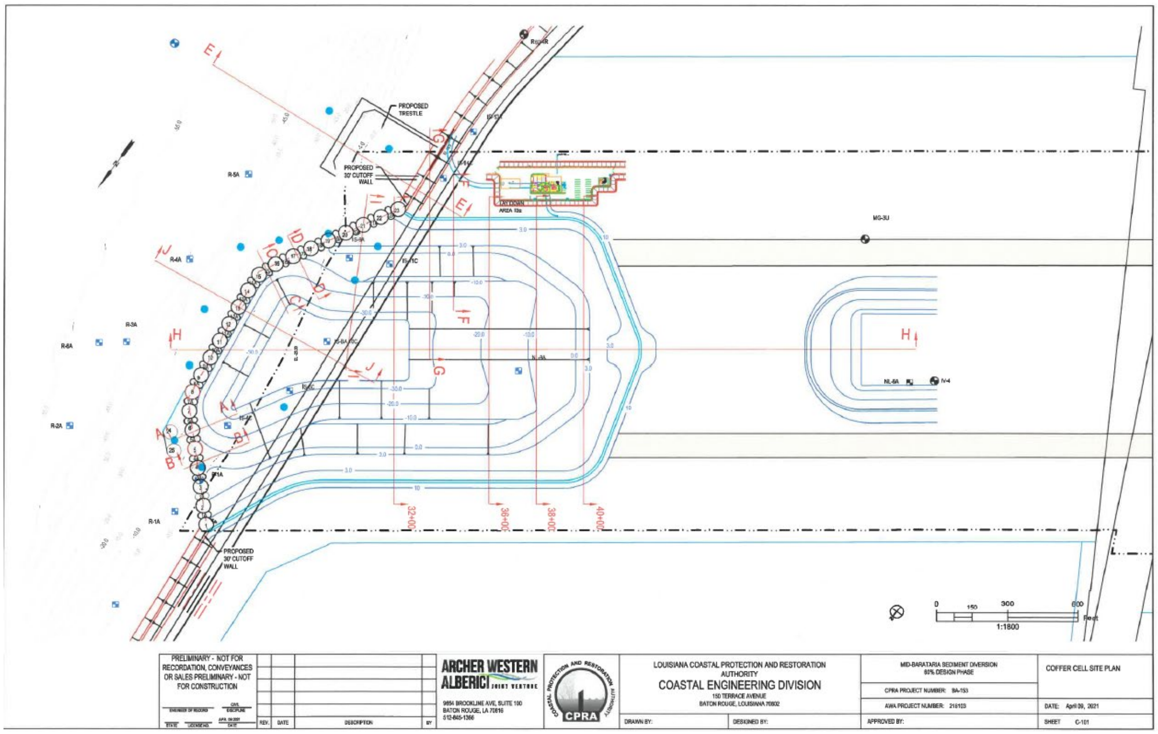

Figure 3.3-8: Cofferdam layout overview.

The cofferdam will be constructed sequentially starting with cell #1 and progressing through cell #23. In the physical model, the cofferdam was constructed following the prototypical sequence. Throughout construction/installation, velocity measurements were made at three locations between cells 11 and 12 on a line perpendicular to the axis of the cofferdam. The river flow during installation of the cofferdam was 1,000,000 cfs. Testing results showed that the maximum velocity occurs when the barges are removed and all of the coffer cells are in place. The velocity increases with distance from the coffer cells to a maximum of 3.23 ft/s when the barges are removed. The barges provide a more favorable condition (lower velocity) for installation of the cofferdam. Complete results are presented in Alden, 2021.

A trestle located at the downstream end of the cofferdam is required for construction. The water velocity was measured at five locations along the outer edge of the trestle. The maximum velocity was 1.63 ft/s and was recorded at the upstream end of the trestle.

Velocity measurements were made across the width of the river with and without the cofferdam in place. The purpose of the measurements was to determine the impact of the cofferdam on the velocity profile in the river. Velocity measurements were made along a line perpendicular to the axis of the middle section of cofferdam for river flows of 1,000,000 cfs, 750,000 cfs, and 600,000 cfs. The measurements started 2.7 feet from the face of the cofferdam and extended across the river in 20 equal intervals of about 108 feet each. Results are shown in **Figure 3.3-10** facing downstream. Each individual measurement is a 16-minute (prototype) time averaged velocity.

The plot shows there is minimal difference with and without the cofferdam at a river flow of 600,000 cfs. At a river flow of 750,000 and 1,000,000 cfs there is a significant variability in the time averaged velocity of adjacent points in areas of high velocity. Near the cofferdam, the velocity profile with and without the cofferdam is approximately equal. The measurement variability 250 feet to 1750 feet from the left bank shows that there is more turbulent fluctuation in the velocity measurement than there is impact from the cofferdam. The cofferdam does not appear to have a significant impact on river velocities.

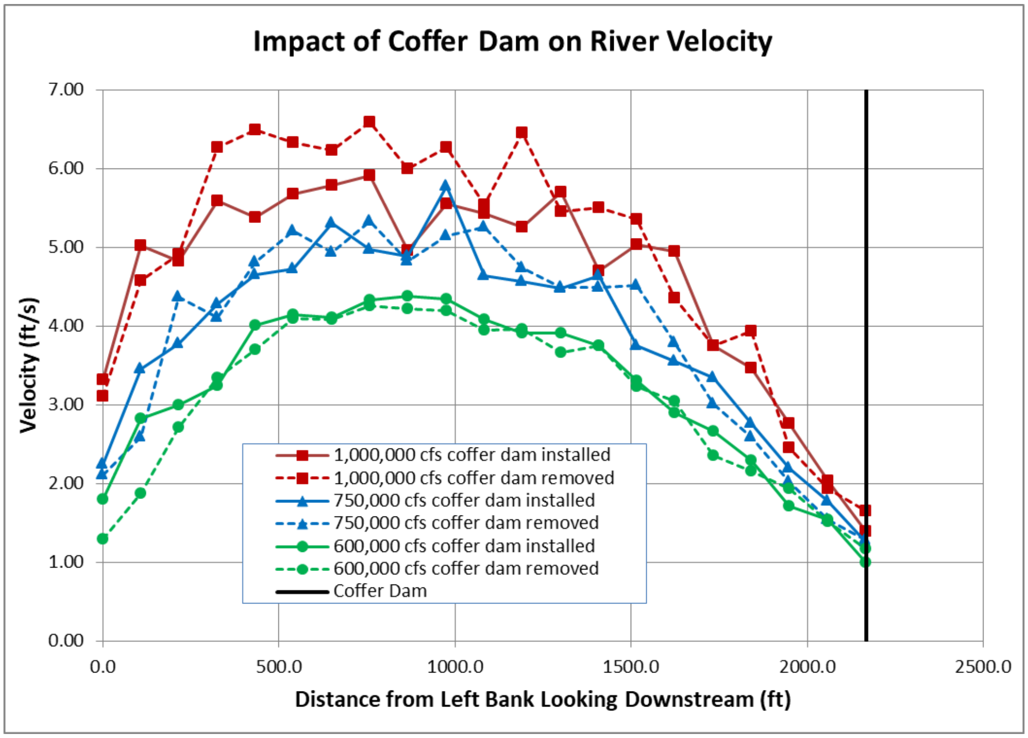

Figure 3.3-9: Velocity profile with and without cofferdam.

<u>Sediment Water Ratio</u>

One objective of the modeling was confirmation of the sediment water ratio (SWR). The SWR is the ratio of sediment concentration in the diversion divided by the sediment concentration in the river. A SWR less than one means that the sediment concentration in the diversion is less than the sediment concentration in the river. Based on numerical modeling and analysis of field data, FTN concluded that particles smaller than about 63 microns are approximately uniformly distributed in the river. However, the distribution of sand is non-uniform, with a vertical and lateral variation in concentration. By design, the physical model did not include mud sized sediment.

A calculation of the SWR relative to MGup1, MGup2, MGup3, and MGup 4 can be made by using the SSC measurements. **Figure 3.3-11** shows the SWR for each test as a function of river flow. A trend for decreasing SWR with increasing river flow is shown. Additional analysis discussion is in Alden, 2021.

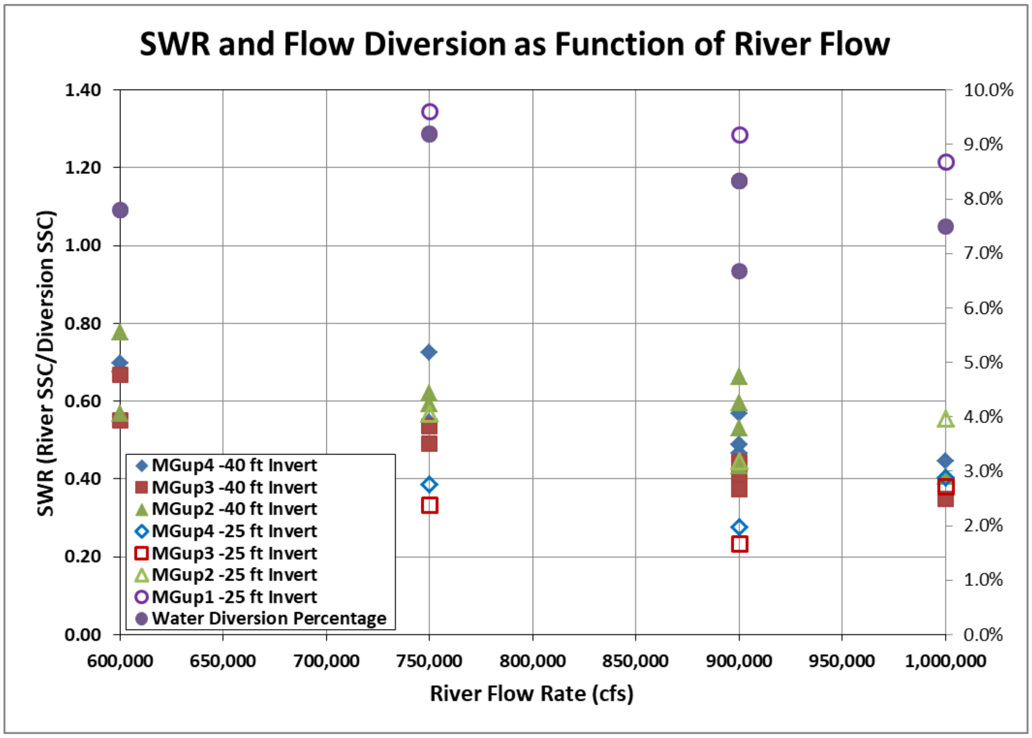

Figure 3.3-10: SWR and flow diversion as a function of river flow.

Computing the SWR relative to the river concentration at MGup1, MGup2, MGup3, and MGup4 shows significant variation due to large differences in the suspended sediment concentration across the width of the river. Further analysis showed that the physical model likely over represents the sediment concentration towards the center of the river. A detailed discussion of the analysis is in Alden 2021. The prototype SWR is likely to be close to the model SWR relative to MGup1.

River Bedforms

At the end of each test, the river bed was laser scanned and the trough to peak bedform size was determined. **Figure 3.3-12** shows the model results.

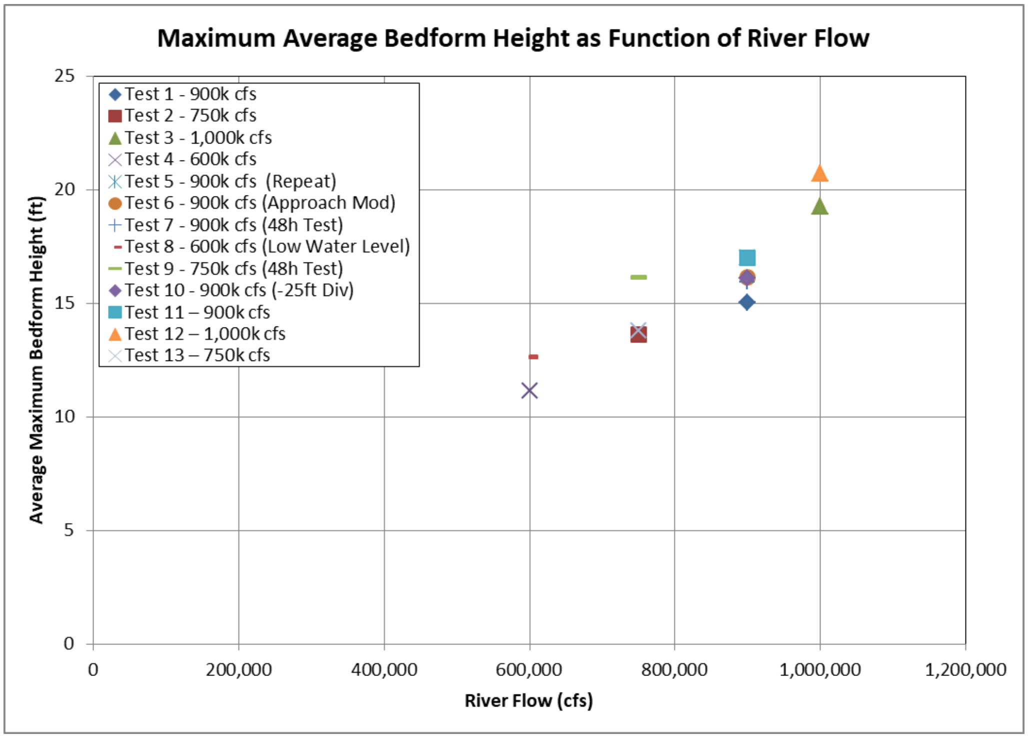

Figure 3.3-11: River flow vs. average bedform height upstream of diversion.

River Morphology

Laser scan data was used to determine area of potential chronic sediment deposition. A minor depositional trend was noted immediately downstream of the diversion on the right bank. Additional information is provided in Alden 2021.

Conveyance Channel Head Loss

The conveyance channel model was run at flows of 40,000 cfs, 57,500 cfs, and 75,000 cfs without sediment in the system. The water surface elevation was measured at nine locations along the length of the channel. For each test, water surface elevation was plotted as a function of distance along the channel and a linear regression was used to determine the slope of the hydraulic grade line. The slope of the energy grade line was used to calculate Manning's n for each flow condition. Results are shown in **Table 3.3-12**.

Table 3.3-2: Manning's n as a Function of Discharge

Flow (cfs)	Manning's n
40,000	.033
57,500	.031
75,000	.029

Comparison of the measured head loss with five predictive equations (Manning-Strickler, Keulegan, Key, Bathurst, and Ferguson) showed that Ferguson is the best predictor of channel roughness for this system. Additional detail is given in Alden 2020.

When sediment is added to the system, the head loss generally decreases, with some notable outliers. **Figure 3.3-13** shows Manning's n as a function of channel flow for all of the tests. Additional analysis is in Alden 2020.

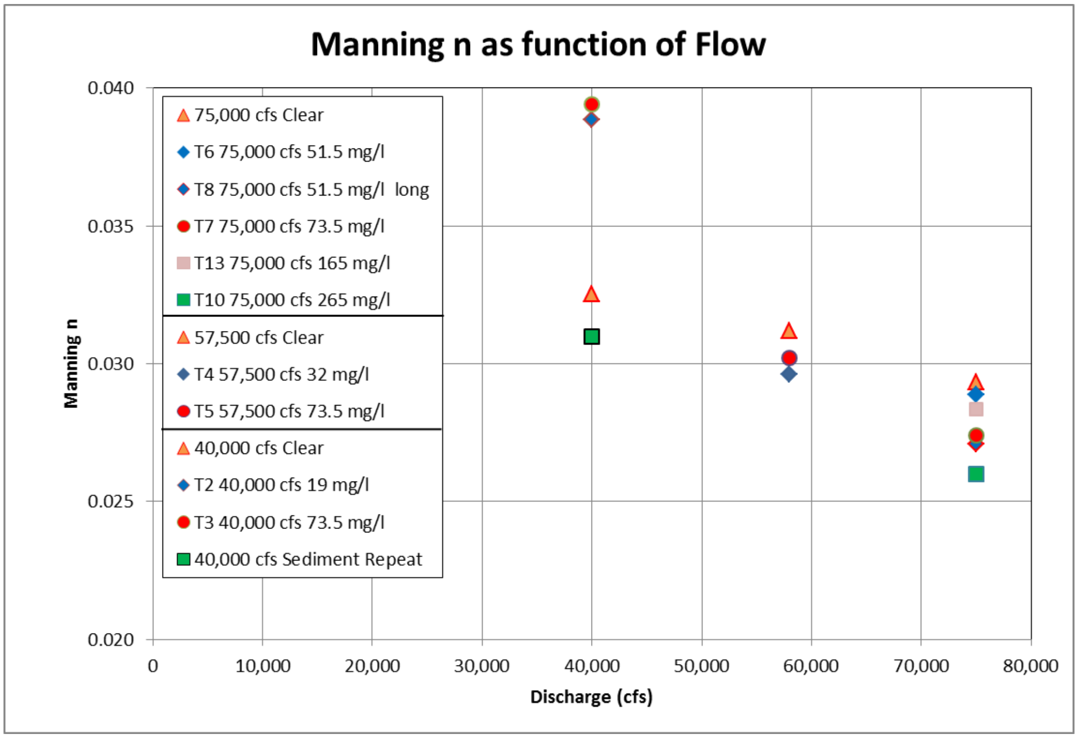

Figure 3.3-12: Manning's n as a function of discharge with sediment.

<u>Conveyance Channel Velocity Measurements</u>
Velocity measurements showed a symmetrical velocity distribution in the channel. Results also showed a vertical velocity profile that evolves along the length of the channel. Additional information is provided in Alden 2020.

<u>Sediment Accumulation</u>
During periods of low flow with high sediment concentration, sediment accumulation at the upstream end of the conveyance channel was observed. However, the model also showed that at higher flows the sediment would be scoured, and the conveyance channel would self-clean.

<u>Conveyance Channel Riprap Stability</u>
Two tests were completed to evaluate riprap stability for beyond design basis conditions. The first test involved lowering the tailwater as low as possible and increasing the model flow as high as possible. The test was run for approximately one hour and no channel riprap movement was observed. The flow meter measurement limit is 104,000 cfs. During the test, the meter recorded 104,000 cfs, indicating a flow of 104,000 cfs or greater. The basin water level was EL 1.64 NAVD88. The basin water level for a typical 75,000 cfs test is approximately 3.1 feet. The computed section averaged velocity in the channel was between 8.4 and 10.3 ft/s, about 40 to 50% greater than the velocity for the design basis flow of 75,000 cfs.

The second test was requested by CPRA to determine the riprap stability at the design flow after it has been disturbed. Five disturbances were created along the length of the channel. The disturbances were far enough apart such that they did not influence each other. No riprap movement was observed.

3.4 Interior Drainage

The Mid-Barataria Sediment Diversion is located within an approximately 7,830-acre portion of land bounded on the east by the Mississippi River Levee and on the west by an existing back levee and coastal marsh/waters. The northern and southern boundaries are defined by natural ridges in topography. The entire drainage basin is of the forced drainage type, in which all flow proceeds to the Wilkinson Pump Station in the southern portion of the basin where it is pumped out into the coastal marsh/waters area.

The MBSD project will effectively bisect the above described drainage basin, creating two hydraulically disconnected northern and southern basins. It is a requirement of this project that all the flow from both basins continue to flow to the Wilkinson Pump Station, as is currently the case. This requirement will be met by the installation of an inverted siphon below the diversion channel, allowing flow from the north basin to proceed south below the diversion channel and into the south basin and then to the Wilkinson Pump Station.

Overall, the interior drainage portion of the design includes the sizing and/or design of the following items:

- Inverted Siphon
- Sluicegate to Drain Isolated Northern Basin
- Diversion Guide Levee Parallel Ditches
- Drainage Design for the Hwy 23 Over Mid-Barataria Bridge.

All the areas of design listed above are based on and supported by the Interior Drainage Report included in **Appendix B** of this DDR.

3.4.1 Inverted Drainage Siphon

3.4.1.1 General Description

The inverted drainage siphon will convey flow from the northern side of the diversion channel to the southern side, beneath the new MBSD channel, from where it will continue to the Wilkinson Pump Station to ultimately be discharged into the coastal marsh areas of Barataria Bay. The level of service requirement of the inverted drainage siphon has been established to be the conveyance of a 10-Year, 24-hour storm event with less than a 0.1-foot increase in the water surface elevation within the drainage basin.

In the preliminary design phase, when the DT performed alternatives screening and analyses, a 25-Year rain event was assumed. As design progressed, it was determined that the Wilkinson Pump Station was designed to match the capacity of the original pump station, and it was not designed for a specific rain event. The DT determined this capacity to be slightly less than a 10-Year event; therefore, the design criteria for the interior drainage design was updated to a 10-Year rain event.

3.4.1.2 Hydraulic Sizing

The inverted drainage siphon hydraulic sizing results in a required bank of six, eight-foot nominal diameter tubes, each approximately 800 feet long passing below the diversion channel. As required, the design allows for the conveyance of a 10-Year, 24-hour storm with less than a 0.1-foot increase in the water surface elevation anywhere within the north basin as compared to the pre-diversion conditions.

Total flow through the inverted siphon under this condition will be approximately 780 cfs with a velocity within the individual tubes of 2.59 fps each. The total head loss across the inverted siphon is 0.31 feet. Details regarding the inverted siphon structural and mechanical designs can be found in Section 5.8 of this DDR.

3.4.2 Drainage Structure for Isolated Northern Basin

The installation of the diversion will hydraulically confine a portion of the northern basin between the new diversion levees, the existing back levee, and the new NOV levee preventing the area from draining into either the new inverted siphon or to the Wilkinson Pump Station via the Back Levee Canal. In order to drain this area, a drainage structure was proposed to penetrate the NOV-NF-W-05a.1 Levee and allow that area to drain into the Timber Canal, just upstream of the inverted siphon, from where it will proceed below the MBSD channel and into the Wilkinson Pump Station to ultimately be discharged into the coastal marsh areas of Barataria Bay. A detailed description of this area can be found in the Interior Drainage Report included as **Appendix B**.

The drainage structure was sized to minimize the increase of the water surface elevations within the subbasins and to minimize the time required to drain the subbasins post diversion when compared to the existing conditions. As with the rest of the interior drainage, the sizing of the drainage structure was based on flows seen during the 10-year, 24-Hour storm. The hydraulic sizing requires a drainage structure which includes 2 (two) 6-foot x 6-foot box culverts. The culverts will have flap gates or other back flow prevention to prohibit flow from entering the isolated subbasins from the Timber Canal. Details regarding the sluice gate structural and mechanical designs can be found in **Section 5.11** of this DDR.

3.4.3 Diversion Guide Levee Parallel Ditches

3.4.3.1 General Description

The diversion guide levee parallel ditches include two new ditches running parallel to the toe of the new diversion levees, one on the north side and one on the south side. These ditches are sized to convey flows generated by the 10-Year, 24-hour storm from all areas/channels whose drainage pattern has been directly disrupted by the new diversion channel. The new ditches will redirect the flow, routing their runoff to either the new inverted siphon suction bay, in the case of the northern channel or just downstream of the inverted siphon in the case of the southern channel. The flow will then be conveyed to the Wilkinson Pump Station for ultimate discharge into the coastal marsh areas of Barataria Bay.

3.4.3.2 Hydraulic Sizing of Diversion Guide Levee Parallel Ditches

The hydraulic sizing of the diversion guide levee parallel ditches results in the following design requirements shown in **Table 3.4-1**. See **Appendix 8** of the Interior Drainage Report, included as **Appendix B** of this DDR, for calculations.

Table 3.4-1: Properties of Guide Levee Ditches

Parameter	North Parallel Ditch	South Parallel Ditch
Flow Capacity	192.4 cfs	119.3 cfs
Required Section	59.28 sf	40.41 sf
Velocity	3.25 fps	2.95 fps
Bottom Width	5 ft	5 ft

Side Slopes	3H:1V	3H:1V
Flow Depth	3.69 ft	2.93 ft
Freeboard Required	1 ft	1 ft
Total Channel Depth	4.69 ft	3.93 ft
Total Channel Width	27.14 ft	22.58 ft

Also included in the construction of the guide levee parallel ditches is the ancillary drainage modifications required to maintain existing flow patterns in the existing interior drainage ditches. This will include regrading the channels which would, under existing conditions, cross the new diversion channel. These existing ditches will be cut off; therefore, they will need to be regraded to provide positive drainage into the newly installed parallel ditches, matching inverts, so their flows can eventually discharge at the Wilkinson Pump Station.

3.4.4 Drainage Design for the Hwy 23 Over Mid-Barataria Bridge

3.4.4.1 General Description

Drainage design for the Hwy 23 over Mid-Barataria bridge consists of drainage modifications associated with installation of a new bridge spanning the MBSD that will allow vehicular travel across the channel. A separate drainage report was prepared for submission to Louisiana Department of Transportation and Development as part of the review process for the bridge. The drainage report associated with the Hwy 23 over Mid-Barataria bridge can be seen in **Appendix 9** of the Interior Drainage Report, included as **Appendix B** of this DDR.

4. GEOTECHNICAL DESIGN

4.1 General

The AECOM's Design Team (DT) performed the geotechnical engineering for the project's permanent structures. Temporary structures are being designed by the Construction Manager at Risk (CMAR).

The DT published a Geotechnical Data Report in July 2021 that included results of the soil design parameters that were approved by CPRA and the US Army Corps of Engineers (USACE). This data report forms the basis of our 60% design.

Detailed figures and calculation packages for the project features designed at this stage are included in our 60% Geotechnical Engineering Report dated July 2021. The geotechnical data gaps that were identified in the 30% design phase have been filled in with additional data. Additional data was obtained through geotechnical exploration programs performed in 2019 and 2020.

4.2 Headworks

The Headworks is comprised of the Intake Structure, the U-frame and Gated Structure. The intake will be constructed in the open and dewatered excavation made to about EL -35 (invert at EL -25). As the design progressed from the draft 30% design (November 2019) to the 60% design, the intake raised the invert from EL -40 to EL -25. This higher intake elevation results in less settlement, lower lateral loads, and less demands on stability. The Geotechnical Engineering Report, included as **Appendix C** of this DDR, includes calculations and figures considering the EL -25 intake elevation. The 60% Geotechnical Engineering Report also includes detailed presentation of soil parameters/stratigraphy and the following geotechnical analyses of each component of the headworks: axial pile load capacity, downdrag, lateral pile resistance, axial pile stiffness, lateral earth pressures, unbalanced load analysis (global stability), settlement, and seepage cutoff design.

Within the cofferdam, slopes of the open excavation will be about 1V:6H. During collaboration with the CMAR, the DT considered that backfilling will be performed in the dry excavation and that re-watering to allow the groundwater to return to its original elevation (EL 0 ±) will occur over several years. Placement of backfill in the dry over soft native soils will generate settlements and lateral loads/bending on piles.

The DT performed finite element modeling using FLAC to evaluate settlements and bending in the piles supporting the U-frame and Gate Structure. The DT also considered controlled re-watering to maintain backfill in a buoyant condition which would reduce settlement and lateral loads. The CMAR has stated that this is a viable alternative. **Appendix C** includes results of finite element modeling using FLAC.

The geotechnical design considered constructability with regard to earthwork. A sand drainage wedge will be installed on the retained side of the U-frame and Gated Structure walls from approximately EL -25 to EL 0. Refer to **Figures 4.2-1** and **4.2-2** for plan and cross-section views, respectively. These figures are referred to in the plans as "Phase II" for the Headworks Backfilling. This sand fill allows for a more efficient structural design of the walls. This sand fill is described as a wedge because it is of limited extent on the retained side of the walls with the remainder of the backfilled material within the excavated cofferdam being compacted local backfill (i.e., reused material from the excavated soils).

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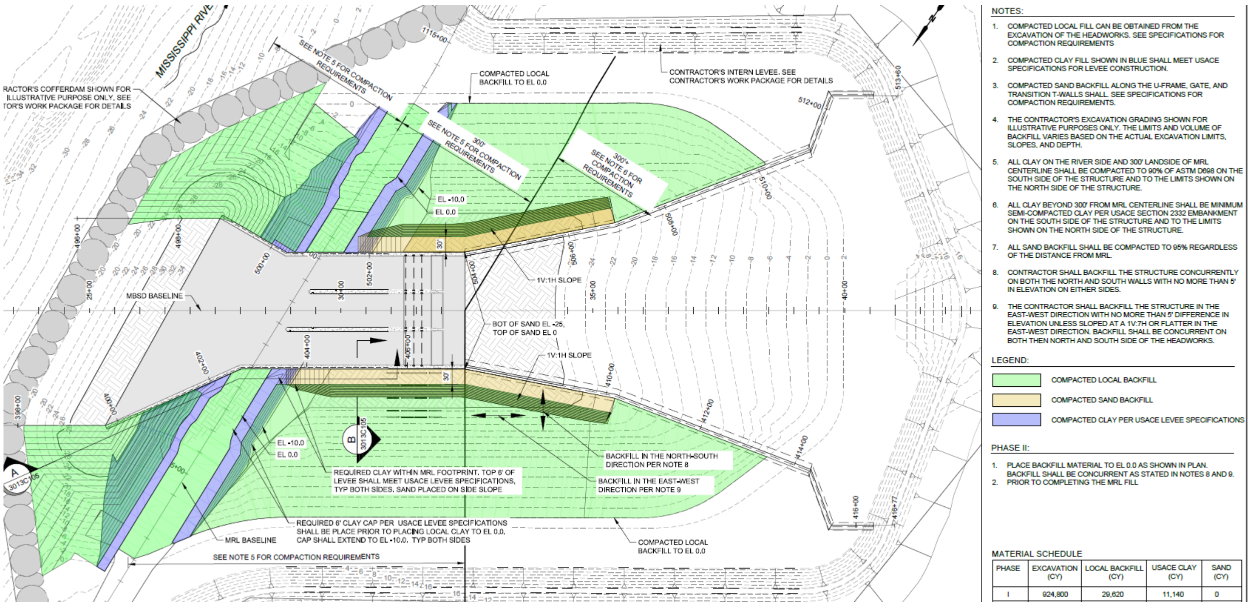


Figure 4.2-1: Sand Backfill Adjacent to U-frame and Gated structure Walls – Phase II (shown in yellow)

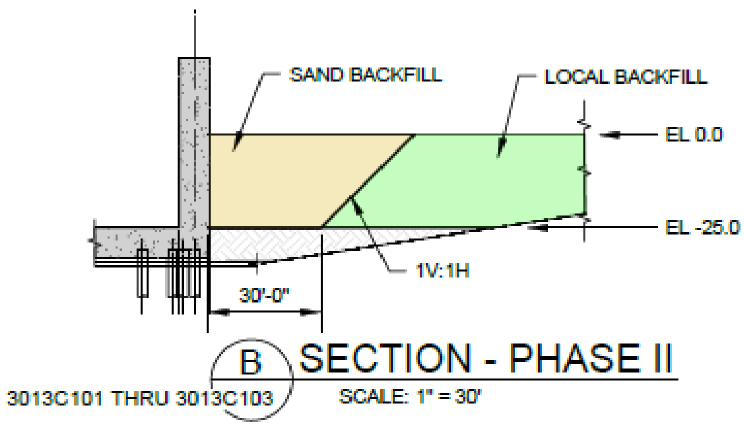


Figure 4.2-2: Sand Backfill Adjacent to U-frame and Gated structure Walls – Phase II (shown in yellow)

The final phase of backfilling in the headworks area is referred to as Phase III in the plans. Refer to Figures 4.2-3 and 4.2-4 for plan and cross-section views, respectively. Note that the sand backfill against the concrete structures is covered with compacted local backfill to prevent erosion/scour of this sand backfill.

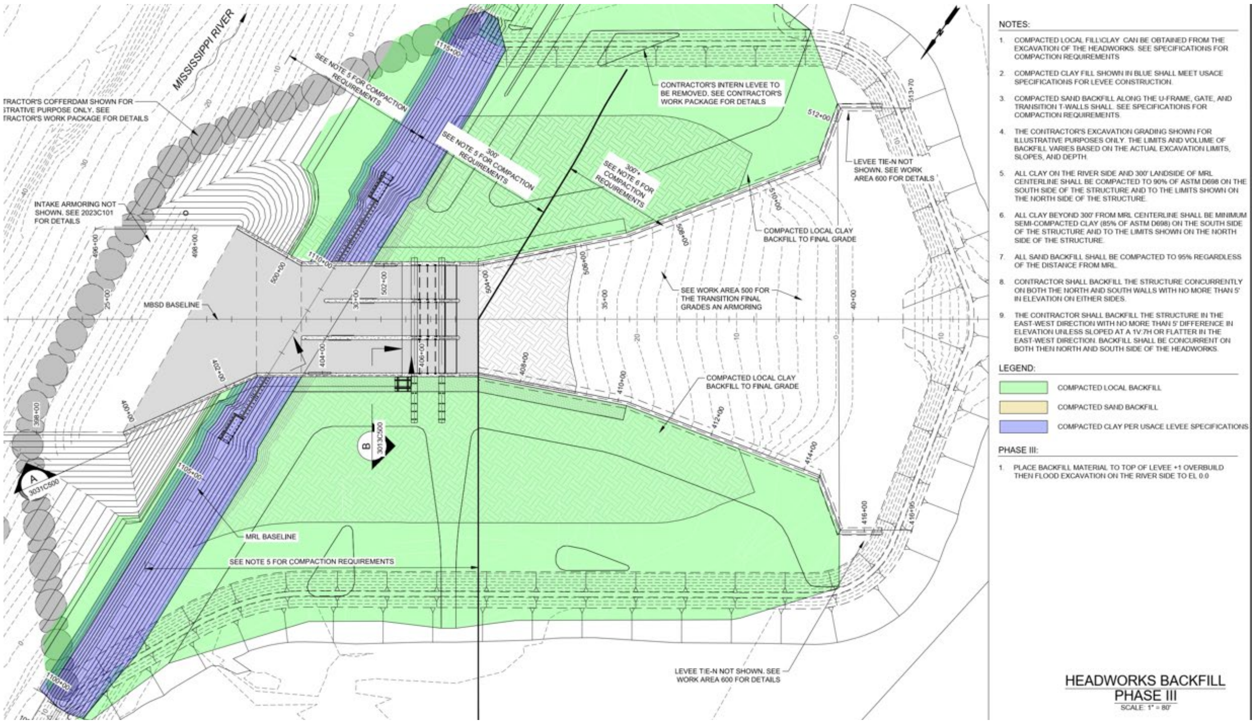


Figure 4.2-3: Compacted Local Backfill – Phase III (plan view)

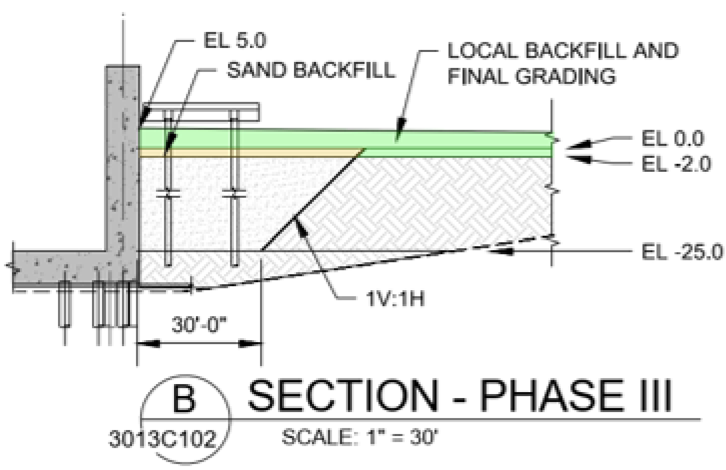


Figure 4.2-4: Compacted Local Backfill – Phase III (View at U-frame and Gated Structure Walls)

Scour protection is needed below the U-frame where it enters the Mississippi River and where the U-frame meets the Transition Channel. The protection will be a sheet pile wall extending from the base of the U-frame to a tip elevation below the expected scour depth that provides adequate embedment. The scour depth was based on degradation modeling, and the size and tip elevation of the sheet pile were conservatively selected. The toe wall is designed as a tie-back wall with the top of sheet restrained by the U-Frame and embedded sufficiently below the theoretical scour depth. The DT recommends sheet pile extending 35 feet below the U-frame invert where it meets the Mississippi River bottom (i.e., tip at EL -76) and 35 feet below the Gate structure invert where it meets the transition section (i.e., tip at EL -76).

4.3 Mississippi River Levee (MRL)

The Mississippi River Levee includes the earthen levee section and the connecting floodwalls (T-walls) that will form a continuous line of riverine protection after the MBSD project is constructed. Construction of the U-frame will require removing the existing MRL within the limits of the temporary excavation. New T-Walls are planned to replace the existing MRL for a limited extent upstream and downstream of the U-frame. The T-Walls will be constructed in-the-dry and consist of monoliths to the north side (upstream) for a length of about 175 feet and south side (downstream) of the U-frame for a length of about 200 feet. However, most of the reconstructed alignment of riverine protection along the MRL within the extent of the excavated cofferdam will comprise compacted backfill. The T-Walls will be built above the degraded levee, remaining above the MR flowline. Near the U-frame, the T-Walls will be constructed atop 35± feet of clay backfill. Please refer to the 60% Geotechnical Engineering Report, included as **Appendix C** in this DDR, for detailed presentation of soil parameters/stratigraphy and geotechnical analyses of the MRL and adjoining T-walls when considering an intake invert at EL -25.

Consideration for Shoaling. The numerical river model indicates deposition (also referred to as “shoaling”) will occur approximately one mile downstream along the right descending bank. Sediment deposition geometry predicted by the numerical model then was analyzed geotechnically to confirm the factor of safety for levee stability meets USACE criteria for the low river condition. Because this deposition results in a higher river bankline, shoaling provides additional stability to the MRL when considering slip surfaces toward the river under a low river condition.

4.4 Transition

The conveyance section transitions from the gate structure with an invert at EL -25 to the typical conveyance channel section that also has an invert at EL -25. This “transition” has flood protection along both sides of the conveyance being provided by pile supported T-walls. The DT considered two representative T-Wall monoliths and developed design parameters for them. These monoliths are W-18 and W-28. We considered two design cases for each T-Wall: the construction case and the operating case. Of these, the construction case has much greater unbalanced loads (UBLs) and much greater potential settlement because the non-buoyant weight of the backfill is placed on the subgrade, and no water is inside the channel to help balance the earth pressures. In all cases analysis was performed assuming an excavation having 1V:7H slopes outside the channel starting 20 feet behind the pile cap for the T-Wall.

Refer to the 60% Geotechnical Engineering Report for detailed presentation of soil parameters/stratigraphy and geotechnical analyses of the transition T-walls. These analyses include axial pile load capacity, downdrag, lateral pile resistance, axial pile stiffness, lateral earth pressures, unbalanced load analysis (global stability), SIBM analysis, settlement, and seepage cutoff. For the construction case with backfill against the wall being in the dry, the calculated UBLs were 17.5k/ft at W-18 and 26.5 k/ft at W-28. These UBLs were too large to practically handle with piles. Consequently, DMM is recommended to carry the UBLs. DMM will also reduce settlement along the batter piles such that SIBMs are negligible. DMM panels comprising interlocking 36-inch diameter DMM columns are oriented perpendicular to the wall alignment. DMM panels are generally spaced at 10 feet on center with the width of the panels being about 55 to 65 feet and the depths ranging based on the heights of the walls. DMM is also used to provide seepage cutoff resistance for the walls instead of traditional sheet piling because of the constructability concerns.

Similar to the U-frame and the gate structure, a sand drainage wedge will also be installed on the retained side of the walls from approximately EL -25 to EL 0. This drainage wedge is limited to the tallest transition T-walls (monoliths W-1 to W-7; W-19 to W-25). This sand fill allows for a more efficient structural design of the walls. This sand fill is described as a wedge because it is of limited extent on the retained side of the walls with the remainder of the backfilled material within the excavated cofferdam being compacted local backfill (i.e., reused material from the excavated soils).

4.5 Conveyance Channel and Levee

The Conveyance Channel Levee (CCL) system is composed of two levees along each side of the Conveyance Channel which acts as a guide for the channel during operation and flood protection during high water or flood events. The CCL is considered hurricane protection between the transition T-walls and the tie-in with the NOV-NF-W-5a.1 levee. Settlement, slope stability, and seepage analyses for the CCL were performed for the hurricane design grades of EL 15.85 feet with conveyance channel side slopes of 4H:1V extending down to EL-25. The centerline of the CCL will be offset approximately 150 feet from the edge of the Conveyance Channel. This offset distance includes a 10-ft wide levee crown, levee side slopes of 4H:1V and approximately a 100-ft wide “bench” between the toe of the levee and the top of the excavated channel slope. Staged construction stability analyses were performed which included strength gain of the foundation soils. Strength gain and staged levee construction will be significantly accelerated by using prefabricated vertical drains (PVDs or wicks). Detailed analyses of settlement, strength gain and levee stability were made considering wick drains being in place. The strength gain allowed for a significant reduction in the levee section such that stability berms are not required.

<u>Closure of the Existing Timber Canal.</u> Construction of the conveyance channel levees (north and south) requires crossing the existing Timber Canal. The siphon must be in operation to direct interior drainage from the north of the MBSD project to the south and toward Wilkinson pump station prior to closing off the existing Timber Canal. This schedule requirement means that the closing off of Timber Canal occurs later in the construction schedule. With Timber Canal having grades near EL -9 versus the typical grades in the area of EL -4, staged construction of levees using wick drains and strength gain would require 6 to 7 stages (total schedule of 2 – 2½ years) to achieve the final constructed grade of EL 20 (i.e., required overbuild for the Reach 8 levees). The DT designed the levee foundation using DMM instead of a wicked foundation to allow the levee construction to be performed rapidly instead of over a 2 to 2½ year period.

<u>Guide Levees.</u> The CCL is no longer hurricane protection and will be designed solely as a guide levee for conveyance beyond the intersection of the CCL with the NOV-NF-W-5a.1 levee system (i.e., to the Barataria Bay side of this intersection). Stability and settlement analyses were performed for the guide levee that will extend from the NOV-NF-W-5a.1 levee to the outfall transition feature considering a levee crown of EL 8.2. Although not a flood protection levee, stability shall also comply with the USACE design criteria. The design grade was set at 2 feet above the higher, future conveyance flow stage (construction to about EL 10). This requires three stages for construction. Refer to the 60% Geotechnical Engineering Report for detailed presentation of soil parameters/stratigraphy and geotechnical analyses. This report also includes description of the construction sequencing assumptions that were made by the DT. This narrative is important to understanding the construction schedule and the development of the 60% Plan Drawings and Specifications.

<u>Closure of the Existing Back Levee Canal.</u> Construction of the guide levees (north and south) requires crossing the existing back levee canal. The existing back levee canal will not be crossed with the new guide levee until the siphon and the drainage structure (constructed within the NOV-NF-W-5a.1 levee) are built

and drainage is routed through these two structures. This is a similar schedule issue as the closure of the existing Timber Canal. This schedule requirement means that the closing off of the existing back levee canal occurs later in the construction schedule. With the existing back levee canal having grades near EL -10 versus the typical grades in the area of EL -4, staged construction of levees using wick drains and strength gain would likely require 4 to 5 stages (total schedule of 1 – 1½ years) to achieve the final constructed grade of EL 10 (i.e., required overbuild for the Reach 8 guide levees). The DT designed the levee foundation using DMM instead of a wicked foundation to allow the levee construction to be performed rapidly instead of over a 1 to 1½ year period.

<u>Wick Drain Test Embankments.</u> The overall design process and understanding of construction schedules was informed by two test sections that started in late 2019 and were completed in early 2021. These test sections comprise levee embankments built in stages with wick drains installed in the foundation soils to accelerate consolidation and strength gain. Measurements from the geotechnical instrumentation provided valuable information to the DT and CMAR, namely that a 3 to 4 month hold period was suitable for each stage of levee construction. Information from the wick drain test sections is included in **Appendix C** (Geotechnical Engineering Report).

<u>Upcoming Change in Design.</u> The 60% design of the conveyance channel levees was dictated by having the conveyance channel constructed in the wet that required a 4H:1V excavated side slope to maintain stability of the excavation and stability of the levee sections with respect to the adjacent excavations. During the latter part of the 60% design phase, the CMAR stated that they preferred to excavate the conveyance channel in the dry and showed that a 7H:1V side slope would be required for the excavations. A design with 7H:1V side slopes (in the dry) has lower risk from a geotechnical standpoint than a 4H:1V side slopes (in the wet). After the 60% design is submitted in early August, a detailed memorandum will be prepared by the DT to document the 7H:1V design with the supporting stability and settlement analyses.

4.6 Railroad (R/R) Bridge and Approach Embankments

The railroad bridge will span the U-frame at approximate Station 30+00. The bridge will also support a vehicle access road which will transition to the MR levee crowns on either side of the conveyance channel. The vehicle access road transitions will be supported by footings bearing on the levee. Allowable soil bearing values and slope stability analyses were performed for the vehicle access road transitions. Analyses performed for the railroad bridge include axial pile load capacities, lateral pile resistances, settlement, SIBM evaluations, downdrag loads and allowable soil bearing values as shown in **Appendix C**.

Approach fills for the Railroad embankments are located outside the general excavation and will be built above the existing subgrade near EL 4. This amount of fill will generate differential settlement between the approach fill and pile supported abutment. The DT recommends use of lightweight fill to reduce settlement to tolerable amounts. This will involve over-excavation of existing soil, placement of geotextile, lightweight fill, the normal weight fill. Details for various approach fill heights are provided in **Appendix C**.

4.7 Reservation Area

The reservation area will be located approximately 500 feet south (downstream) of the headworks. This location is outside of the area that will be excavated for construction of the headworks. This location will reduce the amount of settlement due to backfilling within the headworks excavation. This location also allows the reservation area to be constructed earlier in the schedule, rather than at the end after the headworks backfilling has occurred. The reservation area will be preloaded utilizing wick drains prior to construction to mitigate the post-construction settlement and differential settlement between grade supported and pile supported features. The wick spacing is at 10 feet (rather than the typical 5-ft spacing as with the CCLs) because this area of the site drains more rapidly and the anticipated time before pile foundation installation for the building is two years. During this 2-year timeframe, the area will be filled from EL 4 to about EL 10/11 and used as a working area for the contractor. Refer to the 60% Geotechnical Engineering Report for presentation of axial pile load capacity estimates, time-rate of settlement analyses, and downdrag settlement analyses considering this preloading period.

4.8 Highway 23 Bridge and T-walls

The Highway 23 Bridge will be located at approximate Station 63+25 (where bridge centerline meets MBSD baseline). The bridge will span the Conveyance Channel. Currently 18 bents are planned for support of the bridge. The bents within the conveyance channel (Bents 8 through 11) will utilize a footing-column foundation system. Earthen approach ramps will require a preload surcharge with wick drains to limit settlement at the abutments. Design and construction of the bridge will conform to standard requirements of the LaDOTD. Analyses performed to support the Highway 23 Bridge include axial and lateral pile capacities, downdrag analyses, settlement computations, pile group analyses, design of the approach ramps, and pavement recommendations. The DT performed analyses to support the Highway 23 T-walls that continue the line of protection for the conveyance channel levee system beneath the Highway 23 Bridge. As presented in the 60% Geotechnical Engineering Report, allowable axial pile capacities, estimates of settlement induced bending, downdrag, stability analyses, and seepage analyses were performed for the Highway 23 Bridge and T-walls. Wick drains with preloading will be used at each of the four locations where T-walls tie-into the conveyance channel levees to mitigate the risk of excessive settlement and differential settlement between pile supported and grade supported features and the effect of SIBM on batter piles. The DT also considered the location of the highway piers with respect to the preload embankments at each levee/T-wall tie-in and ensuring that adequate offset distances were designed so the highway bridge piles are not affected. The highway approach ramps were also analyzed for settlement and stability. Pavement recommendations were also made and included in the geotechnical engineering report.

4.9 Siphon

During the 60% design phase, the inverted Siphon adjacent to Timber Canal was planned to be located at Station 109+00. Construction of the inverted Siphon will precede excavation of the Conveyance Channel levee such that interior drainage is maintained from the north of the MBSD project to the south of the MBSD project by having Timber Canal flow into the Siphon complex, thus allowing Wilkinson Pump Station to drain both polders. Slope stability analyses were performed to determine the position of the siphon such that the location does not compromise the safety of a constructed NOV-NF-W-5a.1 levee (adjacent to the siphon's inlet and outfall features) considering temporary (during construction) conditions and permanent (after construction) conditions. The NOV-NF-W-5a.1 levee was considered in place before the MBSD project began. As detailed in the geotechnical engineering report, analyses were performed to estimate allowable pile load capacities for the siphon intake and outfall structures and for the adjoining T-walls, to compute minimum sheet pile tip elevations to mitigate underseepage, and to analyze global stability of the adjoining T-walls. The CMAR in designing the temporary works for the siphon determined that DMM was the best solution for cofferdam support along the siphon pipe alignment. The DT also used DMM at each of the four T-wall alignments that connect with the siphon to reduce the unbalanced loads on the T-walls and mitigate the effects of settlement/differential settlement/SIBM near the adjoining conveyance channel levees. **Appendix C** contains the geotechnical calculations relevant to the permanent works associated with the siphon features.

4.10 Drainage Structure

A H-pile (HP 14) supported drainage structure is planned within the alignment of the NOV-NF-W-5a.1 levee, to the north of the MBSD north conveyance levee. The drainage structure is a typical USACE design feature with headwalls on the flood and protected sides of the structure with H-pile (HP 14) supported T-walls that adjoin the structure to the north and south to tie-into the NOV-NF-W-5a.1 levee system. Pile capacities, settlement, seepage and global stability analyses were performed on these structures. **Appendix C** contains the geotechnical calculations relevant to the drainage structure and the adjoining T-walls.

4.11 Outfall Transition Feature

The Outfall Transition Feature (OTF) is considered the area on the basin side of the USACE's alignment of the NOV-NF-W-5a.1 levee that transitions the Conveyance Channel at EL -25 to the natural ground within the basin (EL -4). Stability of the OTF guide levees and guide walls shall comply with the USACE design criteria. The design grade was set at 2 feet above the higher, future conveyance flow stage. The design of the Outfall Channel considers two primary functions. The first and primary feature is the slope transition between the Conveyance Channel and the natural ground within the basin to reduce the head loss. The analysis is performed with hydraulic models and includes an iterative process to optimize the transition. The second feature provides scour protection near the NOV Levees and the transition channel that is accomplished through sheet pile walls. Geotechnical designs considered stability of the anchored sheet pile walls that serve as guide walls and of the sheet pile wall installed perpendicular to the conveyance channel alignment that serves as an end wall at the interface between the armored section and the native basin floor. The end wall design considered the substantial scour depths predicted by the numerical H&H modeling. Note that this scour hole is predicted without proper erosion/scour protection. The development of a deep scour hole necessitated the design of countermeasures as part of the Outfall Transition Feature's design to prevent progressive back scour towards the diversion discharge at the existing back levee. The 60% geotechnical engineering report presents the supporting analyses.

4.11.1 Soil Erodibility

A soil erodibility sampling and testing program was developed to support the design of the proposed MBSD diversion outfall features. The program provided qualitative and quantitative information for accessing the conditions in the Barataria Basin area adjacent to the outfall and support the hydraulic modeling of the area's evolution as the diversion operates and the Outfall Transition Feature's geotechnical and civil designs. Numerical H&H modeling analysis of the diversion predicts significant scour downstream of the armored section of the Outfall Transition Feature. This scouring and the subsequent development of relatively deep scour holes have been observed in similar, actual outfall configurations on the lower Mississippi River. Examples include the West Bay diversion (Yuill et al., 2016), Mardi Gras Pass (Lopez et al., 2014) and Southwest Pass Outlets (Ayres, 2015)). The soil erodibility testing program was scoped and organized to inform the designs of the outfall features. See the Geotechnical Engineering Report for further detail of the testing program. Separate from the geotechnical engineering report, an outfall erodibility testing report was prepared by the DT. This report was for Phase 1 and identified some data gaps and need for additional data gathering (termed "Phase 2"). The Phase 2 study (data and interpretive reports) was completed during the 60% design.

4.12 Dredged Material Placement Areas (DMPA)

DMPAs will be required for this project in a couple areas on land as well as in the basin, on either side of the OTF structure. The DMPAs will be contained by earthen containment dikes (ECDs) in some areas or possibly by the existing back levee. In some areas, they will not be contained. The details of these strategies will not be established until the 90% design phase. Geotechnical analyses of settlement and stability of the DMPA-filled regions, of the earthen containment dikes, and of the adjacent, existing back levee were performed. The analyses considered adjacent borrow material to be used as a source for the ECD construction, similar to a typical marsh creation project. Unlike a typical CPRA marsh creation design, the DMPAs do not require a certain timeframe where the DMPA needs to remain in an intertidal zone. Considering this, geotechnical analyses were solely supported by data from soil borings in the basin and advanced testing typical of a marsh creation design project was not warranted (i.e., self-weight consolation testing, settling column testing on dredged sediments).

Impact on existing back levee. The existing back levee is located at approximate Station 140+00 and marks the transition from the land side of the project to Barataria Bay. The DT evaluated the stability of the existing back levee with respect to the back levee canal using the parameters from Soil Reach 8 and existing survey data from the NOV-NF-W-05a.1 report dated September 2016 and lidar data from 2013. The stability analyses incorporated the Soil Reach 8 parameters. Strength gain below the levee was estimated by adjusting the strength of the material below the levee to provide a minimum factor of safety of 1.0 with respect to global stability toward the back levee channel. The only potential impact of the MBSD project on this existing back levee is with the placement of dredged material in the basin. The findings were that the DMPA fill has no impact on the existing back levee when considering levee stability into the existing back levee canal. This is addressed in Section 6.7 of this DDR.

5. STRUCTURAL DESIGN

5.1 General

This section addresses all permanent project structures except the Railroad Bridge, Hwy 23 Bridge, and Ancillary Buildings. The size and dimensions of conveyance structures are established by hydraulic modeling to achieve the project goals of delivering a minimum level of sediment with the Mississippi River flowing at 1.0 mil cfs now and at the end of the 50-Year Design Life. Structures that are part of the riverine or hurricane line of flood protection are constructed to match or exceed USACE Design Grades. The selection of Design Grades is dictated in Section 2 Hydraulic Design, Design Criteria included as Appendix A, and are described by Structure herein. Unless noted otherwise, concrete structures are designed in accordance with the more stringent of the USACE HSDRRS and USACE Engineering Manuals (EMs) and Technical Letters (ETLs). All concrete structures are pile founded; all Foundation designs are described in Section 4, Geotechnical Design. A complete Design Criteria for all project structures is provided in **Appendix A**. Note that the ongoing designs resulting from an internal VE study reduce the invert to EL -25. The design at EL -25 will be included in the 60% submittal. This will affect the U-Frame monoliths, Gate Monolith and transition walls.

5.2 Intake

The intake structures are designed to accommodate the required flows into the diversion. The structure will be designed as a reinforced concrete structure and will consist of a U-shaped open channel with an invert elevation of EL -25, a top-of-wall elevation of EL 20.35 when acting as flood protection and an approximate interior width of 215.5 feet. The base slab will be supported on steel pipe piles that are spaced 25 feet horizontally and 8 feet longitudinally for section A, B, and D, and spaced 12.5 feet horizontally and 7.5 feet longitudinally for section C. It will extend from the gate structure, outward approximately 239 feet, where it will skew and transition into section D, designed and oriented to optimize the flow into the structure. See **Figure 5.2-1** for section plan.

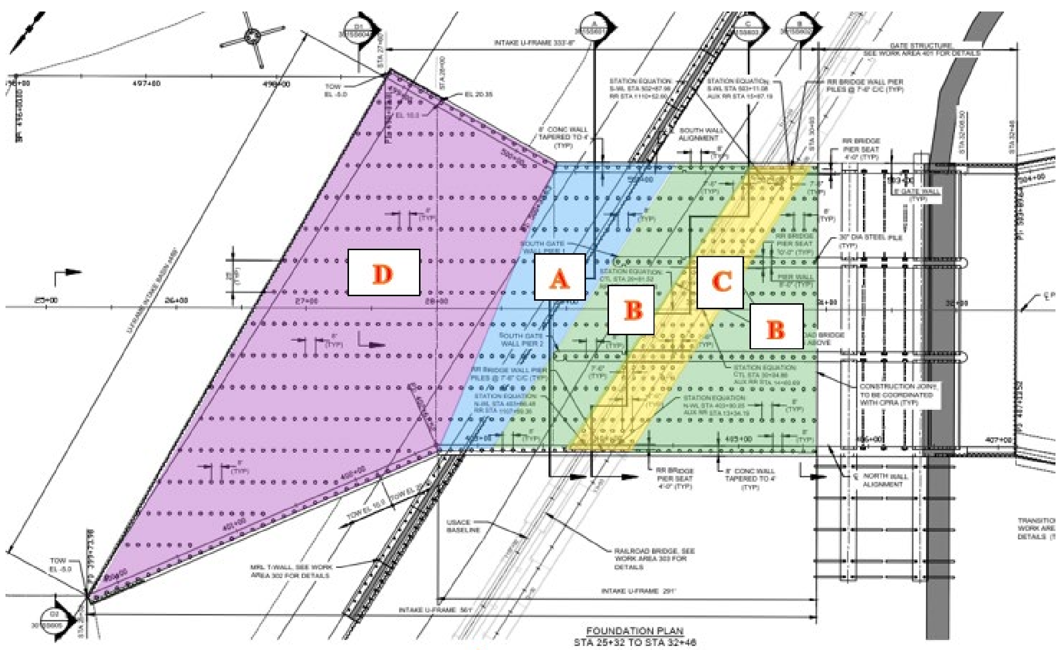


Figure 5.2-13: U-Frame section plans

A plan view of the current design is shown in **Figure 5.2-2**, below, and a general cross section is shown in **Figure 5.2-3.** The selection of this structure was driven primarily by the hydraulic characteristics of this intake. Because the selected intake does not extend significantly into the Mississippi River, it has been determined that the site can be dewatered within a cofferdam, and therefore, in-the-dry construction methods will be utilized. Dewatering bulkhead slots were added at the ends of the RR piers to allow dewatering inspections and repair capability to 60% of the intake length including the RR piers. The opening widths are the same as the gate monolith bays, and the bulkheads are interchangeable. The RR bulkheads are installed using a crane mounted floating plant.

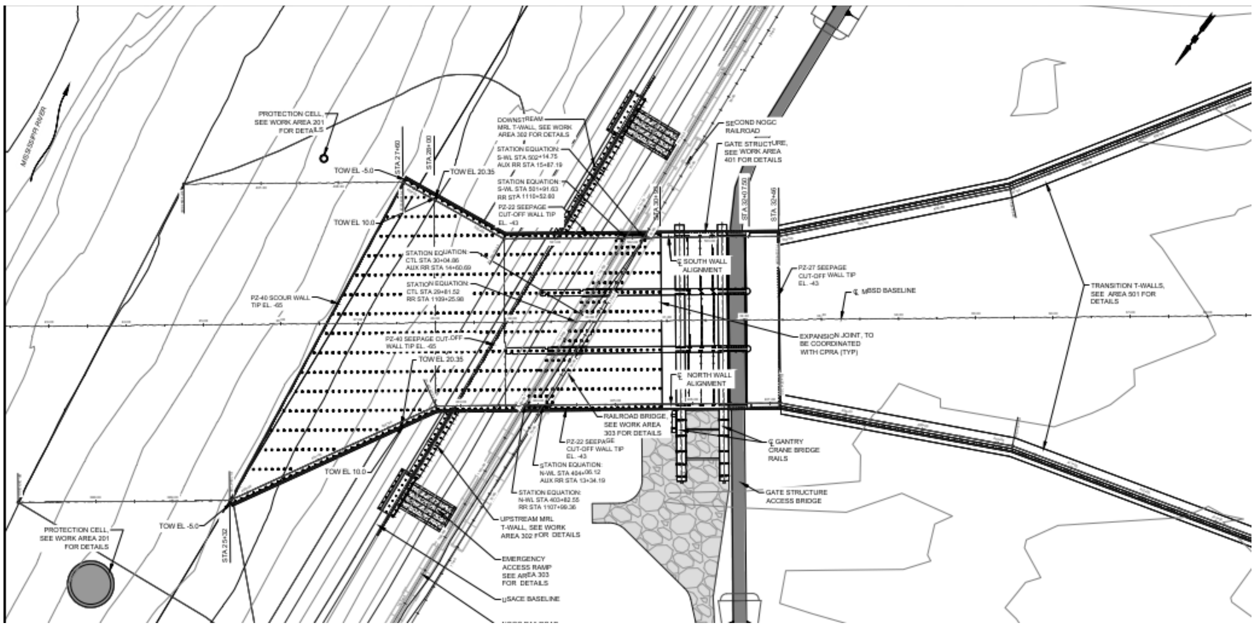


Figure 5.2-2: U-Frame Approach

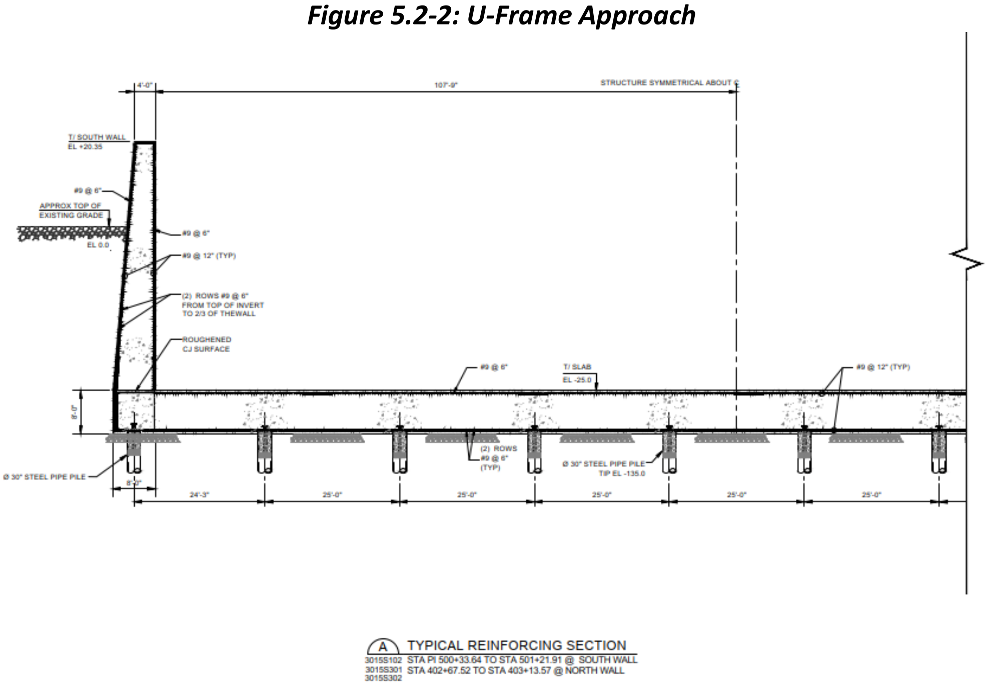


Figure 5.2-3: Typical U-Frame Section

5.2.1 Design Criteria and Loading Conditions

The intake structure is generally designed in accordance with USACE Guidance, mainly relying upon EM 1110-2104, and utilizes the strength design method of ACI 318-14 with AREMA guidance invoked as applicable for the railroad bridge section. With respect to durability, the intake is designed for a 100-Year Service Life.

The intake structure is designed to accommodate the numerous load cases and various water levels that it will be subjected to both during construction, service, and maintenance. For sections that do not have internal piers (section A and D), the thickness and reinforcement of the walls and invert are governed by the construction case plus wind, which has the lateral soil at EL 5.0. For sections B and C which contain two internal piers, the controlling case for walls is maintenance dewatering condition where only one bay is dry, and the water elevation inside of the other two bays is at EL 8.0, while water elevation outside of the structure is at EL 0.0. This provides an imbalanced water pressure on the walls of the structure. For invert design of section B, construction load case plus wind controls. The structure was designed utilizing the strength design method but also checked for serviceability requirements including deflection and crack control.

The governing pile loads result came from the usual load cases for all sections, which they are all under the impervious condition and have high load factors. Those usual load cases cause maximum compression force on the piles. The piles do go into tension under maintenance dewatering load cases, but the total tension loads do not govern the length of the piles – rather, the axial compression force does.

The railroad section was analyzed for the same conditions, but with the addition of the dead load and live loads, including longitudinal traction and braking forces, from the bridge imparted to the interior walls (piers) and exterior walls (abutments). For the railroad bridge section, it was assumed that the train live loading is not imparted during the construction case, but rather only in the service conditions.

5.2.2 Analysis and Design Summaries

For the primary structural analysis of the walls and invert, a two-dimensional nonlinear finite element model was developed using SAP2000. Due to the varying load cases and geometries, four primary models were developed for the intake:

1. Section A. Typical intake section with top of wall at EL 20.35, without middle pier walls.
2. Section B. Typical intake section with top of wall at EL 20.35, with two middle pier walls.
3. Section C. Intake section at railroad bridge with two middle pier walls. interior piers.
4. Section D. Intake flared section at Mississippi River.

The resulting structural design sections are depicted in the project drawings. The connection between piles to invert slab has been investigated for both fixed and pinned connections. Fixed condition analysis was performed for all four sections. Alternatively, pinned condition analysis was performed for section C only, which is the critical section. Initial pile layouts were determined by checking the maximum axial loads under service load combinations with pile capacities. Axial load and bending moment interaction curves were used to check the pile structural capacity. Detail connection design and reinforcement have been performed for both fixed and pinned connections. Pinned connection is shown in the drawings for its optimizing solution for this structure.

The design of the railroad bridge section was similar to the typical sections, except loadings from railroad bridge should be transferred onto the pier walls and contribute to the structural demand of the section.

5.2.2.1 Major Design Elements for Future Design Phases

- Unbalanced slip failure loads from the MRL
- Abrasion-resistant concrete and thermal analysis of pour sequence
- Concrete properties
- Reinforcing steel details
- Durability details
- Seismic Analysis
- Site-specific barge impact load development
- Geotechnical investigations for pile design

5.3 Intake Protection Cell

5.3.1 General Description

The impact protection cell will be designed and constructed at the upriver side of the intake structure to resist any potential barge impacts and environmental loads. Initially, the closed cell will be a part of the CMAR's temporary dewatering cofferdam system but will be enhanced and reinforced during initial construction and will remain after removal of the remainder of the temporary cofferdam to serve as a permanent impact protection cell. These typical cofferdam cells located upriver are set at 61.38 feet in diameter. PS31 steel sheet pile will be used, which has a 19.69-inch nominal width and a tip elevation at EL -105. Although localized damage to the cell could occur from the collision due to barge impact, the cell will be designed and constructed to prevent total collapse and extensive repairs as much as practical. A pile founded 4 feet-6 inches concrete hoop has been designed to resist the impact force of a barge tow as described herein.

5.3.2 Design Criteria and Loading Conditions

Design of the cell shall consider impact from a barge tow consisting of a barge stack and tug. The design barge is considered to be a loaded jumbo open hopper barge with the following parameters as defined in AASHTO GVCB-2-M:

- Length, LB = 195 feet
- Beam, BM = 35 feet
- Loaded Draft, DL = 9 feet
- Deadweight Tonnage, DWT = 1,900 tons

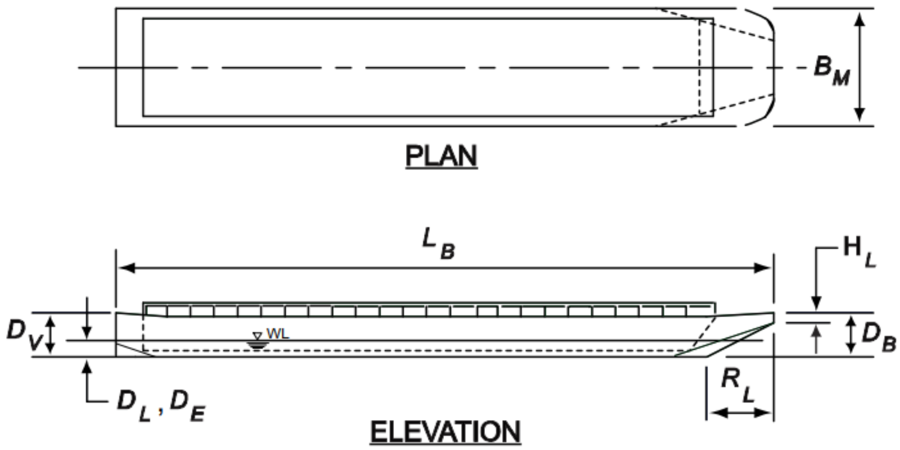


Figure 5.3-1: Design Barge Tow Plan and Elevation View

The design tug is considered to be a loaded jumbo open hopper barge with the following parameters as defined in AASHTO GVCB-2-M:

- Length, L_B = 146 feet
- Beam, B_M = 35 feet
- Loaded Draft, D_L = 9 feet
- Deadweight Tonnage, DWT = 1,305 tons

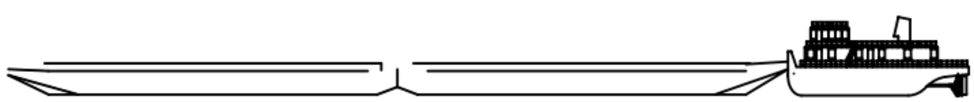


Figure 5.3-2: Typical Barge Configuration Elevation View

In accordance with AASHTO, GVCB-2-M, Section 3.8, the displacement tonnage for barge tows shall equal the displacement of the tug/tow vessel plus the combined displacement of the number of barges in the length of the tow. The number of barges across the width of the tow are neglected in computing the impact energy of the tow (and therefore the impact force as well) since they are assumed to break away upon impact.

The following barge stacks are considered for design:

Table 5.3-1: Design Barge Stacks

Barge Stack	Number of Barges for Energy Calc	Total Mass (tons)
3x5	5	10,805
2x6	6	12,705
5x7	7	14,605

Following the kinetic energy and impact force calculation procedure outlined in AASHTO GVCB-2-M, Section 3, the following values were determined. It is assumed that the velocity of the barge is equal to the river velocity at the noted river flow conditions assuming that the barge is adrift, lost power or traveling near the speed of the river flow upon impact.

Table 5.3-2: Barge Impact Design Forces

# Barges in Tow	Velocity (fps)	Impact Kinetic Energy (kip-ft)	Barge Bow Damage Length (ab) (ft)	Impact Design Force (kips)
5	4.0	6,217	4.57	1,851
6	4.0	7,310	5.23	1,924
7	4.0	8,403	5.87	1,994
5	6.0	13,987	8.79	2,316
6	6.0	16,447	9.94	2,443
7	6.0	18,906	11.03	2,563
5	8.0	24,866	13.47	2,830
6	8.0	29,239	15.11	3,011
7	8.0	33,612	16.64	3,180

Design of the cell shall consider the effect of barge impact loads acting on the face of the cell at the river stages as defined in the table above. The bow/rake of the barge is assumed to be 3 feet above the waterline.

Each load case considers barge impact, wind, and current loads for their respective river conditions given in the design criteria. Load combinations in accordance with ASCE 7-16 and USACE EM 1110-2-2104 are used for design of the concrete superstructure with ACI 318 provisions. AISC provisions and USACE EM 1110-2-2906 are used for the design of steel pipe piles. These load combinations are as described in the following table:

Table 5.3-3: Intake Protection Cell Load Cases

No.	Load Case Name	Description	Factored Load Combination	Load Category	Allowable Overstress Factor
1	Maximum Low River, Barge Impact	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.3*(WA + WS +IM)	Unusual	1.33
2	Low River, Barge Impact	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.3*(WA + WS +IM)	Unusual	1.33
3	High River, Barge Impact	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.3*(WA + WS +IM)	Unusual	1.33
4	Maximum High River, Barge Impact	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.0*(WA + WS +IM)	Extreme	1.75

No.	Load Case Name	Description	Factored Load Combination	Load Category	Allowable Overstress Factor
5	High River w/ Empty Barge Adrift in Hurricane Event	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.3*(WA + WS +IM+ID)	Unusual	1.33
6	High River w/ Empty Barge Adrift in Hurricane Event	- Self-Weight - Current - Wind - Barge Impact	1.2*D + 1.0*(WA + WS +IM+ID)	Extreme	1.75

Notes:
1.The navigation lane is approximately 800 feet from the intake. This distance and the high contour elevation of the MR bank slope at the protection cell greatly reduced the risk of impact by a large vessel; therefore, a ship loading was not considered.
2. The EIS report included a navigation simulation study. The results indicate that vessels will not be pulled into the open diversion structure.

5.3.3 Analysis and Design Summaries

The impact protection cell will consist of a concrete ring wall connected to a concrete base ring slab at the bottom and will be supported by 20 steel pipe piles. The purpose of the reinforced concrete vertical ring is to provide stiffness to the closed cell and minimize damage in the event of barge impact. A closed cell protection dolphin with sheet pile and backfill alone would be more prone to localized damaged (i.e. crushing, deflection, displacement, breaking of sheet piles interlocks, etc.) and would be costly and difficult to repair. The concrete base ring slab will be provided around the bottom of the vertical ring wall at the mudline to engage the piles and provide adequate edge distance and offset from the face of the sheet pile wall. Also, a 1.5 foot thick concrete top slab will be rigidly doweled to the top of the ring wall around the whole cell, supported by 6 supplemental interior piles. The concrete ring will be 4.5 feet thick, extending from the top of the cell at EL 16.65 down to the top of the base ring slab at EL -10 for a total height of 26.65 feet. The concrete base ring slab will be 7 feet wide x 5 feet thick, extending from the top of the base ring at EL -10 down to the mudline at EL -15. **Figure 5.3-3** below shows the intake protection cell plan and elevation view.

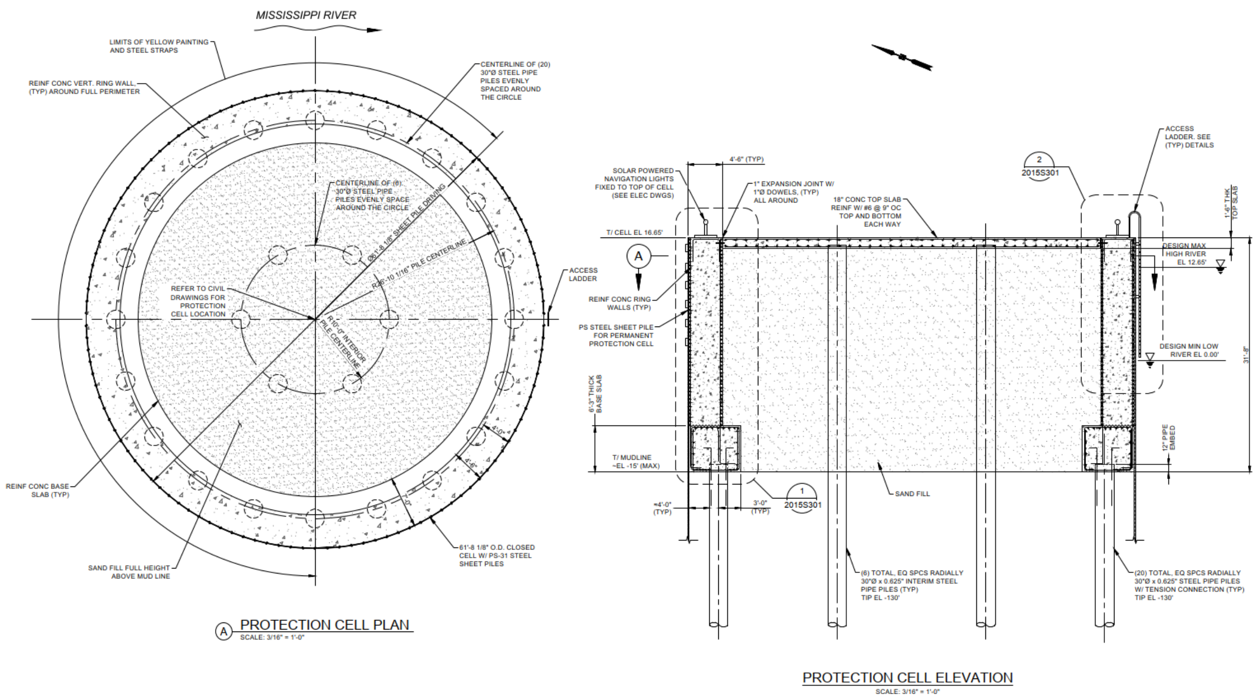


Figure 5.3-3: Intake Protection Cell Plan and Elevation View

CPGA analysis was performed for the impact protection cell to evaluate the load effect on the piles for all six load cases shown in Table 3.A. The protection cell design requires a total of 26 piles – 20 exterior piles under the concrete base ring slab and 6 interior piles supporting the concrete top slab. Piles located along the leading face of the exposed perimeter of the cell will be considered “strong” piles (11 exterior and 3 interior), while piles on the opposing side are considered “weak” piles (9 exterior and 3 interior). Piles which are facing the impact load and with compacted backfill of the closed cell in their shadow were considered “strong” as the resistance of the closed cell backfill material will provide additional resistance; whereas, the piles on the opposite side of the impact load were considered “weak” as the resistance to lateral load is provided only by the soft soils outside of the closed cell at the mudline base of the cell. The “strong” and “weak” side piles were analyzed in LPILE by the geotechnical team to provide lateral spring values which were input in the model to capture lateral resistance capacity of the piles. Similarly, the vertical axial capacity and support of the piles was input into the model using linear springs in accordance with CPGA methods for modification of axial stiffness (k = AE/L) by a coefficient c33, which accounts for the restraining effect of the surrounding soils. A c33 value of 1.70 was utilized to represent a combination of skin friction and end bearing support of the piles. Additionally, CPGA analysis considers factors of safety from USACE EM 1110-2-2906 for an assumed dynamic pile load test. All piles are 30 inch x 0.625 inch open end steel vertical pipe piles with a tip elevation at EL -130. Exterior piles are embedded 12 inches into the concrete base ring slab and interior piles are embedded 6 inches into the concrete top slab for full pile head fixity condition. The pile layout can be seen in **Figure 5.3-4** below, showing which piles were considered “strong” versus “weak” side piles.

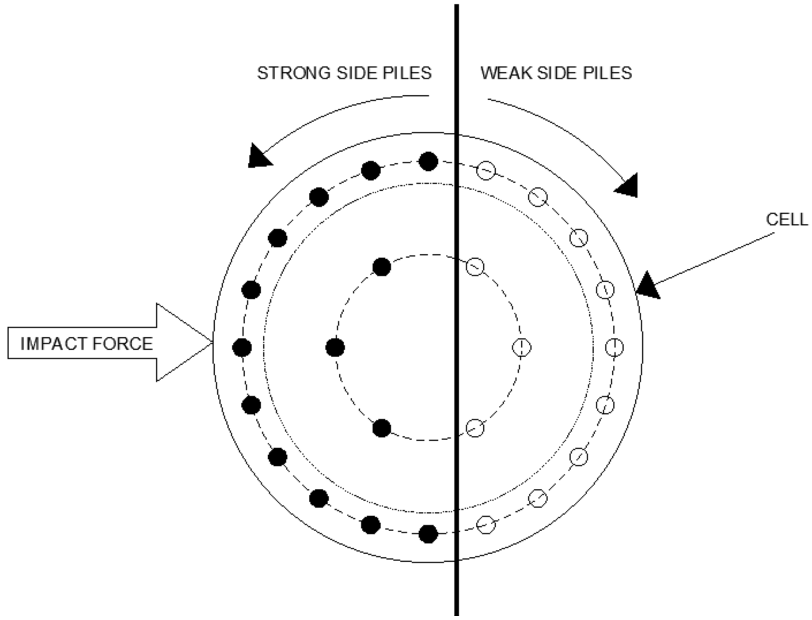


Figure 5.3-4: Intake Protection Cell Pile Layout

A three-dimensional finite element SAP2000 model for the concrete ring and base slab was created to apply all applicable loads for each load case, as shown in **Table 5.3-1**. The full barge impact load applied in SAP was assumed to occur at a single point but distributed on a 45-degree angle in each direction along the face of the cell. Wind load is applied on the exposed area of the cell above the waterline and current load on the exposed area of the cell below the waterline for each load combination and respective river condition. Piles are modeled as springs connected to the concrete base slab frame and concrete top slab shell area element with the appropriate spring stiffness from CPGA considering “strong” and “weak” side P-y curve data. Reinforcement in the concrete ring and base slab is designed for the maximum factored loads from SAP2000 results. The full structural calculation package for the design of the intake protection cell is found in **Appendix D.3**. The SAP2000 model created for the protection cell can be seen in **Figure 5.3-5** below, which shows the results for worst-case bending due to load combination 4 with the maximum barge impact force.

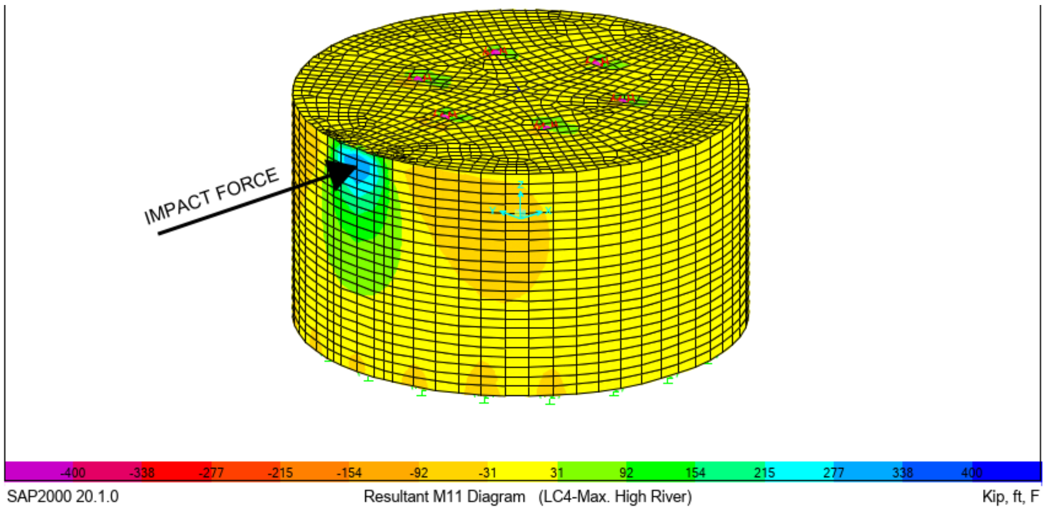


Figure 5.3-5: SAP2000 Model for Worst-Case Bending

5.4 Gate Structure

5.4.1 General

The gated structure is part of the Headworks, located between the Intake U-Frame and the Discharge Transition T-Wall. The structure houses three slots for bulkhead gates; gates are sized to meet the project goal of delivering 75,000 cfs conveyance with the River flowing at 1.0 mil cfs. The overall opening size was increased to meet future conditions that are negatively affected by sea level rise and a higher tailwater created by land building; the selection process is described in the previously submitted BODR.

The change from tainter gates to bulkhead stacks was made between the 30% and 60% Submittals. The Gate Structure has been modified substantially during this phase to account for the change in gate type as well as the reduction from four to three gate bays.

The Gate Monolith structure consists of three gate bays. It also includes slots for dewatering/emergency bulkhead placement, a riverside access bridge, and gantry crane rails for lifting and placing bulkheads; all component designs are described separately below. The Gate Monolith is pile founded. Walls riverward of gates are constructed to EL 20.35, based on the design 50-Year future Hurricane event. The discharge side walls are constructed to EL 15.85 to match the Basin Side 50-Year future Hurricane event.

5.4.2 Gate Monolith Design

The Gate Monolith is a 3-bay concrete structure that provides support for bulkhead gates, machinery facilities, and an access bridge. The structure consists of a 231.5 feet wide x 153 feet long x 8 feet thick base slab supported by 240 open steel 30 inch x 9/16 inch pipe piles, and four 8-feet thick walls. Top of the slab (TOS) elevation is at EL -25.0, top of the wall (TOW) elevation on the river side is at EL 20.35 and on the basin side is at EL 15.85.

The operating gate slots include vertical steel columns that extend approximately 12 feet – 10 inches above the center monolith piers. These columns serve as guides to resist wind on the bulkhead stack when the bulkheads are in the stored position.

a. Design Criteria and Load Conditions

EM 1110-2-2104 (2016) and ACI 318 (2014) are used to define the load cases for reinforced concrete design. For the analysis of the structure, 21 load cases are considered and are as follows:

Table 5.4-1: Gate Monolith Design Load Cases

No.	Load Case Name	Description	Factored Load Combinations	Load Category
1	Construction + Backfill + Downdrag (no uplift)	- Dead (str. + bridge wt. + Crane Rails) - Crane Loaded (Moving load) - Live Load on Access Bridge (Moving load) - Lateral soil up to EL 5.0 on both sides - Temp. construction surcharge of 200 psf on slab and backfill - Downdrag	1.6 (D+EH+L_s + LL)	Unusual
2	Construction + Downdrag +Backfill (no uplift) plus Wind	- Dead (str. + bridge wt. + Crane Rails) - Lateral soil up to EL -18 on one side (15 feet diff. backfill) - Temp. construction surcharge of 200 psf on slab and backfill - ASCE Wind - Downdrag	1.6 (D+EH+EV_d+L_s+L +W)	Unusual
3	Water @ Design SWL No Wind (R/S Imper.), + Downdrag (See Note 5)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0 - Gates closed (Gate wt.) - Downdrag - R/S impervious cutoff (uplift, water @ EL -1.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0 on exterior walls	2.2 (D+EH+ EV_d+Hs+Hu+I)	Usual
4	Water @ Design SWL/Flowline, No Wind (R/S Per.) + Downdrag (See Note 6)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0 - Gates closed (Gate wt.) - Crane Girder DL - Downdrag - R/S pervious cutoff, var. linear uplift head (Riverine water level @ 8.0 and Basin @ -1.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	2.2 (D+EH+ EV_d+Hs+Hu+I)	Usual
5	Water @ Design SWL + Wind + Wave (R/S Imper.) + Debris on Gates + Downdrag (See Note 5)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0 - Gates closed (Gate wt.) - Downdrag - ASCE wind pressure on gate above 14.85 + bridge - Wave load on gates (50 yr future) - 500 plf debris load on gate @ SWL - R/S impervious cutoff, uniform upliftwith water @ EL -1.0 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu+Hw+W+I)	Unusual

No.	Load Case Name	Description	Factored Load Combinations	Load Category
6	Water @ Design SWL + Wind + Wave (R/S Per.) + Debris on Gates + Downdrag (See Note 6)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0 - Gates closed (Gate wt.) - Downdrag - ASCE wind pressure on gate above 14.85 - Wave load on gates (50 yr future) - 500 plf debris load on gate @ SWL - R/S pervious cutoff, var. linear uplift head (Riverine water level @ 8.0 and Basin @ -1.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu+Hw+W+I)	Unusual
7-1-	Water to TOW @ EL 20.35 (R/S Imper.) + Debris on Gates + Siltation + Downdrag **Resiliency Check – Maximum Differential Head**	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 20.35 - B/S tailwater at EL -3.0 - Gates closed (Gate wt.) - Downdrag - 500 plf debris load on gate @ SWL - 10 feet heigh sediment load all over the slab - R/S impervious cutoff, uniform upliftwith water @ EL -1.0 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.2 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{\text{Gate wt.}}$	Extreme
7-2-	Water to TOW @ EL 20.35 (R/S Per.) +Debris on Gates + Downdrag **Resiliency Check Maximum Differential Head**	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 20.35 - B/S tailwater at EL -3.0 - Gates closed (Gate wt.) - Downdrag - 500 plf debris load on gate @ SWL - R/S pervious cutoff, var. linear uplift head with R/S head @ EL 1.5 and B/S @ EL -1.0 (Riverine water level @ 8.0 and Basin @ -1.0) - Lateral hydrostatic pressure from water table at EL 0.0	0.9 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{\text{Gate wt.}}$	Extreme
8	Not for 60%, Resiliency load case that includes the 100 yr Hurricane SWL			
9-	Reverse Head, Basin Side Hurricane @ EL 9.35 + Downdrag (B/S Imper.) (See Note 7)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL -1.0 - B/S tailwater at EL 9.35 - Gates closed (Gate wt.) - Downdrag - Impervious cutoff, uniform uplift with water @ EL -1.0 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	2.2 (D+EH+ EV_d+Hs+Hu)	Usual
10-	Reverse Head, Basin Side Hurricane @ EL 9.35+ Wave + Wind + Downdrag (B/S Imper.) (See Note 7)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL -1.0 - B/S tailwater at EL 9.35 - Gates closed (Gate wt.) - ASCE 7 wind pressure on gate above 9.35 - Wave load on gates (B/S) from Basin Side Hurricane (50 yr Future) - Downdrag - Impervious cutoff, uniform uplift with water @ EL -1.0 - Lateral soil up to EL 5.0	1.6 (D+EH+Hs+Hu+Hw+W)	Unusual

No.	Load Case Name	Description	Factored Load Combinations	Load Category
		- Lateral hydrostatic pressure from water table at EL 0.0		
11-	Reverse Head to TOW @ Basin Side (EL 15.85) + Debris on Gates (B/S) + Downdrag **Resiliency Check, Maximum Reverse Differential Head** (B/S Imper.) (See Note 7)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL -1.0 - B/S tailwater at EL 15.85 - Gates closed (Gate wt.) - Downdrag - 500 plf debris load on gate @ SWL (B/S) - Impervious cutoff, uniform uplift with water @ EL -1.0 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.2 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{Gate\ wt.}$	Extreme
12-	River @ 1,000,000 cfs, 75,000 cfs Conveyance Operation (Start of Operation) + Downdrag (R/S Imper.) (See Note 8)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 6.9 - B/S tailwater at EL 6.9 - Gates Open - Crane Loaded (Moving Load) - Downdrag - Impervious cutoff, uniform uplift with water @ EL -1.0 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	2.2 (D+EH+Hs+Hu+Ll)	Usual
13-	River @ 1,000,000 cfs, 75,000 cfs Conveyance Operation (Long Term Operation) + Downdrag (R/S Per.) (See Note 9)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 6.9 - B/S tailwater at EL 6.9 - Gates Open - Crane Loaded (Moving Load) - Downdrag - Pervious cutoff, uniform uplift with water @ EL 6.9 - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	2.2 (D+EH+Hs+Hu+Ll)	Usual
14-	River @ 1,250,000 cfs, Operation (Start of Operation) + Downdrag (R/S Imper). (See Note 8)	- Dead (str. + bridge wt. + Crane Rails) - Live (Gate Wt.) - R/S SWL @ EL 9.0 - B/S tailwater at EL 9.0 - Gates open - Downdrag - Crane Loaded (Moving Load) - Impervious cutoff (uplift, water @ EL -1.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu+W +Ll)	Unusual
15-	River @ 1,250,000 cfs, Non-Operation + Downdrag + Crane + Gates Closed + Debris (Imper.)	- Dead (str. + bridge wt. + Crane Rails) - Live (Gate Wt.) - R/S SWL @ EL 12.65 - B/S tailwater at EL -1.0 - Gates closed (Gate wt.) - Downdrag - Crane Loaded (Moving Load) - 500 plf debris load on gate @ SWL (B/S) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu+W +Ll)	Unusual

No.	Load Case Name	Description	Factored Load Combinations	Load Category
16-	Maintenance Dewatering, One Bay Dry, One Side Bay using Stoplogs Adjucent to the Gate (Per.) (See Note 10)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 8.0 - B/S tailwater at EL 8.0 - Stoplogs wt. - Pervious cutoff, uniform uplift with water @ EL 8.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu)	Unusual
17-	Maintenance Dewatering, One Bay Dry, Center Bay using Stoplogs @ RR U-Frame and Adjucent to the Gate Str. @ B/S (Per.) (See Note 10)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 8.0 - B/S tailwater at EL 8.0 - Pervious cutoff, uniform uplift with water @ EL 8.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.6 (D+EH+Hs+Hu)	Unusual
18-	Maintenance Dewatering All Bays using Cofferdam closure at each end of headworks. (Per. See Note 11)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 8.0 - B/S tailwater at EL 3.0 - Pervious cutoff, var. linear uplift (Riverine water level @ 8.0 and Basin @ 3.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	0.9 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{Gate\ wt.}$	Extreme
19-	Emergency Dewatering w/river at Flowline, One bay Dry, Center Bay using Stoplogs Adjacent to the Gate Str. @ B/S (Per.) (See Note 10)	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 12.65 - B/S tailwater at EL -1.0 - Pervious cutoff, uniform uplift with water @ EL 8.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	0.9 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{Gate\ wt.}$	Extreme
20-	Emergency Closure One Side Bay using Stoplogs, Adjacent to the Gate , 2 other gates closed	- Dead (str. + bridge wt. + Crane Rails) - R/S SWL @ EL 16.65 - B/S tailwater at EL -1.0 - Crane loaded (Moving load) - Stoplogs wt. - Pervious cutoff, var. linear uplift head with (Riverine water level @ EL 8.0 and Basin @ -1.0) - Lateral soil up to EL 5.0 - Lateral hydrostatic pressure from water table at EL 0.0	1.2 (D+EH)+1.3 (Hs+Hu)+I + 1.0 $L_{Gate\ wt}$	Extreme
21-	Not for 60%, EQ (OBE) @ Normal Operation (Per.)	-		
22-	Not for 60%, EQ (MDE) @ Normal Operation (Per.)	-		
23-	Maintenance Dewatering (Check Internal Walls)	- Hydrostatic pressure on the wall up to EL 8.0 on one side w/ no water on the other side	1.6 Hs	Unusual

Notes.

1. Vessel impact on the gate monolith is not considered. The Debris impact is 500 Lbs/LF at the surface of the water. The intake is 90 degrees to traffic flow. Vessels would need to pass between the 66-foot wide opening at the RR piers. This is considered highly unlikely. Hurricane driven barges are assumed to be blocked by the RR piers. The RR piers and intake walls riverward of the piers are designed for a hurricane driven barge impact force.
2. Gate piers are in line and continuous with the Intake wall piers and are protected from any impact force.
3. Pile Load tests shall be performed. The factor of safety for piles by Load Category is:

Usual – 2.0
Unusual – 1.5
Extreme – 1.2

3. No uplift applied for construction cases. Assume dewatering system in effect. No downdrag force applied to side walls. Assume gantry crane not operated w/wind.
4. Crane load not included in high wind hurricane events.
5. Direct hurricane loading is short term, for uplift use low water stage at EL -1.0 for uplift force. Assume R/S cut of is impervious. Short term hurricane stages not considered in uplift pressures.
6. Direct hurricane loading is short term. Short term hurricane stages not considered in uplift pressures. Assume R/S cutoff is pervious. Use long term high river stage EL 8.0 similar to maintenance dewatering and maximum gate operation. Uplift is assumed to be uniformly varying from river to transition.
7. Reverse head impervious case. B/S cutoff is impervious, uplift equals river low water at EL -1.0. Short term hurricane stages not considered in uplift pressures.
8. Operating conveyance at 1.0 mil. cfs. Start of operation. Impervious R/S cutoff, uplift equals basin side low water at EL -1.0.
9. Operating conveyance at 1.0 mil. cfs. Pervious R/S cutoff, uplift equals the conveyance stage at EL 6.9.
10. Maintenance Dewatering. The uplift condition conservatively uses the high river stage equal to EL 8.0 resulting in the greatest uplift on the dewatered bays.
11. LC 18 is an extreme event that assumes cofferdams have been installed at each end of the headworks as part of a major dewatering contract. EL 8.0 uniformly varying uplift to end of headworks uplift at EL 3.0.
12. Gate Bays not dewatered for emergency closure load cases.

Backfill is assumed to be sand with dry unit weight of 125 pcf and at rest earth pressure coefficient K_0 of 0.5. The construction case includes a height differential in the backfill of 15 feet and a down drag force is applied on the side walls because of the backfill soil. ASCE 7-16 was used to calculate wind pressure at different elevations that was applied on walls and gates in open position. Applied gate weights, crane loads and bridge loads are based on their respective designs.

The vertical gate guides in the center bay's operating slots were designed for Usual, Unusual and Extreme wind speeds. The Usual wind speed corresponds to the maximum wind speed in which the crane will be operated, 44 mph. The Unusual wind speed corresponds to the 25-Year storm event as shown in ASCE 7-16. The Extreme wind speed corresponds to the maximum Risk Category III-IV wind speed as shown in ASCE 7-16.

b. Analysis and Design Summaries

A SAP2000 finite element model is developed to analyze stresses, displacements and pile reactions (**Figure 5-4-1**). Walls and slab are modeled using area shell elements with corresponding thicknesses and properties. Bridge and crane rails are modeled as frames. The model consists of 17,610 joints and 17,050 area elements with an averaged 2 feet x 2 feet structured mesh. Hydrostatic and wave hydrodynamic loads are applied to the walls and gates. Water pressure over the slab and uplift are applied to the slab. Soil lateral forces and down drag are applied to the exterior walls. Access bridge, crane rail and bulkhead loads are transferred to the walls. See **Figure 5.4-2** for an example of applying hydrostatic loads on walls, slab and gates.

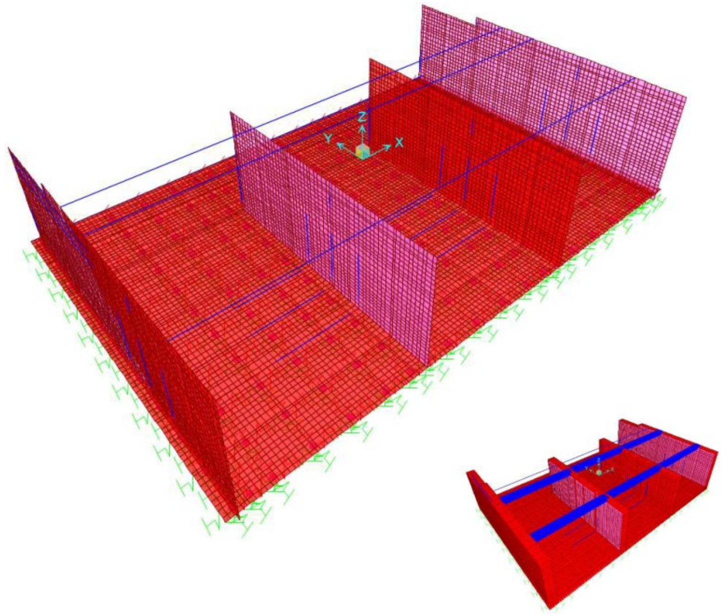


Figure 5.4-1: SAP2000 FEM

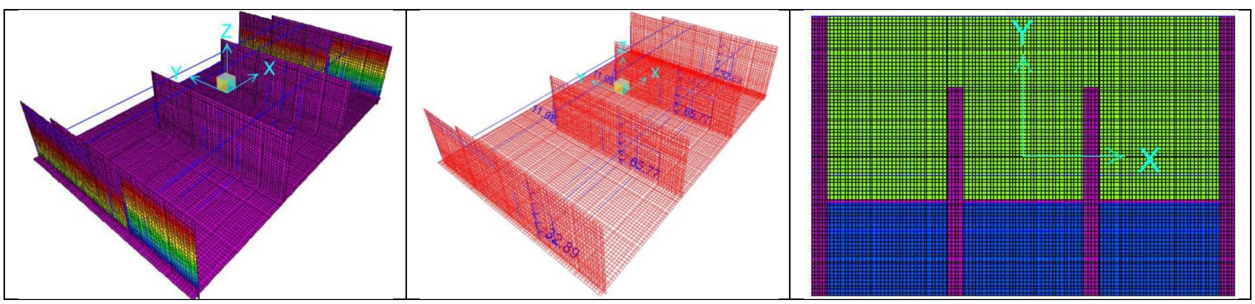
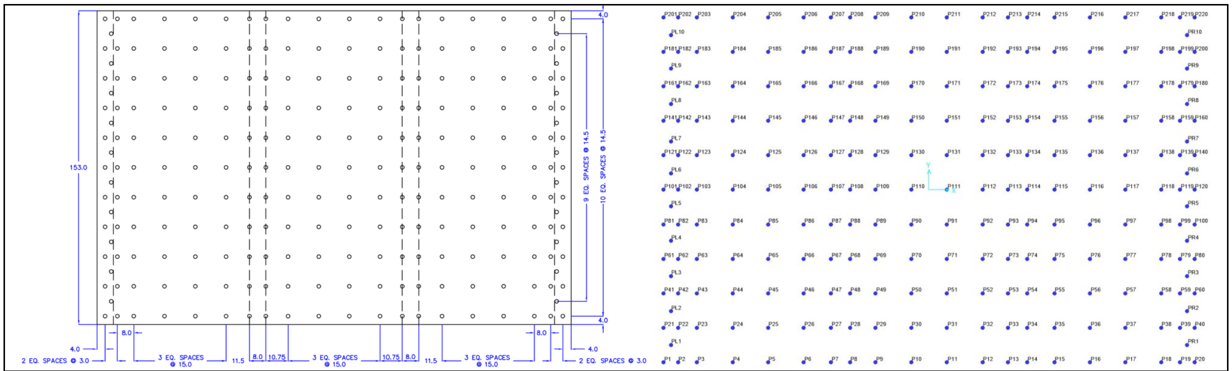


Figure 5.4-2: An example of applying hydrostatic pressure on walls, gates and slab

To model the piles in SAP2000, both fixed and pinned pile head connections were investigated. Piles were modeled as springs with the appropriate stiffness matrixes that represent pile property, soil characteristic and the fixity condition. P-Y curves provided by geotechnical experts for fixed and pinned head piles were used to find the soil subgrade modulus using USACE-CPGA method. The subgrade modulus for a single pile was updated according to the Hurricane & Storm Damage Risk Reduction System Design Guidelines (HSDRRS) pile group reduction factors and then the pile stiffness matrix was generated and plugged into the SAP2000 model. The pile layout and the stiffness matrix that have been used for the fixed head condition are shown in the **Figure 5.4-3** as an example.



PILE STIFFNESSES AS CALCULATED FROM PROPERTIES					
0.76936E+02	0.00000E+00	0.00000E+00	0.00000E+00	0.78504E+04	0.00000E+00
0.00000E+00	0.76936E+02	0.00000E+00	-0.78504E+04	0.00000E+00	0.00000E+00
0.00000E+00	0.00000E+00	0.21808E+04	0.00000E+00	0.00000E+00	0.00000E+00
0.00000E+00	-0.78504E+04	0.00000E+00	0.16021E+07	0.00000E+00	0.00000E+00
0.78504E+04	0.00000E+00	0.00000E+00	0.00000E+00	0.16021E+07	0.00000E+00
0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00

Figure 5.4-3: Pile layout, springs that are used in the SAP2000 model and the pile stiffness matrix generated in CPGA as an input for the spring stiffness

As part of the analysis, the compression, tension and displacement of the piles have been evaluated considering the load cases allowable over stresses and checked against the allowable soil and structural pile capacities and displacements to confirm that the pile type and layout have been satisfactory. The structural capacity check of the pile layout was based on the combined bending moment formula of the CPGA user manual. The bending moment check has been conducted by superimposing the service load induced moments and unbalanced load induced moments. Service load induced moments are calculated based on the regression analysis of lateral loads at the top of the piles and induced moments, or P-M curve data. So, P-M curves provided by geotechnical group for the 30 inch piles, have been used to develop a relation between lateral forces at the pile head and the moments. **Figure 5.4-4** shows pile properties and an example of a P-M curve.

Fixity Condition	Pile Properties						Soil Data	Pile Group Factors	
	Fy (ksi)	A (in^2)	Ix (in^4)	Iy (in^4)	Sx (in^3)	Sy (in^3)	Es w/out Group Reaction Factor (ksi)	Group Vertical Reduction Factor	Group Lateral Reduction Factor
Pinned	50	52.02	5636.9	5636.9	375.8	375.8	0.437	1	0.807
Fixed	50	52.02	5636.9	5636.9	375.8	375.8	0.465	1	0.807

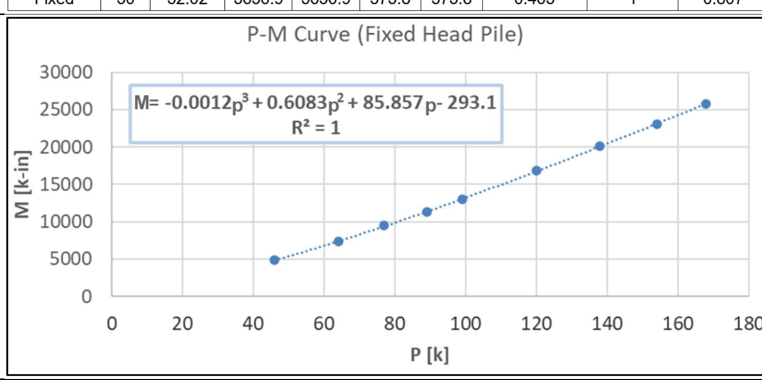


Figure 5.4-4: Pile properties and the P-M curve for the fixed head connection

Pile structural capacity for both fixed head and pinned head connections were analyzed considering the service loads and the unbalanced loads. FLAC analysis that have been conducted by the geotechnical group show that the unbalanced moments are reducing by increasing the distance from the sides of the slab and in the middle of the slab, the moments are almost zero. This is the main reason of having an additional staggered 2nd rows of piles on each side of the slab, in addition to the point that pile spacing increasing by moving toward the center of the slab, except under the walls that closer piles are needed because of the compressive loads of the walls.

In order to adding up the unbalanced induced moments and service load induced moments, the direction of the moments should also be considered. For instance, while for the fixed head condition, the maximum unbalanced moments occur at the top of the piles, but the direction of the moments are in the opposite direction of the service load induced moments. Therefore, they cancel each other's effect. For a conservative design, we added the maximum unbalanced moment with the service induced moments that were in the same direction, even if they were not necessarily occurred in the same location of the pile. Then the combined moments were calculated and checked versus the pile structural capacity considering the allowable overstresses. The same procedure has been done for the pinned head connection.

The other analysis is focused on the shear and moments in the walls and slab and consequently designing the sections and reinforcements accordingly. **Figure 5.4-5** shows an example of the moment output, which is the positive moments envelope in the slab. For more efficient and economic design, the slab is divided into sub regions, so that the reinforcement of the slab will be defined based on the local regions' moments considering the maximum and minimum allowable reinforcements permitted by EM 1110-2-2104. The shear capacity of the section and the need for shear reinforcement is also evaluated based on the EM 1110-2-2104 criterion.

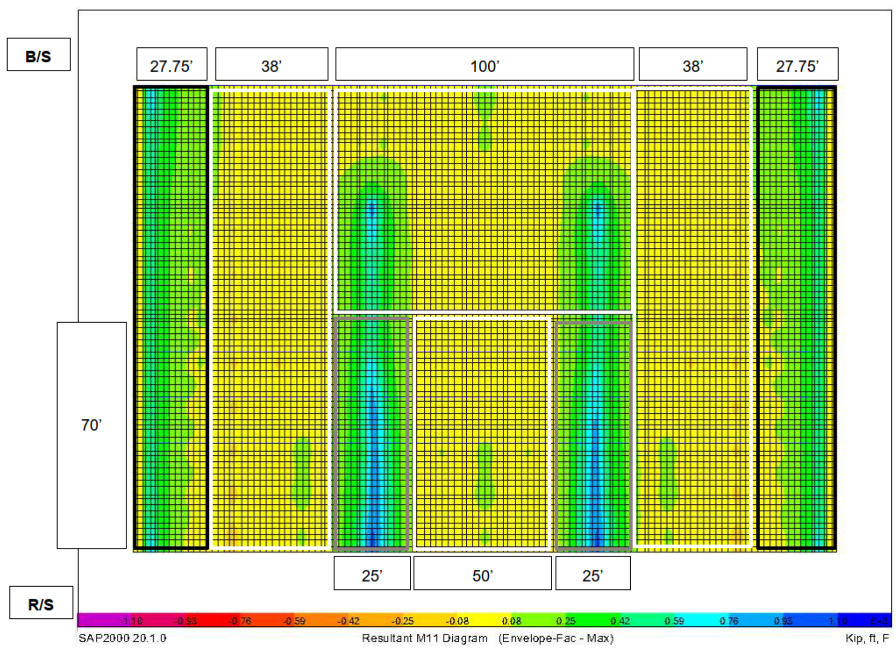


Figure 5.4-5: Positive moment envelope in the direction perpendicular to the flow

A separate SAP2000 model is developed for the steel bulkhead gate guides. A diagonal brace is used to support the vertical guide. Calculations for the gate guides can be found in **Appendix D.4.1**.

5.4.3 Bulkhead Gates Design

Stacks of steel bulkhead gates are used to control flow into the Diversion Channel and also perform maintenance and emergency dewatering operations. Each 66 foot - 6 inch wide gate bay is fully closed with a stack of four bulkhead gates. The supporting sill is at EL -25.0 and the top of stack elevation is EL 20.35. Each bulkhead section is approximately 11 feet - 4 inches tall x 66 feet - 6 inches wide x 7 feet - 11 inches deep and is primarily comprised of a skin plate with stiffener ribs on the riverside face of the unit, three horizontal trusses, secondary bracing, and two roller assemblies on each side of the gate.

Trusses are comprised of large W-shape chords with smaller W-shape web braces. Single angle shapes are used for diagonal/cross bracing perpendicular to the skin plate and between downstream truss girders. Vertical 5/8 inch thick by 6 inch wide intercostal plates provide additional stiffness to the skin plate; intercostals are spaced at approximately 3 feet - 8 inches.

a. Single Bulkhead Design Considerations

All bulkheads supplied for the MBSD Headworks will be of the same size and construction; they can be used in any position within the gate stack and will also be used for dewatering activities. The decision to use one type for all panels, as opposed to a larger and smaller unit, was deliberated by the structural, mechanical, operations and CMAR teams during this design phase. The DT's original intent was to provide two bulkhead types: a "dewatering section" for the worst case hydraulic loads and another for gates with less load demand (used higher in the dewatering stack and/or for operations only). Preliminary member sizes of all main components were developed for the two types and a weight comparison was performed. The difference in required steel is approximately 7.4 tons, or 16% of the total structure weight.

The eliminated risk of improper bulkhead placement is a significant benefit to the one-panel option. The result of misplacing a typical closure bulkhead section at the bottom of a dewatering stack could be catastrophic. The DT considered including descriptions of the two bulkhead types in the Operations Manual and differentiating panels with unique paint or other markings, but it was decided that this knowledge could still be lost decades from now after new coats of paint and overturning of operations staff. Lifetime structure operations will be streamlined with the ability to use any panel, located in any stored area, for any purpose; fewer movements of the gantry crane are required for both emergency and routine operations. Also, if a bulkhead becomes damaged a replacement is more readily available.

Effects on the crane design were also considered: if two types of bulkheads were to be employed, the crane must be able to lift a total of two dewatering and two typical units; if one type is employed all four units are the heavier variety. It was determined that the increased weight of the stack was not substantial in terms of crane capacity or operability.

Increasing steel strength from 50 ksi to 65 ksi was also examined during this effort in an attempt to decrease the total weight of the 4-bulkhead stack. The change in steel strength reduces the overall required bulkhead unit weight approximately 8%, however the CMAR and steel suppliers suggest that 65 ksi steel might be cost prohibitive and/or not readily available. The bulkhead designs will therefore continue to use Grade 50 steel.

b. Other Operational Considerations

Bulkheads will either be stored standing vertically on two cradles on top of the gated structure or on the adjacent storage platform. A gantry crane will be used to move the panels between these storage locations and their operational positions. The gantry will lift the panels lifting points comprised of four pad eyes on the top of each panel; the pad eyes have been located on web members of the top truss such that a balanced lift is achieved.

The bulkhead panels are designed to be installed with the skin plate towards the river regardless of their placement within the concrete structure. This aids operations because gates do not require flipping in orientation by the gantry crane or a separate lifting system when being placed in the downstream dewatering slots; this maneuver would be cumbersome and time-consuming. In terms of structural design, this requires that the gate be designed to retain hydraulic differential in both lateral directions.

c. Design Criteria and Loading Conditions

Structural analysis uses the LRFD design procedure described in ETL 1110-2-584, *Design of Hydraulic Steel Structures*, including Appendix G – Bulkheads and Stoplogs and Appendix E – Vertical Lift Gates. EM 1110-2-2701, *Vertical Lift Gates*, and the superseded EM 1110-2-2105, *Design of Hydraulic Steel Structures* (1993), are also used as a reference where ETL design information is limited.

All steel, including the skin plate, is assumed to be 50 ksi.

The load cases examined for the 60% design are correlated to the load cases applied to the gate monolith structure and guided by the requirements of ETL-1110-2-584. The design cases are as follows:

Table 5.4-2: Bulkhead Gate Design Load Cases

No.	Load Case Name	Description	Factored Load Combination	Load Category
1[3]	Water @ Design SWL	- Dead (self wt.) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0	1.2(D) + 1.6(Hs)	Strength I Usual
2	Water @ Design SWL + Wind + Wave + Debris on Gates + Siltation	- Dead (self wt.) - R/S SWL @ EL 14.85 - B/S tailwater at EL -1.0 - ASCE 7 -16 wind pressure above top of wave - Wave load on gates - Debris Load at waterline - Lateral siltation load	1.2(D+W+IM) + 1.4(Hs) + 1.6(Hd+G)	Strength I Unusual
3[4]	Water to TOW @ EL 20.35 + Debris on Gates + Siltation **Resiliency Check**	- Dead (self wt.) - R/S SWL @ EL 20.35 - B/S tailwater at EL -3.0 - Debris Load at waterline - Lateral siltation load	1.2(D+Hs+G+IM)	Extreme I Extreme
4	Water at 100 yr SWL + Debris on Gates + Siltation **Resiliency Check**	- Dead (self wt.) - R/S at 100 YR SWL @ EL 17.85 - B/S tailwater at EL -1.0 - Debris Load at waterline - Lateral siltation load	1.2(D+Hs+G+IM)	Extreme I Extreme

No.	Load Case Name	Description	Factored Load Combination	Load Category
5	Reverse Head, SWL Basin Side Hurricane @ EL 9.35 + Siltation	- Dead (self wt.) - R/S SWL @ EL -1.0 - B/S tailwater at EL 9.35 - Lateral siltation load	1.2(D) + 1.6(Hs+G)	Strength I Usual
6	Reverse Head, Basin Side Hurricane @ EL 9.35 + Wave + Wind	- Dead (self wt.) - R/S SWL @ EL -1.0 - B/S tailwater at EL 9.35 - ASCE 7wind pressure above top of wave - Wave load on gates (B/S) from Basin Side Hurricane	1.2(D+W) +1.4(Hs) + 1.6(Hd)	Strength I Unusual
7	Reverse Head, Basin Side Hurricane @ EL 15.35 + Debris on Gates **Resiliency Check**	- Dead (self wt.) - R/S at 100 YR SWL @ EL -1.0 - B/S tailwater at EL 15.85 - Debris Load at waterline	1.2(D+Hs+IM)	Extreme I Extreme
8a	Maintenance Dewatering Gate	- Dead (self wt.) - R/S SWL @ EL 8.0 - Dry Chamber EL -25	1.2(D) + 1.4(Hs)	Strength I Unusual
8b	Maintenance Dewatering Gate Reverse Head	- Dead (self wt.) - B/S SWL @ EL 8.0 - Dry Chamber EL -25	1.2(D) + 1.4(Hs)	Strength I Unusual
9[7]	Emergency Dewatering	- Dead (self wt.) - R/S SWL @ EL 9.0 - Dry Chamber EL -25	1.2(D+Hs)	Extreme I Extreme
10	EQ (OBE) @ Normal Operation	- Dead (self wt.) - R/S SWL @ EL 8.0 - B/S tailwater at EL -1.0	1.2(D+Hs) + 1.0(OBE)	Extreme I Unusual
11	Lifting	- Dead (self wt.) - Weight of either one or three bulkheads stacked above - Vertical siltation load on framing members below EL -15.0	1.2(D) + 1.6(G)	Strength I Usual
12	Stored Position	- Dead (self wt.) - Supported on cradles at bottom of bulkhead	1.2(D)	Strength I Usual
13	Dogged Positions	- Dead (self wt.) - Supported by wheel axles dogged within the guide slots	1.2(D)	Strength I Usual

Notes:

1. Load Factors shall be used in conjunction with the USACE Performance Factor, α, per ETL 1110-2-584
2. One typical design for all operating bulkheads; the bottom bulkhead carries the most load and is the primary design.
3. The Hurricane 50-YearSWL and Riverine Flowline include similar loads and load factors, only the greater of the two shall be analyzed. In the MBSD the 50-YearSWL is higher (EL 14.85) and controls
4. The Riverine Design Grade at EL 16.65 is an Extreme Load Case. The TOW resiliency check is also an Extreme load case and much higher at EL 20.35. The Riverine Design Grade does not control.
5. Vessel impact on the gate monolith is not considered. The Debris impact is 500 Lbs/LF at the surface of the water.
6. Lateral siltation load will be applied up to EL -15.0 (siltation height of 10 feet).
7. D = Dead Load, Hs = Hydrostatic, Hd = Wave, G = Gravity (Vertical siltation), Qf = Wheel/Seal friction, IM = Debris impact, W = Wind, OBE/MDE = Seismic
8. Machinery Loads (Q) shall only be applied to the lift mechanisms (padeyes) and wheel assemblies. The loads shall be included in load combination 11.

Included in the LRFD procedure is the USACE performance factor α, which further reduces the design nominal resistance beyond the traditional resistance factor ϕ. For this project α is set to 0.85 because maintenance and repair may be difficult and disruptive and because brackish water will likely back up to the gate on the conveyance channel side.

d. Analysis and Design Description

The most highly-loaded bulkhead position (bottom bulkhead in the four-panel stack) is analyzed with a 3-D SAP 2000 finite element model. The model is used to determine the sizing of all structural members including truss girders, web frames, and diagonal/cross braces. The skin plate and ribs are included in the model; in addition to SAP analysis, hand calculations are also included for these items that follow ETL procedures. Main horizontal chords are designed as continuous members. Horizontal and vertical struts are modeled with pinned connections. **Figure 5.4-6** below shows two versions of the bulkhead gate model; the one in the background has the skin plate removed so that more of the framing elements are visible. All frames are assigned rolled shapes. Hydrostatic, hydrodynamic and lateral siltation loads are applied to the front or back faces of the skin plate depending on load case. The weight of the three bulkhead panels stacked above are imparted to the design bulkhead frame for operational, full stack lifting, and full stack dogged load cases; the weight of one panel stacked above is imparted to the design bulkhead frame for a separate dogged load case.

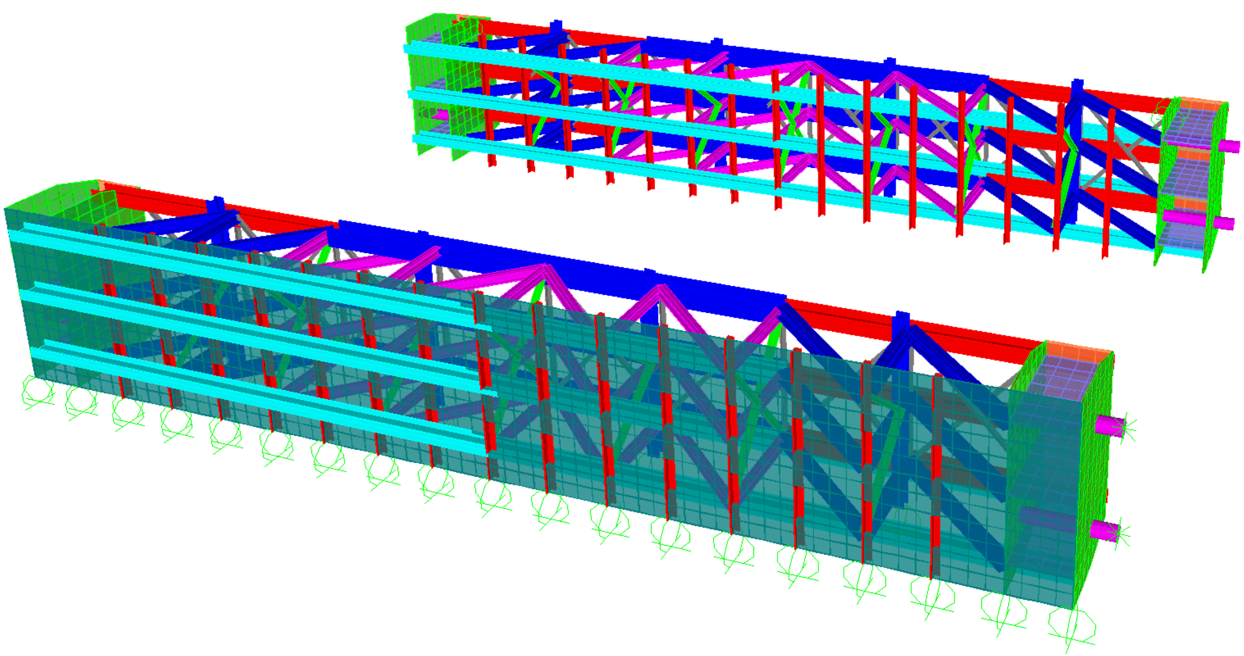


Figure 5.4-6: 3-D SAP2000 Model of Bulkhead Gate

A number of support conditions are explored with the model. These are as follows:

- Lifting conditions:
  - Single bulkhead panel is being lifting via four lifting eyes (in groups of two) welded to web members of the top truss
  - Full stack of bulkheads is being lifted via the two lifting bars; bottom bulkhead modeled
- Operational condition: bulkhead is installed in slots, resting on the sill plate at EL -25.0
- Stored condition: bulkhead is sitting upright on two supports at the bottom of the frame

- Dogged conditions:
    - bottom wheel dogged, one additional bulkhead stacked above
    - bottom wheel dogged, full stack of bulkheads on top
    - top wheel dogged, full stack of bulkheads on top

The load cases described in **Table 5.4-2** are input in SAP2000, with applicable load factors, and SAP's steel design process analyzes all members using the AISC Manual of Steel Construction (15^{th} Ed) and the USACE's ϕ*α*nominal resistance limit.

Fatigue of gate components due to cyclic loading is not expected to be a significant stress factor for the bulkheads. Per AISC, special fatigue consideration is required when the number of cycles of live load application exceeds 20,000, or 200 times per year over a 100-Year lifespan. While there are a number of high-stress events possible over the Diversion's life, these are unusual or extreme occurrences; cyclic loads of a large magnitude (e.g. storm waves or downstream dewatering pressure) will only affect some bulkhead panels for short periods of time. Routine loads on the bulkheads from operation head differential, by contrast, are light loads in terms of overall capacity.

The most likely source of fatigue will be vibration of the bottom bulkhead panel due to water flow underneath. This typically occurs due to defects in the bottom seal, either through ineffective design or wear and tear. This will primarily be addressed through bottom seal detailing, which will be expanded upon in the next design phase. Also, a benefit of using a single bulkhead panel for all positions is that no one bulkhead panel will always be subjected to the worst case vibration forces. Bulkheads used at the bottom of the stack can be rotated amongst the 16 total panels, so no one unit sees a full lifetime of worst-case vibration.

Detailed fracture critical analysis will be performed as needed in the next design phase. Additional detailing and local designs of bulkhead connections and components will also be included.

e. Results Summary

Hand calculations provide a required skin and intercostal plate thickness of 5/8 inches; this includes an assumed 1/16 inch sacrificial thickness to account for corrosion losses. Calculations of the composite skin/intercostal member show the intercostals shall be 6 inches deep.

All primary structural members in the 3-D SAP model are checked for moment, shear, axial force, and a demand/capacity ratio that combines axial and moment stresses. The DT has decided to limit the allowable maximum stress ratio for all frame elements to 0.85 to provide for sacrificial thickness and possible future fatigue detail requirements. The bulkhead members required by this design are shown in the figures below.

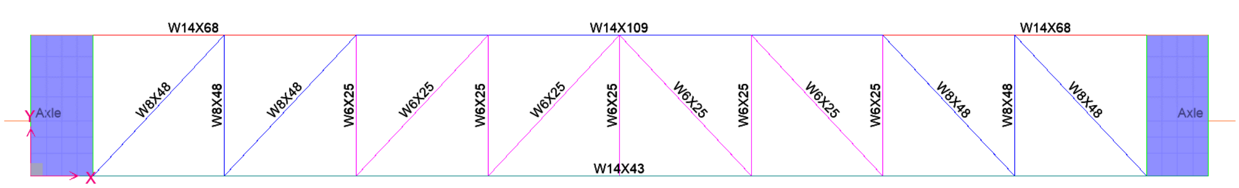


Figure 5.4-7: Plan View of Typical Bulkhead Truss

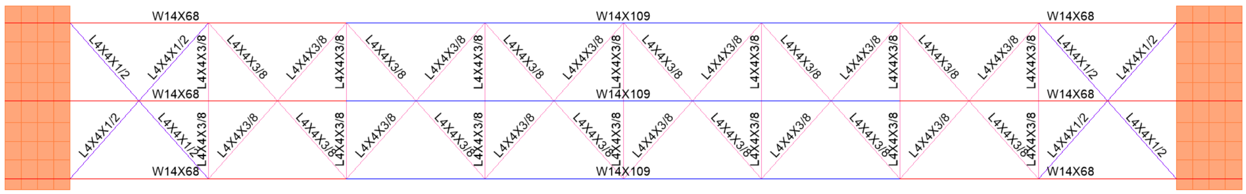


Figure 5.4-8: Elevation View of Downstream (Basin) Side Framing

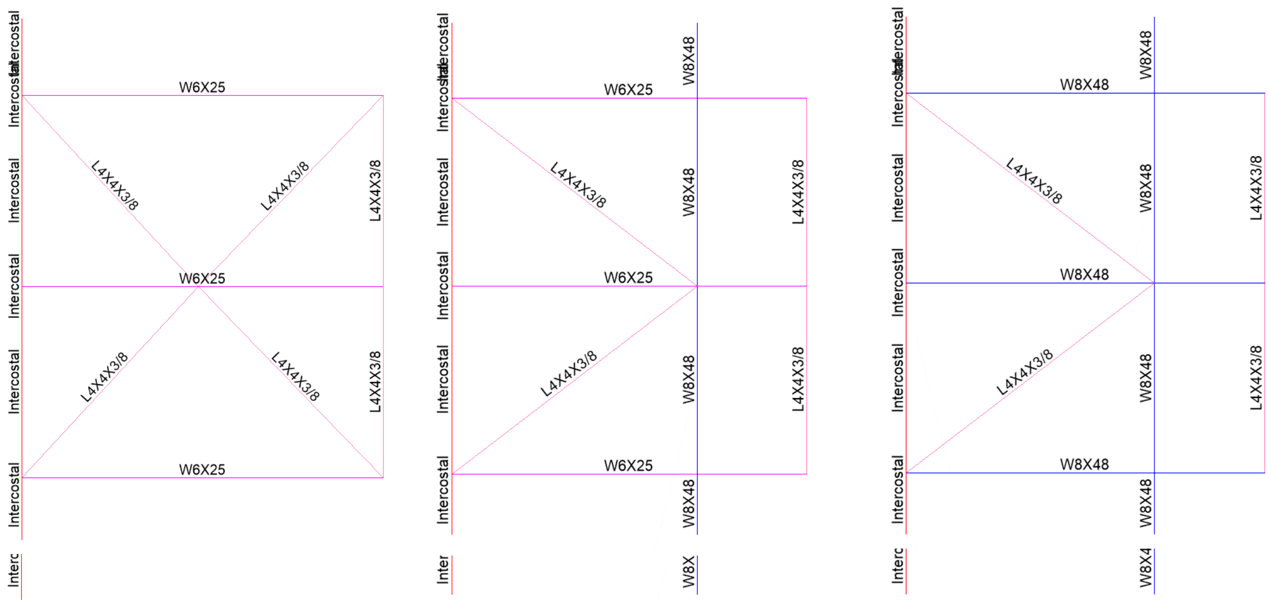


Figure 5.4-9: Cross-Sections of Bulkhead Framing

Deflection was also examined at a number of points around the bulkhead panel to confirm that deflections are not excessive, will not affect the proper function of the gate or its seals, and will not overstress the wheel box components. The maximum service-level bulkhead deflection was found to be 1.63 inches at the top midpoint of the panel (1.59 inches at the seal plate), which equates to approximately L/490 in the panel's longest direction.

5.4.4 Access Bridge Design

A maintenance and access bridge should be built on top of the gated structure to provide an access to the side of the gantry crane, and also, to connect the North and the South sides of the gated structure. The width of the bridge over the structure should be wider than the access bridge outside of the structure to accommodate a 25 ton mobile crane with extended outriggers. The mobile crane will be used to lift small equipment and tools that might be needed for the maintenance or repairs of the gantry crane in emergency situations. According to dimensions and specifications of existing mobile cranes, 20 ft clear width is enough for a 25 ton or smaller cranes with extended outriggers. The total width of the bridge over the structure needs to be 24 feet, including the side barriers. The access bridge outside of the structure can be a one lane bridge with a total width of 15 feet. No side walk-way is considered for this bridge. The bridge will be comprised of reinforced concrete slabs and precast pre-stressed concrete I-beams supported on the gated structure piers, for the segment of the bridge that is placed over the structure. The pre-stressed girders of the other segment of the bridge that is placed on each side of the gated structure will be supported by a series of steel pipe piles and concrete pile bent systems spacing 50 feet apart. **Figure 5.4-10** to **Figure 5.4-12** shows the maintenance bridge and cross sections of the 15 foot and 24 foot wide superstructures.

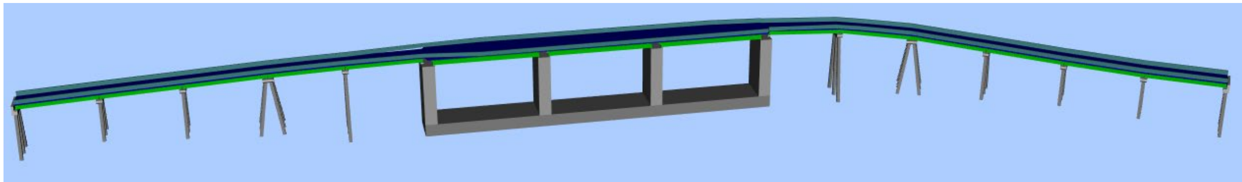


Figure 5.4-10: 3D view of Maintenance Bridge

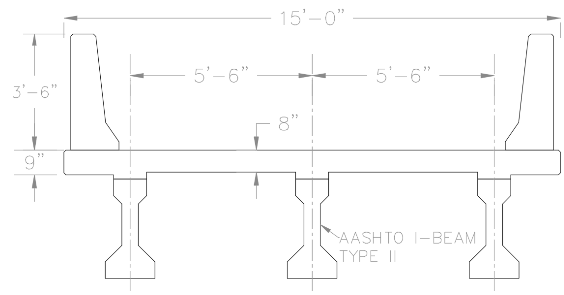


Figure 5.4-11: Section with 15 ft. width

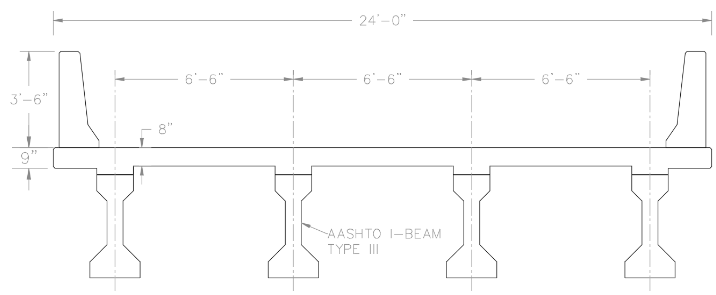
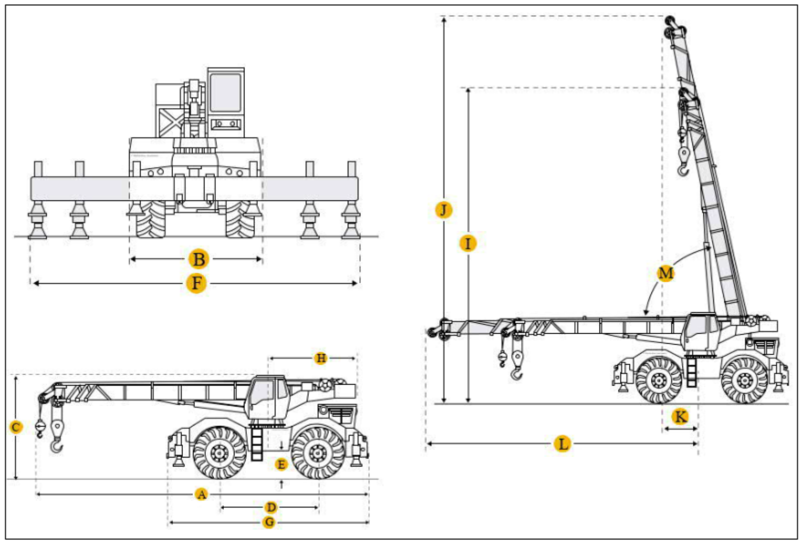


Figure 5.4-12: Section with 24 ft. width (over the gated structure)

a. Design Criteria and Load Conditions

AASHTO's LRFD vehicle and 25 ton mobile crane loads are used within the AASHTO load combinations for design of the superstructure and substructure of the bridge. The standard design vehicle that was used is H20 truck plus 640 lb/ft lane load that was modeled as a moving load over the slab to provide envelopes of responses for the design of both superstructure and substructure. In addition to standard moving loads, a Grove RT525 mobile crane in loaded condition with fully extended outriggers is used for the design of the bridge over the gated structure. To find the worst loading case of the crane, three boom lengths and radiuses with maximum loads were extracted from the load chart of the crane. Then the loaded boom modeled as a moving load to find extreme reactions under the outrigger legs in 360 degree rotation and finally, the reactions were moved along the span of the bridge to provide the most critical loading. The crane dimension and the load chart are shown in the **Figure 5.4-13** and **Figure 5.4-14**. **Figure 5.4-15** shows the modeled loaded crane with 3 different boom radius, length and angle moving 360 degree, and the worst reactions under the outriggers.



A	36.34 ft	E	1.63 ft	I	28 ft	M	75 deg
B	8.75 ft	F	18.38 ft	J	70 ft	Max Lift Capacity	50 Kips
C	10.8 ft	G	20 ft	K	10 ft		
D	10.09 ft	H	10.26 ft	L	65 ft		

Figure 5.4-13: Grove RT525 mobile crane dimension

ON OUTRIGGERS FULLY EXTENDED - OVER FRONT

Radius in Feet	Main Boom Length in Feet								
	32	38	44	50	56	62	68	74	80
10	50,000 (63)	39,500 (67.5)	37,500 (71)	36,950 (74)					
12	41,300 (58.5)	37,000 (64)	36,000 (68.5)	35,000 (71.5)	32,400 (74)				
15	31,750 (52)	31,500 (59)	30,950 (64)	30,300 (67.5)	29,750 (70.5)	29,150 (73)			
20	24,050 (38)	24,050 (49)	24,050 (56)	24,050 (61)	23,800 (65)	23,400 (68)	22,250 (70.5)	20,500 (72.5)	19,000 (74)
25	17,950 (9.5)	17,950 (37.5)	17,950 (47.5)	17,950 (54)	17,950 (59)	17,950 (63)	17,950 (66)	17,650 (68)	16,600 (70.5)
30		15,350 (18.5)	15,350 (37)	15,350 (46)	15,350 (52.5)	15,350 (57.5)	15,350 (61)	14,900 (64)	14,550 (66.5)
35	See Warning Note 16		12,850 (21.5)	12,850 (37)	12,850 (45)	12,850 (51.5)	12,850 (56)	12,850 (59.5)	12,500 (62.5)
40				10,550 (24)	10,550 (37)	10,550 (45)	10,550 (50.5)	10,550 (54.5)	10,550 (58)
45					8,590 (25.5)	8,590 (37.5)	8,590 (44)	8,590 (49.5)	8,590 (53.5)
50						7,070 (28)	7,070 (37)	7,070 (43.5)	7,070 (48.5)
55							5,880 (28.5)	5,880 (37)	5,880 (43.5)
60							4,930 (14)	4,930 (29)	4,930 (37.5)
65								4,150 (17.5)	4,150 (30)
70									3,490 (20)
Minimum boom angle (deg.) for indicated length (no load)									0
Maximum boom length (ft.) at 0 deg. boom angle (no load)									80

NOTE: Boom angles are in degrees. A6-829-007267 & -006832

Figure 5.4-14: Grove RT525 mobile crane load chart

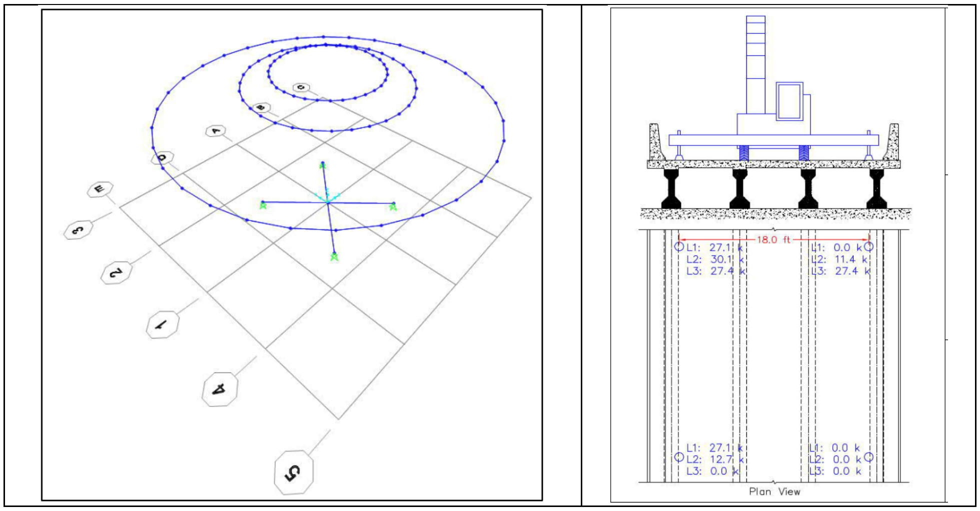


Figure 5.4-15: Loaded crane modeled as a moving load in 3 different boom lengths and angles, and the worst reactions under the outriggers

To design and evaluate the performance of the structure various limit states were considered. A limit state is a condition beyond which a system (or a component of a system) ceases to fulfill the function for which it was designed, i.e. the system or component is loaded beyond its capability to resist. The Limit State Objectives used in design are described in **Table 5.4-3**.

Table 5.4-3: Access Bridge Design Load Cases

AASHTO Designation	Limit State Objective	Load Factor
Service I	Limit compressive stress in girder and deck to maintain adequate factor of safety against concrete crushing	1.0 (DC) + 1.0 (DW) + 1.0 (LL)
Service III	Limit tensile stress in girder to maintain factor of safety against concrete tension cracking	1.0 (DC) + 1.0 (DW) + 0.8 (LL)
Strength I	Provide adequate resistance to girder "breaking" failure (normal vehicular use)	1.25(DC) + 1.5(DW) + 1.75 (LL)
Strength II	Provide adequate resistance to girder "breaking" failure (special design vehicles)	1.25(DC) + 1.5(DW) + 1.35 (LL)
Fatigue I	Limit stresses caused by repetitive vehicle live load	1.5 (LL)

a. Analysis and Design Summaries

Precast pre-stressed concrete girders acting as simply supported beams will carry loads from the superstructure to the substructure. The bridge structure requires an 8 inch thick slab between the girders and 9 inch thick slab for the overhangs. Live load distribution factors are calculated with different methods and reasonably conservative ones are used to distribute the live loads over the interior and exterior girders. **Figure 5.4-16** shows an example of live load distribution factors for moment. All the girders are designed based on the governing beam. The 24 foot wide bridge over the gated structure with a span of 68.5 feet requires four AASHTO I-Beam Type III girders and the typical 15 foot wide bridge outside of the gated structure with a span of 50 feet requires three AASHTO I-Beam Type II girders.

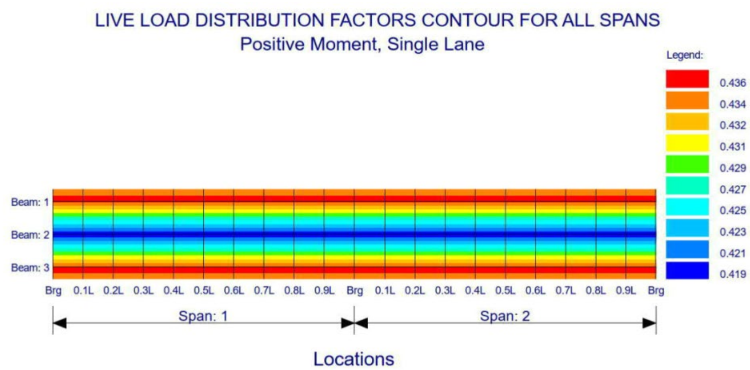


Figure 5.4-16: An example of live load distribution factor for positive moment

Driven 16-inch pipe piles with 0.5-inch thickness transfer superstructure load to the soil. The piles are directly connected to 2.5-feet x 3.5-feet x 16.5-feet pile bents and extended into the soil to EL -131.0. Each pile bent has 3 piles and to strengthen the bridge against lateral loads, the middle pile is vertical and the sides are battered 6V:1H outward. Also, longitudinal battered piles are employed to increase the longitudinal strength of the bridge. The approach bridges outside of the gated structure have 5% slopes to connect grade elevation of approximately 10 feet to the top of the gated structure; the bridge is horizontal across the structure.

Various finite element models were developed to analyze different structural elements. **Figure 5.4-17** shows an example of a 2D model developed to analyze the slab. The size and number of prestressed girders as well as reinforcing strands are determined by the procedure described in the AASHTO and checked with LADOTD Bridge Design Manual recommendations. These references were used to design the slab and pile bent reinforcements as well. In order to check the structural capacity of the pile, 2 load cases are checked. Load case one was dead load plus wind load and load case two was dead and live load plus braking load. The loads resultant applied at the center of the pile bent and US Army Corps of Engineers pile group analysis program (CPGA) was used to check the pile capacity (**Figure 5.4-18**). It should be noted that the influence of the downdrag and the backfill over the battered piles were also considered in the pile analysis procedure.

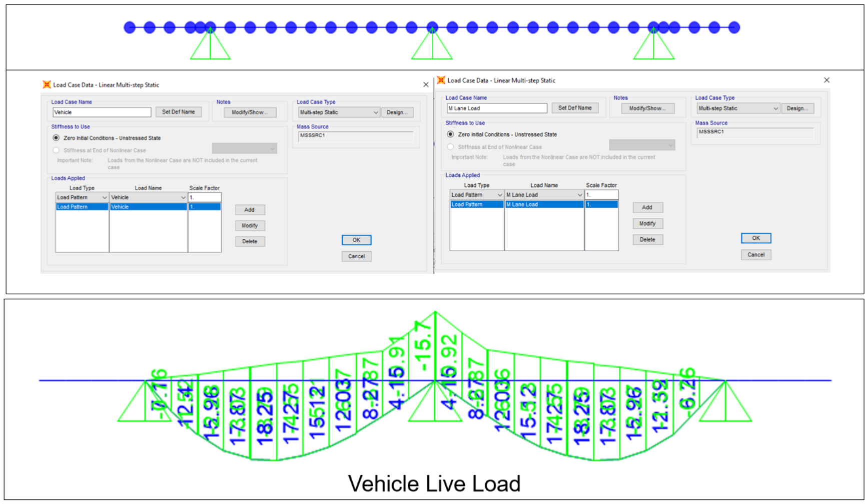


Figure 5.4-17: An example of a SAP2000 model developed for the deck design and the envelope of the bending moment diagram for the vehicle live load

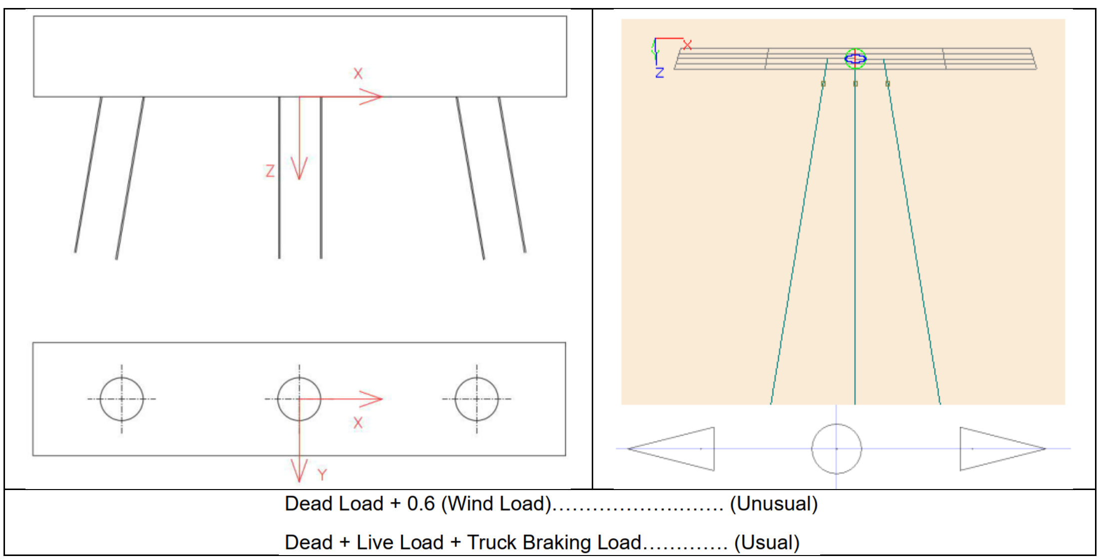


Figure 5.4-18: Pile load combinations and CPGA model

5.5 Gantry Crane Rail Beam and Platform

5.5.1 Gantry Crane Rail Beam Over Gate Structure

The closure and emergency gate system(s) for the diversion structure are controlled by steel truss bulkhead gates that will be operated by a rail-mounted portal gantry crane. The intake monolith consists of 3-bays each of which are 70 feet – 6 inches center to center. The gate piers are 8 feet thick therefore, the clear span between the gate pier walls is 66 feet – 6 inches. The supporting structure for the gantry crane rail is comprised of three (3) precast, prestressed concrete Type IV AASHTO girders with a concrete deck platform and diaphragm infill. The crane rail beam is designed for the strength and stiffness requirements to support the crane and maintain operability for the crane rail system. See **Figure 5.5-1** for proposed section through the gantry crane rail beam.

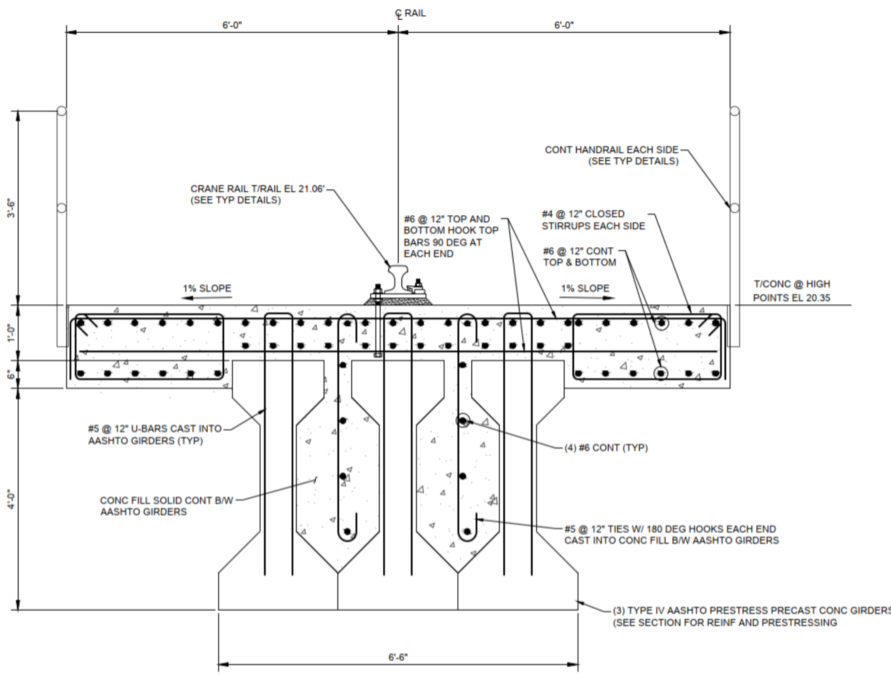


Figure 5.5-1: Gantry Crane Rail Beam Cross-Section

The dimensions of the gantry crane rail beam structure are controlled by the width of the gantry crane leg, wheels, motor and ancillary equipment mounted to the lower portion of the gantry leg. A 12 foot wide platform was assumed in order to maintain a 36 inch wide clear walkway path for personnel. Note that based on preliminary data furnished by the crane manufacturers, adequate space can be maintained on one side of the crane leg at all positions as to allow for the 36 inch personnel walkway width. Steel handrail is provided on the outside face of the concrete and crane rail is fixed to a continuous steel sole plate which will be installed on non-shrink epoxy leveling grout after placement of the concrete gantry crane beam and slab structure.

a. Design Criteria and Loading Conditions

Design loads for the gantry crane rail beam structure include self-weight dead load of all fixed components, platform level live load (assumed to be 100 psf per IBC code requirements for egress), wind loads on the crane and beam structure, longitudinal braking load of the crane as well as the predominant load effect from gantry crane self-weight and lifted load. Gantry crane wheel loads were estimated based on required bulkhead lifting load rated capacity and compared to preliminary wheel load data furnished by crane manufacturers. Several operational conditions were considered for design of the gantry crane rail beam structure since the lifted load will vary based on magnitude of load and position along the span. Load analysis for the gantry crane rail beam structure is fully developed in Appendix D.5.1.

Load cases for design were generated based on the varying crane activities as outlined in **Table 5.5-1**.

Table 5.5-1: Gantry Crane Rail Beam Load Cases

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Crane Stationary + Lifting 4 Bulkheads + Friction/Drag Force on Wheels	- Crane stationary positioned +/- 10' from gate pier centerline - Crane self-weight + lifting 4-bulkheads - Include additional load due friction and drag force on bulkheads - Live load on platform - No impact on lifted load	$1.2*(D + D_{CRANE}) + 1.6*(LL + LL_{CRANE})$	Usual
2	Crane Stationary + Lifting 4 Bulkheads + Impact Factor + Operational Wind	- Crane stationary positioned +/- 10' from gate pier centerline - Crane self-weight + lifting 4-bulkheads - Live load on platform - 25% impact on lifted load - 50 mph operational wind	$1.2*(D + D_{CRANE}) + 1.6*(LL + 1.25*LL_{CRANE} + W)$	Usual
3	Crane Traveling + Carrying 2 Bulkheads + Impact Factor + Operational Wind	- Crane traveling - Crane self-weight + lifting/carrying 2-bulkheads - 25% Impact - Live load on platform - 50 mph operational wind	$1.2*(D + DL_{CRANE}) + 1.6*(LL + 1.25*LL_{CRANE} + W)$	Usual
4	Crane Traveling + Carrying 2 Bulkheads + Impact Factor + Braking Force (No Wind)	- Crane traveling - Crane self-weight + lifting/carrying 2-bulkheads - Live load on platform - 25% Impact - 10% longitudinal braking force - No wind	$1.2*(D + D_{CRANE}) + 1.6*(LL + 1.25*LL_{CRANE} + BL_{CRANE})$	Usual
5	Hurricane Storm Condition + Crane Stationary, Tied-Down and Stowed + Wind	- Crane stationary positioned centered on gate piers at tie-down and stowage position - Crane self-weight - 165 mph storm wind	$(1.2 \text{ or } 0.9) *(D + D_{CRANE}) + 1.0*(W)$	Extreme

b. Analysis and Design Summaries

The gantry crane rail beam structure was analyzed using a combination of hand calculations, spreadsheets and SAP2000 finite element analysis model. The SAP2000 model was used to combine all applicable load effects and load combinations modeling the beam as a simple-span between gate pier supports. A single AASHTO girder was modeled and the stiffness properties were adjusted to account for the composite action and 3-sections of girders acting together. Configuration of the AASHTO girders is such that each girder is spaced with bottom flange tight to the adjacent beam. The concrete top deck and side support beams are then placed with reinforced concrete filling the diaphragm between the webs of adjacent girders. When fully cured, the reinforced concrete top slab and edge beams forms a composite section that is designed to support the combined load effects described above while providing lateral stiffness to resist out of plane transverse wind loads that may act on the crane, lifted bulkheads and crane beam system itself.

The analysis results from the SAP2000 model were used to design the AASHTO girder beam. Due to the strength and stiffness requirements for the gantry crane, three Type IV precast, prestressed concrete girders were selected. A spreadsheet was developed to design the prestressing strand and mild reinforced steel to be placed in the girders as well as the reinforced concrete platform and edge beams. Allowable stresses were considered in the girder design in accordance with ACI and AASHTO provisions. Prestressing strands are harped at 0.4 x the span length to accommodate initial prestress action. For the gantry crane rail beam girders, 26-total strands are required with 8-harped strands and 18-straight strands positioned per typical AASHTO beam configurations using a 0.6 inch diameter, Grade 270 strand.

Stiffness and deflection requirements for the gantry crane rail beam are per CMAA Specification No. 70 – *Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes.* The operational criteria for live load deflection of the supporting structure is defined to be Span/600. The gantry crane rail beam is designed to accommodate these requirements with assistance from initial prestress and camber in the girders which relieves a portion of the self-weight dead load deflection in the system.

Full analysis and design calculations for the gantry crane rail beam structure are included in the Structural Calculation Appendix.

5.5.2 Gantry Crane Platform

The gantry crane platform is designed to provide a laydown area for the bulkhead gates to be stored when not in use. The gantry crane will only use this platform to perform these operational activities, such as storing the bulkheads.

a. Design Criteria and Loading Conditions

Load cases were generated based on varying crane activities. Two usual load cases were analyzed over the gantry crane platform for the crane traveling while carrying up to two bulkheads – one case considers operational wind and the other case considers a braking force with no concurrent wind. The third load case is an unusual case analyzed over the platform for the crane traveling without any bulkheads under a maximum out-of-service wind load of 100 mph for non-hurricane events. None of the three load cases consider any additional load due to silt or hydrostatic forces acting on the bulkheads, as this is assumed to have been cleared prior to travel. Load combinations in general accordance with ASCE 7-16 are used for design of the concrete superstructure with ACI 318 provisions. In addition, all load cases conservatively assume a 25% impact applied to the moving crane wheel loads. Load combinations used for the design are as follows:

Table 5.5-2: Gantry Crane Platform Load Cases

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Crane Traveling + Carrying 2 Bulkheads + Operational Wind	- Crane traveling - Crane self-weight + lifting/carrying 2-bulkheads - No additional load due to silt, hydrostatic - 25% Impact - 50 mph transverse operational wind	$1.2*D + 1.6*(LL + 1.25LL_{CRANE} + W)$	Usual
2	Crane Traveling + Carrying 2 Bulkheads + Braking Force (No Wind)	- Crane traveling - Crane self-weight + lifting/carrying 2-bulkheads - No additional load due to silt, hydrostatic - 25% Impact - 10% longitudinal braking force - No wind	$1.2*D + 1.6*(LL + 1.25LL_{CRANE} + BL_{CRANE})$	Usual
3	Crane Traveling without Bulkheads + Max. Out-of-Service Wind Load	- Crane traveling - Crane self-weight - No additional load due to silt, hydrostatic - 25% Impact - 100 mph transverse max. out-of-service wind	$1.2*D + 1.6*(LL + 1.25LL_{CRANE} + W)$	Unusual

b. Analysis and Design Summaries

The concrete superstructure of the gantry crane platform will consist of a 9-inch top slab supported on a rectangular pile cap beam, forming a T-beam shape. The design width of the concrete top slab of the platform considers the width of the crane legs, as well as a walkway clearance on either side of the crane rails. Crane legs are assumed to be 6 feet wide, therefore, a 12 foot platform width allows for 3 feet of walking space on either side of the crane leg. Handrails are to be mounted on the outside edge of the platform concrete. This 9 inch x 12 foot top slab will be supported on a 3.25 foot thick x 7.5 foot wide pile cap beam, sized based on this 12 foot top width. The following figure shows the proposed section through the gantry crane platform.

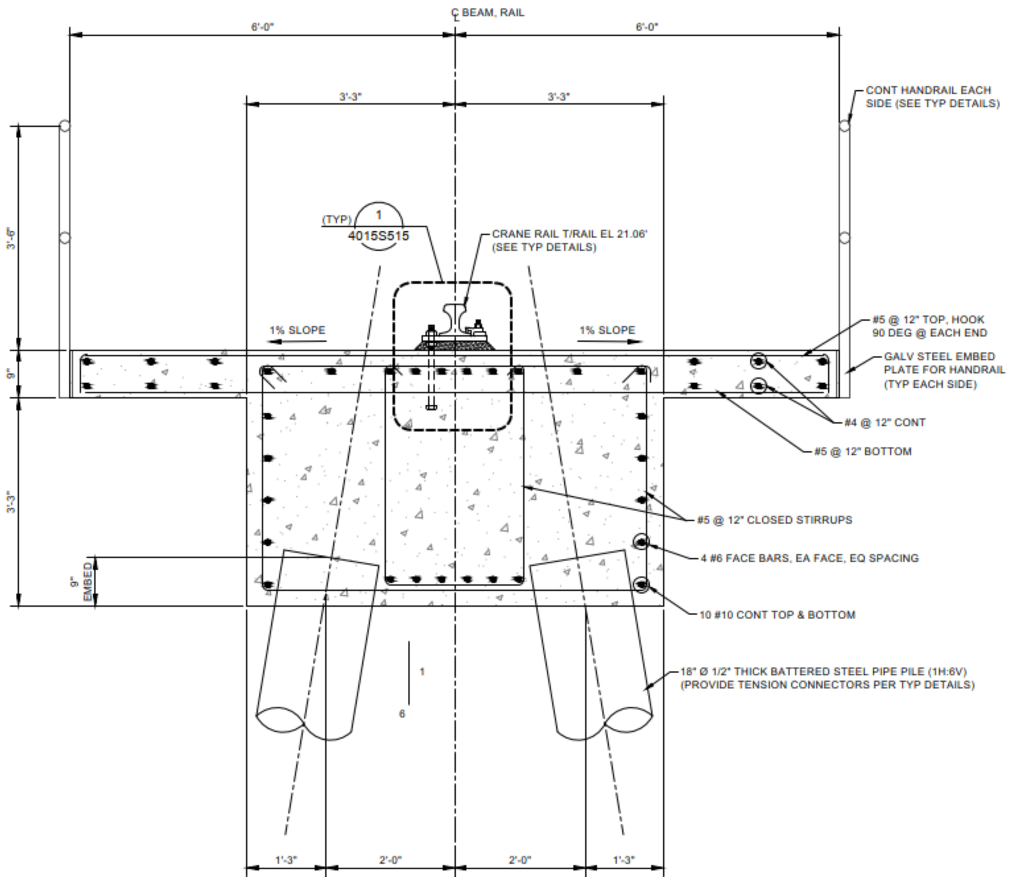


Figure 5.5-2: Gantry Crane Platform Cross-Section

The gantry crane platform will be supported by 18 inch x 0.5 inch open end steel pipe piles connected directly to the concrete pile cap beam. The overall pile layout and spacing under the platform was designed to clear the adjacent piles for the bridge and bulkhead storage beams, while accounting for the minimum pile spacing requirement and minimizing the maximum span of the concrete platform to a practical limit of less than 24 feet. Considering these spacing restrictions, the platform design uses 5 pile bents with a maximum span length of 20.25 feet along the platform. Each pile bent consists of two piles battered outward at 1H:6V, for a total of 10 piles. The total length of the platform is 92 feet. Piles under the gantry crane platform will have the same tip elevation as those under the gate monolith at EL -131 to minimize differential settlements across adjacent structures.

A SAP2000 structural finite element model was used to analyze stresses, displacements, and pile reactions. All the applicable loads for each load combination described in **Table 5.5-2** are applied in the model. Both dead and live crane wheel loads were defined as moving live loads in the model. Using factored loads from the SAP2000 model, reinforcement in the pile cap beam and top slab is designed for minimum reinforcement, flexural, shear, and torsional resistance.

As an additional check, the load effect on piles is evaluated by running CPGA for a unit width of one pile bent using the most critical unfactored load case with the maximum service-level pile reaction from SAP2000 results. Both CPGA and SAP2000 analyses for structural pile capacity consider factors of safety per USACE EM 1110-2-2906 for an assumed tension and compression pile load test. Comparable results between the SAP2000 model and CPGA output for pile reactions and lateral displacements demonstrated all piles to be adequate for the envelope of load conditions and combinations. See **Appendix D5.2** for the full structural calculation package on the gantry crane platform design.

5.6 Bulkhead Storage Support Grade Beams

5.6.1 General Description

The bulkheads will be stored on two concrete support grade beams, one for each end of the bulkhead. Both concrete grade beams will span between the two gantry crane rail platforms. Each end of the bulkhead will rest on two vertical support legs for a total of 4 support legs per bulkhead – supports will be centered on the width of each grade beam. Paint markings will be included on top of concrete to guide proper placement of the bulkheads onto the grade beam supports by the gantry crane.

5.6.2 Design Criteria and Loading Conditions

The grade beams will be designed for dead load only, which includes the beam self-weight plus the self-weight of the bulkheads. The design assumes the weight of each steel bulkhead to be 150 kips (i.e., 75 kips per grade beam). Assuming the bulkheads can be placed anywhere along the beam, the grade beams will be designed for the worst-case loading due to varying bulkhead positions. The only standard ASCE load combination considered for design is 1.4*D. Further analyses in the next submittal will consider also lateral loading, such as wind acting on the storage support grade beams.

5.6.3 Analysis and Design Summaries

Each concrete beam is proposed to be 3 feet wide x 3 feet thick and will span for a total length of 43.25 feet between each gantry crane platform. The two grade beams will each be supported by a total of four 16 inch x 0.5 inch vertical steel pipe piles embedded 6 inches into the concrete beam. Piles will be equally spaced at 13 feet along the beam and are to have a pile tip elevation at EL -131. See **Figure 5.6-1** for proposed section through the bulkhead support grade beams.

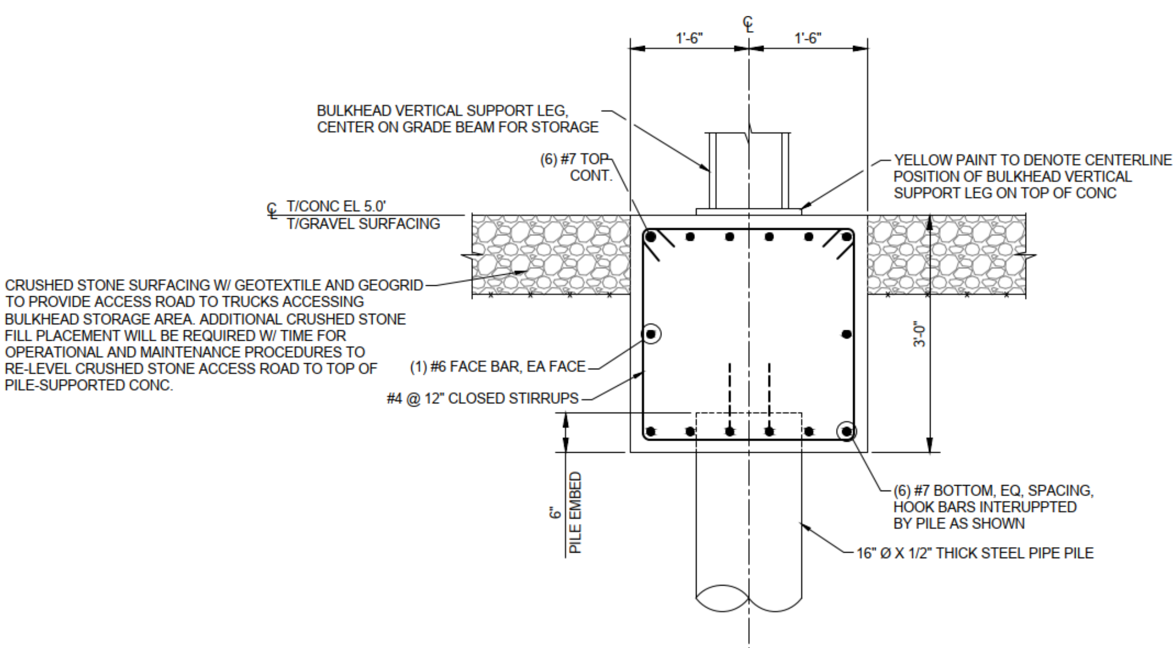


Figure 5.6-1: Bulkhead Storage Support Grade Beam Cross-Section

A SAP2000 frame model for one concrete grade beam was created to analyze stresses and pile reactions due to the imposed loads. The beam is defined as a frame section in SAP2000 and modeled as one continuous beam with the moment-released, vertical joint restraints at each pile location. Each bulkhead self-weight is distributed equally to the vertical support legs, which was applied as two point loads in the SAP2000 model. Using factored loads from SAP2000 results, reinforcement in the concrete beam is designed for the maximum moment and shear due to varying bulkhead positions along the beam. All 16 inch x 0.5 inch steel pipe piles were determined to be adequate at the stated pile tip elevation of EL -131, which was checked using the pile capacity curve for the maximum unfactored joint reaction at each pile location in SAP2000. Note that a pile tip elevation of EL -131 was selected to match the tip elevations of adjacent piles which are placed within the influence zone of the gate monolith structure in order to minimize the potential for differential settlements between the structures. All structural calculations for the bulkhead support grade beams can be found in **Appendix D.6**.

5.7 Electric Building and Generator Platform

5.7.1 General Description

The electric building and generator platform will be designed and constructed immediately adjacent to the outside face of the gantry crane rail platform, separated by approximately 1 foot. The total structure will consist of a pile-supported concrete platform slab supported by 4 concrete pile cap beams, all of which support the electrical building and an elevated platform for the generator and fuel storage tank. A set of steel stairs from the grade elevation to the platform level will grant access to the electric building on the platform. On the platform, two sets of steel stairs will be provided – one set from the platform level leading to the generator on the elevated platform and another set from the elevated platform leading to the gate pier elevation. Note that the elevated grating walkway around the generator is provided to allow maintenance access to the equipment from an elevated position above the fuel storage tank.

5.7.2 Design Criteria and Loading Conditions

The preliminary design loads for the concrete platform include the self-weight of all fixed components, equipment live load for the electrical building, and typical deck live load for the remaining platform surface (assumed to be 100 psf per IBC standards for egress). This preliminary design phase considers vertical dead and live loads. Further analyses in the next submittal will consider also lateral loading such as wind acting on the platform, equipment and electrical building. Load cases were generated based on standard load combinations from ASCE 7-16 as shown below in **Table 5.7-1**.

Table 5.7-1: Preliminary Electric Building and Generator Platform Load Cases

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Dead Load	- Self-Weight - Electric building dead load - Generator dead load	1.4*D	Usual
2	Dead Load + Live Load	- Self-Weight - Dead and live load for electric building - Generator dead load - Deck live load	1.2*D + 1.6*L	Usual

5.7.3 Analysis and Design Summaries

The pile-supported concrete platform slab will be 28 feet wide x 32 feet long x 1 foot thick supported on two 2 foot x 2 foot concrete pile cap beams spanning in each longitudinal and transverse direction for a total of four concrete beams. The platform will be supported by four piles located at the intersection of each concrete beam. Design is based on a past experience and data on generator equipment with an assumed 13 foot x 5.5 foot generator weighing 8,200 lb, as well as a 2,000-gallon tank (4,700 lb self-weight) filled with 60 pcf diesel (20,000 lb maximum dead load) for a total dead load of about 500 psf for the generator footprint. The electric building consists of four 12 feet wide x 10 feet high x 8 inches thick concrete walls with an 8 inch concrete slab roof for a total self-weight equal to approximately 550 psf acting over the footprint of the building. Equipment live load for the electric building is assumed to be 250 psf. All four piles will be 18 inch x 0.5 inch vertical steel pipe piles embedded 6 inches into the pile cap beam with a tip elevation at EL -131. Note that a pile tip elevation of EL -131 was selected to match the tip elevations of adjacent piles which are placed within the influence zone of the gate monolith structure in order to minimize the potential for differential settlements between the structures. The following figure shows the proposed cross-section through the electric building and generator platform in both the short (detail 1) and long (detail 2) directions.

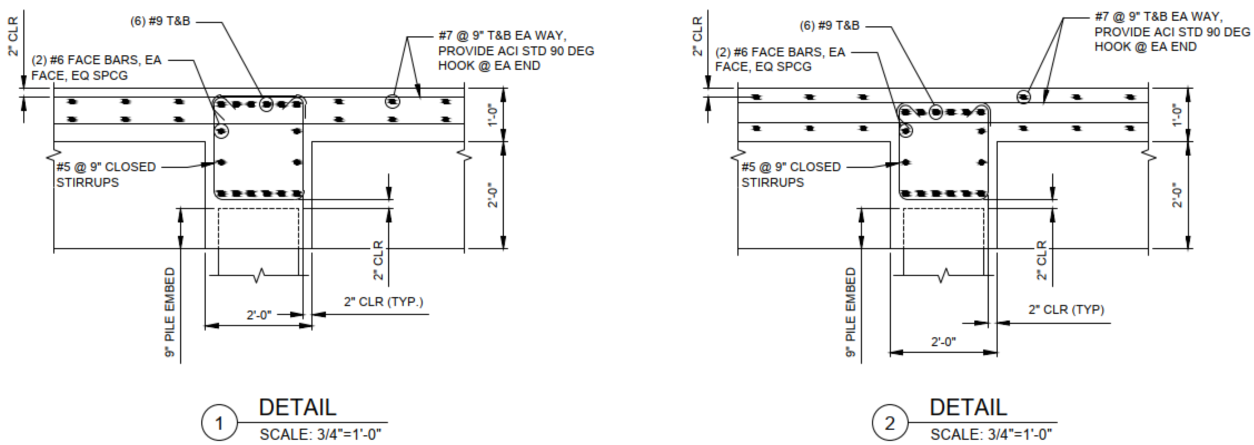


Figure 5.7-1: Electric Building and Generator Platform Cross-Section

A SAP2000 structural finite element model was used to analyze stresses and pile reactions for the concrete platform considering all applicable loads for each load combination in **Table 5.7-1**. The platform slab is modeled as a shell area element, while all the concrete pile cap beams and steel piles are modeled as frame elements. All dead and live loads are applied as uniform area surface pressures to the appropriate designated area for each component of the structure. Analysis results from the SAP2000 model are used to design the reinforcement in the concrete platform slab and pile cap beams. Maximum unfactored pile reactions from SAP2000 results are used to verify pile size and structural pile capacity for a tip elevation at EL -131. The full structural calculation package for the electrical building and generator platform is included in **Appendix D.7**.

5.8 Mississippi River Levee (MRL) Tie-Ins

5.8.1 General Description

The U-Frame Intake Structure is enclosed on both the north and south sides with inverted T-Wall monoliths that form the Mississippi River Levee (MRL) tie-in. The joint between the U-Frame and T-Wall monoliths will be sealed with water stops which can provide lateral movements between these two structures. There is a total of seven (7) MRL T-walls, three (3) (N-1 thru N-3) T-Walls located on the north and four (4) (S-1 thru S-4) located on the south side of the U-Frame. The T-Walls on the North side of the U-Frame extend approximately 156 feet and the South side T-Walls extend approximately 204 feet. The MRL T-Walls will be an in-the dry construction. There is no need for braced construction to construct these T-Walls. The top of the base slab for the all MRL T-Walls is at EL 12.0 and TOW EL 20.35. Settlement induced bending moment on both North and South side T-Walls from settlement has been considered for design as per geo-tech report.

5.8.2 Design Features

5.8.2.1 Base Slab and Stem

All MRL monoliths have a TOW EL 20.35, TOS EL 12.0, 3 foot thick base slab and 3 foot – 9 inch thick stem wall. Two T-Walls (N-3 and S-4) have a 23 foot – 2 inch wide base slab to support a proposed emergency maintenance roadway to railway track on the land side. The other five T-Walls have a 15 foot wide base slab and do not support a maintenance road. Top of roadway elevation is at EL 16.85. A continuous cut-off sheet pile curtain wall is embedded 9 inches into the base slabs. All monoliths are pile supported by 14 inch steel H-piles with pile tips set to mitigate differential settlement among monoliths. Settlement calculations are not performed in the 60% design phase. See **Appendix D** for pile layout, tip elevations, sizes and other design features. Batter piles are battered at 1:12 slope on north and south side T-Walls except for N3 and S4 battered at 1:6.

5.8.2.2 Cut-off Wall Sheet Pile

The cut-off wall of sheet piling is provided to limit seepage to a tip elevation at EL -65.0, and the embedment criteria is specified in the Geotechnical Report Section 4. Cutoff sheet pile will extend via a sheet pile transition wall into the levee embankment. Cut-off sheet pile will be extended 30 feet beyond the T-Wall at the guide levee tie-in for the T-Wall monoliths. The top of the sheet pile at these locations is set to match with the guide levee tie-in crown elevation.

5.8.3 Design Criteria and Loading Conditions

The load cases as described in the MBSD Design Criteria Table 5-5 (**Appendix A**) are used as a guide for creating the load cases evaluated in the analysis, which were considered most likely to control the design. Engineering judgment is used in selecting the load cases by comparing the magnitude of the applied loads and the allowable overstress. Only the basic load cases are evaluated. The basic load cases selected for the analysis are as stated in the table below.

The analysis evaluated the pervious and impervious cut-off wall uplift conditions. The following table shows the selected load cases. The hydraulic grade and design grades are from the MBSD Design Criteria **Appendix A**.

Table 5.8-1: MRL T-Wall Design Load Case Summary

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction (w/ Backfill) + Downdrag (no uplift)	- Dead (including bridge wt) - Live Load on Access Bridge (Moving load) - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall)	1.6 (D+EH+EV+ES+Ds)	Unusual
2	Construction (w/ Backfill) + Downdrag + Wind (no uplift)	- Dead (including bridge wt) - Live Load on Access Bridge (Moving load) - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+ES+Ds+W)	Unusual
3a	Water @ Design SWL No Wind (Impervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
3b	Water @ Design SWL No Wind (Pervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual

No.	Load Case Name	Description	Factored Load Combination	Load Category
4a	Water @ Design SWL + Wind (Impervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual
4b	Water @ Design SWL + Wind (Pervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual
5a	Water @ Design SWL + Wave (Impervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual
5b	Water @ Design SWL + Wave (Pervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual

No.	Load Case Name	Description	Factored Load Combination	Load Category
6a	Water @ Design SWL + Wind + Unusual Barge Impact (Impervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (140 mph, min. 50 psf) [1] - Unusual Barge Impact (225 kips) [1]	1.6 (D+EH+Hs+Hu+W+BI)	Unusual
6b	Water @ Design SWL + Wind + Unusual Barge Impact (Pervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (140 mph, min. 50 psf) [1] - Unusual Barge Impact (225 kips) [1]	1.6 (D+EH+Hs+Hu+W+BI)	Unusual
7a	Reverse Head (Impervious)	- R/S @ El. 0.0 - B/S @ El. 9.35 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
7b	Reverse Head (Pervious)	- R/S @ El. 0.0 - B/S @ El. 9.35 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual

No.	Load Case Name	Description	Factored Load Combination	Load Category
8a	Reverse Head + Wind (Impervious)	- R/S @ El. 0.0 - B/S @ El. 9.35 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
8b	Reverse Head + Wind (Pervious)	- R/S @ El. 0.0 - B/S @ El. 9.35 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
9a	Water to TOW (Impervious) **Resiliency Check**	- R/S @ El. 20.35 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
9b	Water to TOW (Pervious) **Resiliency Check**	- R/S @ El. 20.35 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
12	Normal Operation	- R/S @ El. 6.0 - B/S @ El. 1.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Bridge Live Load	2.2 (D+EH+EV+Hs+Hu+L)	Usual

Notes: 1) Vessel impact on the MRL T-Wall monolith is based on The Hurricane Strom Damage Risk Reduction System Design Guidance (HSDRRSDG). The barge impact used from the HSDRRS-DG is based on a 100-Year SWL with wind speed of 140 mph. The extreme case 1 with 450 kips barge impact for 500-Year SWL is a resilience check and is not considered. Since the 500-Year SWL is not available in the design criteria.

2) Pile Load tests shall be performed. The factor of safety for piles by Load Category is:

Usual – 2.0
Unusual – 1.5
Extreme – 1.2

3) The dead load of the Access Bridge on the MRL T-Wall Monolith is included in all load cases.
4) Live load + Dead load for Access bridge load is a separate load case (12) without hurricane loads on T wall.

5.8.4 Analysis and Design Summaries

Analysis of the 3-dimensional MRL T-Walls is performed using a combination of hand calculations, excel spreadsheets and GROUP2016. The hand calculations, provided in the **Appendix D**, consider the self-weight of the T-Wall monolith, the water weight and pressure, the soil weight and pressure, and uplift forces. There are no unbalanced loads considered during the analysis due to the proposed deep soil mixing beneath the base slabs.

The vertical, lateral and moment forces for each are individually calculated and are added together to create the load combinations. The load combinations are then entered in GROUP2016 to analyze the pile group and to determine the individual pile demands. Soil layers and parameters entered in GROUP2016 are provided by AECOM. Once the calculated loads, pile properties, and soil parameters are entered in GROUP2016, the results are used to determine the capacities and deflection of the piles. The calculation of the pile capacities is done by using the pile capacity curves for 14-inch H-piles, provided by AECOM. The pile design capacities are determined based on a factor of safety of 2.5, assuming PDA tests will be conducted during construction. The deflection of the piles is also checked by using the allowable deflection values stated in the HSDRRS Design Guidelines.

Hand calculations were also performed to check the design of the stem wall and the base slab of the inverted T-Wall monolith in accordance with the MBSD Design Criteria. The stem and base slab of the T-Wall monoliths are sized by checking only the shear strength of the concrete to determine the necessary thickness. Shear is checked using EM 1110-2-2104 (Design of Concrete Hydraulic Structures). Moment calculations for the stem and base slab were performed and steel reinforcement was chosen based on requirements. The stem of the T-Walls is designed using the pressure calculations of the TOW load case. The base slab is designed by analyzing the weight of slab, weight of soil, weight of water, uplift and the pile reactions from the governing pile load from GROUP2016. Two of the base slabs have an additional load due to the HS-20 Truck Load. Factored concrete design loads shown in **Table 5.8-1** are used to confirm the adequacy of the stem wall and slab thickness.

The MRL T-Wall's stem is 3 feet – 9 inches thick and 8 feet – 4 inches tall. The base slabs have a top slab elevation of EL 12.0 and are 3 feet thick. The N-3 and S-4 T-Walls have 14-inch plumb and battered H-piles spaced at 8 feet o.c. which have a tip EL -85.0. The other five T-Walls have 14-inch plumb and battered H-piles spaced at 8 feet o.c. which have a tip EL -50.0. North and South side T-Wall piles are embedded into the base slab 9 inches to create a pinned connection. See **Appendix D** for pile layout, tip elevations, sizes and other design features.

5.9 Transition Structure

5.9.1 General Description

There is a total of thirty-six (36) Transition T-walls, eighteen (18) identical T-Walls on North and South side of Conveyance Channel. All Transition T-Walls have a TOW EL 15.85. The top of base slab elevations decrease as the T-Walls approach the guide levee tie-ins. The top of base slab at the U-Frame is EL -27 and gradually raises to EL 0.0 at the guide levee tie-ins. The base slab width and thickness for monoliths W-1 thru W-6 is 30 feet and 7 feet, respectively. Monoliths W-7 thru W-12 base slab width and thickness is 30 feet and 5 feet, respectively. Monoliths W-13 thru W-16 base slab width and thickness is 15 feet and 5 feet, respectively. Monoliths W-17 and W-18 base slab width and thickness is 15 feet and 3 feet – 6 inches, respectively. The stem wall for monoliths W-1 thru W-12 are 2 feet – 6 inches thick at the top and thicken at a 1H:12V slope toward the land side. Monoliths W-13 thru W-18 have uniform walls with a thickness of 2 feet – 6 inches. The Transition T-Walls W-1 thru W-5 are supported by 24-inch diameter by ¾-inch pipe piles, monoliths W-6 thru W-9 are supported by 24-inch diameter by 5/8-inch pipe piles, monoliths W-10 thru W-14 are supported by 24-inch diameter by ½-inch pipe piles, monoliths W-15 and W-16 are supported by 24-inch diameter x ¾-inch pipe piles and monoliths W-17 and W-18 are supported by 24-inch diameter by ½-inch pipe piles. All batter piles are battered at 1:3. All monoliths will have a 6-inch thick lean concrete (2500 Psi) mud mat which is placed below base slab as a level platform for base slab construction. Geo-tech has designed the DMM panels below the monoliths in between the piles to avoid SIBMs on the piles and unbalanced loads on monoliths. Geo-tech reports and civil drawings shows the extent of DMM panels in the transition wall area.

All Monoliths W-1 to W-18 (W-19 to W-36) contain a continuous DMM panel with PZ-22 sheet pile (10 feet +/-) cutoff wall is beneath the base slab for seepage and are pile supported. Base slab elevations are set to match finished grade so that the base slab generally has 2 to 4 feet of cover on the channel side. The land side of the T-Walls is backfilled with sand to EL 0.0 and clay backfill from EL 0.0 to EL 4.0. An 8-foot clear roadway is also proposed on top of the T-Wall to provide small utility vehicle access across from the U-Frame and Gated Diversion Structure to T-Wall and guide levee tie-ins in accordance with the MBSD Design Criteria. Side mounted LADOTD guard rails are also proposed on both sides of the roadway.

5.9.2 Design Features

5.9.2.1 Base Slab and Stem

The base slab width and thickness for monoliths W-1 thru W-6 is 30 feet and 7 feet, respectively. Monoliths W-7 thru W-12 base slab width and thickness is 30 feet and 5 feet, respectively. Monoliths W-13 thru W-16 base slab width and thickness is 15 feet and 5 feet, respectively. Monoliths W-17 and W-18 base slab width and thickness is 15 feet and 3 feet – 6 inches, respectively. The stem walls for monoliths W-1 thru W-12 are 2 feet – 6 inches thick at the top and thicken at a 1H:12V slope toward the land side. Monoliths W-13 thru W-18 have uniform walls with a thickness of 2 feet – 6 inches.

5.9.2.2 Cut-off Wall Sheet Pile

The cut-off wall in combination with DMM panel and sheet piling is provided to limit seepage to a certain tip elevation provided by AECOM (see Geotechnical Report), and the embedment criteria is specified in the Geotechnical Report Section 4. Cut-off sheet pile will be extended 30 feet beyond W-18/W-36 monoliths at the guide levee tie-in to tip elevation -45. The top of the sheet pile at these locations is set to match with the guide levee tie-in crown elevation.

5.9.3 Design Criteria and Loading Conditions

The load cases as described in the MBSD Design Criteria were used as a guide for creating the load cases evaluated in the analysis, which were considered most likely to control the design. All load cases and combinations evaluated are shown in table below. The following table shows the selected load cases. The hydraulic grade and design grades are from the MBSD Design Criteria **Appendix A**.

Table 5.9-1: Transition T-Wall Design Load Case Summary

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction (w/ Backfill) + Downdrag (no uplift)	- Dead - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall)	1.6 (D+EH+EV+ES+Ds)	Unusual
2	Construction (w/ Backfill) + Downdrag + Wind (no uplift)	- Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+ES+Ds+W)	Unusual
3a	Water @ Design SWL No Wind (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
3b	Water @ Design SWL No Wind (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
4a	Water @ Design SWL + Wind (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual
4b	Water @ Design SWL + Wind (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual
5a	Water @ Design SWL + Wave (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual
5b	Water @ Design SWL + Wave (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual
6a	Water @ Design SWL + Wind + Debris Impact (Impervious)	- R/S @ El. 9.35 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf) [1] - Debris Impact (0.5 kip/ft)	1.6 (D+EH+Hs+Hu+W+I)	Unusual

No.	Load Case Name	Description	Factored Load Combination	Load Category
6b	Water @ Design SWL + Wind + Debris Impact (Pervious)	- R/S @ El. 9.35 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (150 mph, min. 50 psf) [1] - Debris Impact (0.5 kip/ft)	1.6 (D+EH+Hs+Hu+W+I)	Unusual
7a	Design Flow 75K cfs (River @ 1,000,000 cfs) + Wave + Wind (Impervious)	- C/S @ El. 6.9 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wave load (50 yr future) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+Hw+W)	Unusual
7b	Design Flow 75K cfs (River @ 1,000,000 cfs) + Wave + Wind (Pervious)	- C/S @ El. 6.9 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wave load (50 yr future) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+Hw+W)	Unusual
8a	Reverse Head + Wind (Impervious)	- C/S @ El. -2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
8b	Reverse Head + Wind (Pervious)	- C/S @ El. -2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
9a	Water to TOW (Impervious) **Resiliency Check**	- C/S @ El. 15.85 - L/S @ El. 1.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
9b	Water to TOW (Pervious) **Resiliency Check**	- C/S @ El. 15.85 - L/S @ El. 1.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
10a	Low Flow Maintenance Operation (5000 cfs) (Impervious)	- C/S @ El. 2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Vehical Live Load	1.6 (D+EH+EV+Hs+Hu+L)	Unusual

No.	Load Case Name	Description	Factored Load Combination	Load Category
10b	Low Flow Maintenance Operation (5000 cfs) (Pervious)	- C/S @ El. 2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Vehical Live Load	1.6 (D+EH+EV+Hs+Hu+L)	Unusual

Notes: 1) No Unbalanced loads.
2) D= Dead Load, EVd= Down Drag, EH= Lateral Earth, EV= Vertical Earth, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind
3) Debris impact is 500 Lbs/LF at the surface of the water.

5.9.4 Analysis and Design Summaries

Analysis of the 3-dimensional Transition T-Walls is performed using a combination of hand calculations, excel spreadsheets and GROUP2016. The hand calculations, provided in the Appendix D, consider the self-weight of the T-Wall monolith, the water weight and pressure, the soil weight and pressure, and uplift forces. All other external forces due to wind, wave, debris impact are also considered with the appropriate loading combinations according to design criteria. There are no unbalanced loads considered during the analysis due to the proposed deep soil mixing method beneath the base slabs of tall T-Wall monoliths (W-1 to W-18 and W-19 to W-36). See section 9 of geo-tech report for details on Deep Mixing Method (DMM) used for eliminating unbalanced loads and settlement induced bending moments on piles.

The vertical force, lateral force and moments for each loading condition are individually calculated and are added together to create the loading combinations. The calculated load combinations are taken over the longitudinal pile spacing of 10 feet and are then entered in GROUP2019 to analyze the pile group and to determine the individual pile demands. Full GROUP models for each monolith will be presented in the next design phase. GROUP models have been created for all Transition T-Wall monoliths for the 60% design phase (W-1 thru W-5, W-6, W-7, W-8, W-9, W-10 & W-11, W-12, W-13 thru W-16 and W-17 & W-18). Soil layer parameters are provided by AECOM Geotechnical engineers and are used in GROUP2019 to analyze soil/pile interaction. All monoliths are analyzed as both fixed and pinned head conditions. It was determined that monoliths W-1 thru W-9 shall have a fixed head connection (due to too much deflection if pinned) and monoliths W-10 thru W-14 and W-17 thru W-18 shall be pinned. W-15 and W-16 are required to have plumb piles on the land side due to the interference of battered piles between monoliths W-14 and W-15. In order for the land side piles for W-15 and W-16 to be plumb, the monoliths must have a fixed head connection. Once the calculated load combinations, pile properties, pile fixity and soil parameters are entered into GROUP2019, the results are obtained to determine the design pile loads and deflection of the piles for each monolith. The calculation of the pile tip elevations is done by using the pile capacity curves for 24-inch diameter pipe piles, provided by AECOM geo-tech engineers. The pile design capacities are based on a factor of safety of 2, assuming static load tests will be conducted during construction, and applied overstress factors provided in the HSDRRS Design Guidelines. The deflection of the piles is also checked against the allowable deflection values stated in the MBSD Design Criteria.

Hand calculations were also performed to check the design of the stem wall and the base slab of the inverted T-Wall monolith in accordance with the MBSD Design Criteria. The stem and base slab of the T-Wall monoliths are sized by checking the shear and flexural strength of the concrete to determine the necessary thickness and rebar requirements. The stem of the T-Walls is designed using the pressure calculations of the Construction w/ Backfill, Construction + Wind, SWL and TOW load case. The base slab is designed using vertical loads (soil, water, surcharge, etc.) and pile loads obtained from GROUP for load cases: Construction w/ Backfill, Construction + Wind, SWL and TOW. The Stem and Base slab are designed following EM 1110-2-2104 (Design of Concrete Hydraulic Structures) design requirements.

5.10 Outfall Transition Feature (OTF)

Structural components of the OTF include flared and buttressed sheet pile guide walls and a cantilevered sheet pile end wall that serve to mitigate potential scour at the end of the outfall, transition flow from the Channel to the open marsh, and retain outfall riprap protection.

Braced sheet pile guide walls extend from the intersection of the MBSD Guide Levee and the Existing Back Levee out into the outfall basin. The northern guide wall extends approximately 310 feet into the basin; the south wall extends approximately 874 feet. The top of the sheet pile wall is EL 8.2; the top of the riprap channel protection adjacent to the wall varies with a minimum EL -2.0. Walls are primarily comprised of a PZC-13 sheet pile, HP14x73 waler, and battered HP14x73 piles spaced at either 6 feet or 10 feet on center. A typical cross-section of the braced OTF guide walls is shown in **Figure 5.10-1** below.

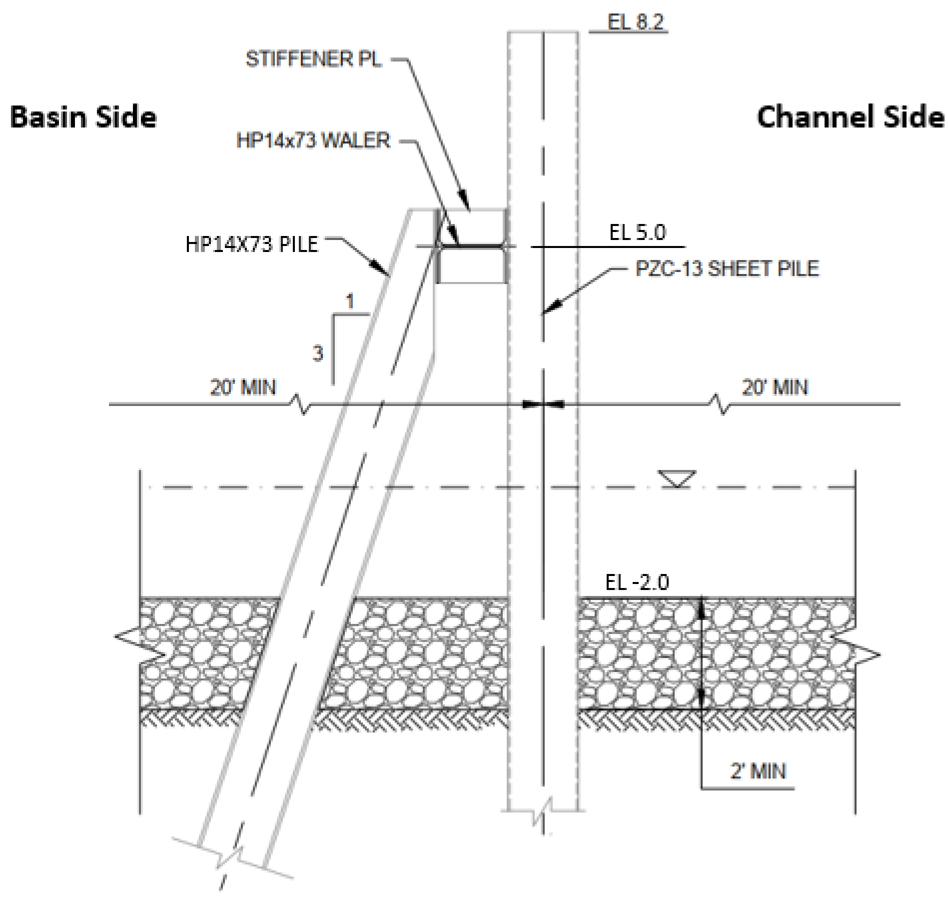


Figure 5.10-1: OTF Braced Guide Wall Typical Section

A cantilevered sheet pile end wall is located at the west (basin) end of the OTF. The length of this wall runs north-south, extending 100 feet past the northern and southern guide walls. The end wall extends approximately 1,442 feet north and 1,393 feet south of the project base line. The top of the wall is EL -5.0 with riprap channel protection at EL -4.0. Riprap protection extends 100 feet west of the end wall into the basin, to ensure a stable slope is maintained should a scour hole develop on the basin side of the end wall. The end wall detail is shown in **Figure 5.10-2** below.

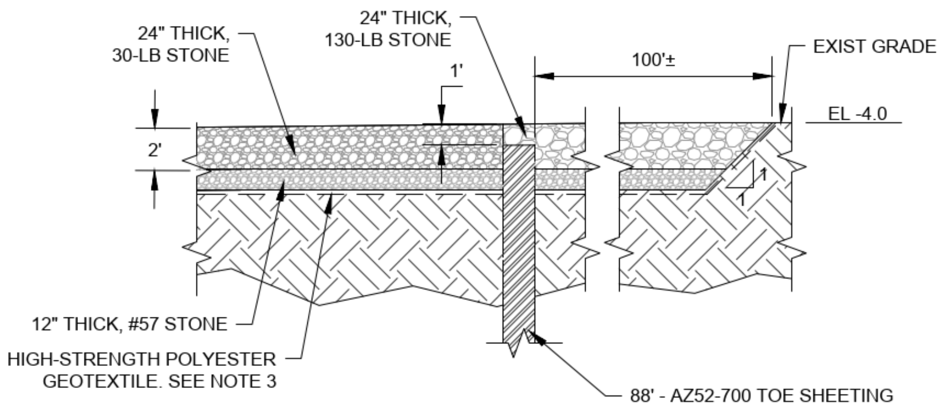


Figure 5.10-2 OTF Sheet Pile End Wall Detail

5.10.1 OTF Guide Walls

a. Shell Pipeline Crossing

A 20-inch diameter Shell petroleum pipeline crosses beneath both guide walls approximately 70 feet from the proposed sheet piles to be driven into the existing Back Levee. Shell will be relocating this pipe to a deeper elevation on the same alignment; when reinstalled the pipe shall be no shallower than EL -90.0 based on AECOM recommendations submitted in December 2020. HP and sheet pile installed within 25 feet of this pipeline will be limited to a tip elevation of EL -70.0 or higher, providing a 20 foot clearance between these features and the pipeline.

b. Design Criteria and Loading Conditions

The braced wall system is designed with a set of two load cases that represent worst case future conditions. For the next submission additional load cases may be added to this analysis. Current design load cases are as follows:

1) 50-Year Future Conditions, Diversion Not Operating, Sediment Accumulation to EL 3.0 in Channel
2) Load Case 1 Plus Tropical Storm/Hurricane Wave in Basin

Both cases assume water level to be EL 0.0 on both sides of the wall (walls extend into open basin, no head differential). Sediment has accumulated from EL -2.0 to EL 3.0 on the Channel Side of the wall, however no sedimentation is assumed on the Basin Side. Sediment is assumed to rise no higher than EL 3.0 because maintenance dredging activities will be prescribed when accumulation approaches this level.

Load Case 2 builds upon Load Case 1, adding a 240 psf uniform wave load to the Channel Side of the wall from EL 3.0 (top of sediment) to EL 8.2; wave force acts in the same direction as the uneven lateral sediment load. The design hydrostatic EL 0.0 used in both Load Cases is chosen to maximize the wave force. Higher water surface elevations will result in waves continuing over the top of the submerged wall, reducing the overall wave pressure. The current wave force derives from preliminary Basin Side hydraulics analysis and will be refined in the next submission as OTF scour protection and other geometry/features are finalized.

Steel components are checked for both geotechnical and structural capacity using pile capacity curves supplied by the geotechnical team and the Hydraulic Steel Structures requirements of ETL 1110-2-584, respectively. Because these structures are located in a brackish marsh and will rarely be inspected, the USACE performance factor α is set to 0.85. A general LRFD load combination of 1.2*D + 1.6*L is used for structural design of steel members and a service-level combination of loads is used for determining required geotechnical pile lengths.

c. Analysis and Results Summary

A 2-D SAP2000 model is created to represent a single battered HP pile and tributary width of sheet pile wall. Pile and sheet pile are assumed to be fixed at 20 times the pile diameter below the riprapped mudline EL -2.0. Geotechnical analysis suggests assuming a soil failure plane of EL -17.0 at the sheet pile; this is projected upward at a 45° angle to find the assumed failure plane at the battered pile. Pile capacity above these failure planes is neglected when determining required tip elevations. The sheet pile is assumed to act as a capacity pile in this frame structure, its length determined based on tension reactions from the design load cases. The frame model is loaded with the maximum anchor force provided by the geotechnical team (using LPile analysis) multiplied by an assumed pile spacing.

First, the section of wall spanning the Shell pipeline is examined to determine the H-pile spacing required to achieve tip elevations of EL -70.0 or higher. A 6 foot spacing and 3V:1H batter result in a required H-Pile tip elevation of EL -65.0 and sheet pile tip elevation of EL -50.0. This arrangement will be employed within a 25 foot buffer zone on either side of the pipeline.

Use of shorter piles at a closer spacing is not the most economical configuration for this wall, so a second typical arrangement is developed to reduce the number of battered piles. A spacing of 10 feet is used, which reduces the number of H-piles by approximately 40%, resulting in required tip elevations of EL -80.0 for the H-piles and EL -65.0 for the sheet pile.

5.10.2 OTF End wall

a. Design Criteria and Loading Conditions

The end wall is designed to protect the channel from scour that may occur as water and sediment exit the diversion structure. The scour depth at the end wall is predicted to be approximately 11 feet. Due to uncertainty in the modeling and potential consolidation of the marsh floor, the end wall is designed to protect against a scour depth of 18 feet, with a factor of safety of 1.5 applied to the required tip elevation. The resulting tip elevation is EL 93 as calculated by the geotechnical analysis. Additionally, the mudline deflection was evaluated for a scour depth of 23 feet to ensure stability in an extreme scour case. An AZ52-700 sheet pile was selected based on geotechnical design parameters. See Section 4.11 for a description of the geotechnical analysis. A structural analysis of the sheet pile wall is performed for the largest shear and moment calculated by the geotechnical analyses.

b. Analysis and Results Summary
Excel calculations were performed to determine the structural adequacy of the AZ52-700 sheet pile Allowable stress design was used in accordance with EM 1110-2-2504 Section 6-3. The allowable bending and shear stresses are 25ksi and 16.5ksi, respectively. The AZ52-700 sheet pile is adequate for the actual shear and moment forces determined from the geotechnical analysis. See Appendix D for structural calculations.

5.11 Inverted Siphon

5.11.1 General Description

The Inverted Siphon consists of the Intake Structure at the north diversion channel levee, the Inverted Siphon piping beneath the diversion channel, the Outlet Structure at the south diversion channel levee, tie-in T-Walls to transition between diversion levee and Intake and Outlet, and earth retaining walls (Wing Walls) to transition the drainage canal in/out of the siphon structures. The Inlet and Outlet Structures are pile supported, reinforced concrete, subdivided rectangular U-frame channels with partition walls subdividing the structures at between the Inverted Siphon pipes. Each Structure also includes floodwalls on either side to provide continuous flood protection with the Diversion Guide Levees. A section view of the combined inlet/wingwall structure is below.

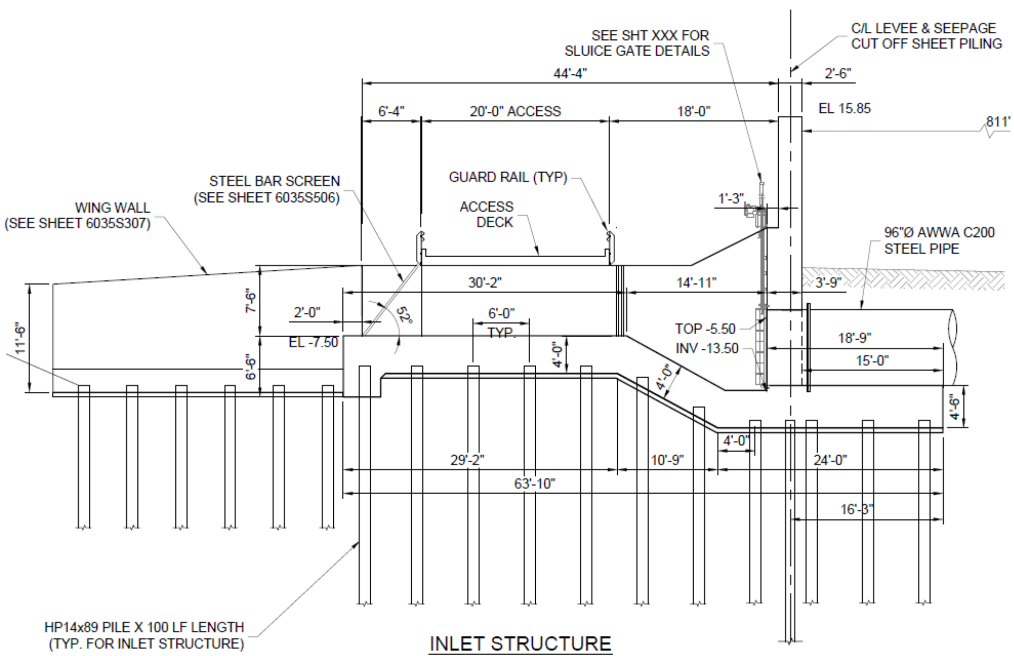


Figure 5.11-1: Combined Inlet/Wingwall Structure

The Inlet and Outlet Structures each feature an access deck 20 feet wide spanning the width of the structure to facilitate maintenance and access to the guide levee beyond the Siphon. Both structures feature pile-supported wing walls. The channels of both structures include slots for the placement of stop logs to allow isolation and dewatering of each pipe independently. The Inlet Structure also includes a sluice gate that provides positive cutoff from floodwaters protecting one polder should the other polder get overtopped or breached. The sluice gate is designed to resist a direct and reverse head (floodwaters coming from the south polder). As such, sluice gates are only needed on one side. The sluice gates also aide in the maintenance flushing of the siphons. A water hyacinth screen will be placed upstream of the Inlet Structure, and both structures will be equipped with bar screen to prevent debris and animals entering the Siphon.

Steel H-piles will be utilized for the Inlet and Outlet Structure, wing wall and floodwall foundations. The minimal differential fill and use of vertical piles eliminated the concern for levee instability and settlement effects on the inlet and outlet structures pile foundation and wing walls. DMM has been included at the Tie-In Floodwalls to eliminate both lateral loads from levee instability and downdrag due to settlement. Preload was considered, but the duration needed greatly impact the construction schedule. The Inlet and Outlet Structures have been designed as floodwalls and U-Frame channels, as applicable, in accordance with EM 1110-2-2007, Structural Design of Concrete Lined Channels, EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures and ACI 318-14, Building Code Requirements for Structural Concrete.

The Siphon piping will be backfilled using flowable backfill to allow void-free placement around the pipes, which will be placed 2 feet-0 inches apart to minimize excavation and backfill. This will provide a factor of safety against uplift of greater than 1.2 during construction and maintenance, and greater than 1.3 in the final condition.

The pile foundations have been designed in accordance with EM 1110-2-2906, Design of Pile Foundations, based on the allowable pile capacities provided by Eustis Engineering for each component Structure.

Pile loads were determined from CPGA analysis, pile length determined from service loads, and the base slabs of each structure analyzed from the factored pile reactions. Stem walls and partition walls were proportioned from direct application of factored loads. Calculations are attached in **Appendix D**. The piles were analyzed as both pinned and fixed pile connections.

5.11.2 Design Criteria

The Inlet and Outlet Structures were designed to include features and proportioned such that the following functional criteria are met:

1. The siphon piping shall be submerged by one foot or more at the drainage canal's low water elevation assuring full flow capacity and minimizing the infiltration of oxygen. Less oxygen greatly reduces corrosion potential. Th steel pipes will be lined on the inside surface with cement mortar and the exterior with a polyurethane liner.

2. Each pipe shall be capable of individual isolation and unwatering for maintenance.

3. Each pipe shall be capable of sealing at the culvert inlet (HSDRRS requirement) by installing sluice gates.

4. Debris is screened, collected, and removed upstream of pipes. Bar screens are also provided on the Outlet Structure in the event of a reverse flow condition through the siphon and to prevent the migration of debris, flora and fauna into the pipes.

5. Personnel and vehicular access bridges are provided for cleaning the bar screens and for operations and maintenance of the inlet sluice gates. Access bridges are designed for an H20 truck loading, ramp geometry accommodates an SU 30 vehicle.

6. Sluice gates shall be operated by commercial power that is run along the north guide levee. Portable actuators w/ generators will be used as backup power.

7. The overall operation of the Inverted Siphon will be passive. For the majority of operating conditions most or all of the pipes will remain open.

8. Operator safety and facility security are maintained with appropriate fencing and lighting.

9. A timber pile and floating boom system will be placed across the drainage canal approximately 300 feet upstream of the Inlet to screen floating vegetation such as water hyacinths.

5.11.3 Excavation

The method of excavation for construction of the Inverted Siphon will be sloped excavation. DMM shall be used to allow vertical cuts for the pipe place in the channel bottom. DMM is also used to allow the excavation and placement of the 96-inch pipe along the channel slope. The inlet and outlet structure excavation will be open cut using 1 on 9 side slopes. Excavation plans are further detailed in Section 13 of this DDR. Excavation slopes and soil stabilization measures have been determined by the geotechnical specialists on the CMAR team and reviewed by the Design Team.

5.11.4 Siphon Piping Design Criteria

The Inverted Siphon piping has been designed according to EM 1110-2-2902, using the number and diameter of pipes provided by the completed interior drainage model. Design criteria include the following:

1. The alignment shall maintain minimum clear cover between diversion channel bottom and top of pipe. A minimum of four (4) foot clearance is provided for the non-navigable sediment diversion channel.

2. Each individual pipe shall resist buoyant force when dewatered, during design flow of the diversion channel, by combination of pipe weight, tremie slab and backfill weight.

3. The pipe shall adequately resist soil pressures, hydrostatic pressures (positive and negative), and remain serviceable should differential settlement be induced after construction by surface features.

4. Siphon piping to be designed for FS against buoyancy of 1.2 for construction and maintenance and 1.3 for final operating condition.

5. The maximum allowable deflection per joint along the siphon pipes (exclusive of flexible fitting connections) is 1.0 degrees.

DMM will be employed under the sloped portions of the Siphon pipes to allow excavation and installation in the dry. The 1 on 4 channel slope is not stable in a dry channel. The geotechnical team determined it was not required beneath the horizontal berm portions, settlement was minimal. The Inverted Siphon piping is 96-inch steel pipe (AWWA C200, Welded Steel Pipe). The previous version of this document indicated fiberglass pipe would be used. Steel was selected instead based on the Design Team's finding of inadequate performance of large-diameter fiberglass pipe in several recent projects. The foundations for the Inverted Siphon piping have been designed to minimize differential settlement issues. Flexible joints are located at each foundation type transition, thus there are 6 flexible joints for each Inverted Siphon pipe. They are located at the faces of the Structures, top of bank and toe of bank.

5.11.5 Inverted Siphon Geometry

The Inverted Siphon profile transversely crosses the Diversion Channel and levee. The invert of the Inverted Siphon at the Inlet and Outlet Structures is -13.5. The Inverted Siphon pipe then descends beneath the bottom of the Diversion Channel at a 4H:1V slope. The invert of the Inverted Siphon piping below the Diversion Channel bottom is at EL -37.12.

Pile supported T-Walls with PZ 22 sheet pile cutoff tie in to the Inlet and Outlet Structure headwalls providing continuous flood protection as the levee slopes down on either side of the siphon. The access roads along the Guide Levees will cross each structure to provide vehicular and personnel access for structure maintenance.

5.11.6 Inlet and Outlet Structure Description

The Inlet and Outlet Structures are designed as U-Frame channels. The U-channel is subdivided between each Inverted Siphon pipe location. There are 20-degree wing walls at the Inlet Structure's entrance and a headwall at the end of the structure where the influent transfers to the Inverted Siphon piping. The Outlet Structure features 30-degree exit wing walls.

The headwall of each structure will also function as a floodwall tying into T-Walls on either side of each Structure. The length of each structure is 48'-10". The width of each structure is 82'-10". The height of both structures is 13 feet with top of U-Channel wall EL 0.0 and an invert EL -9.0 that transitions down to EL -13.5 at the Inverted Siphon piping.

The height of each headwall is 29.35 feet with the top of wall at EL 15.85. PZ 22sheet pile will be driven below the headwall to provide seepage cutoff. The Inlet Structure feeds six 96-inch Inverted Siphon pipes. Eight-foot sluice gates are provided for each Inverted Siphon pipe at the Inlet Structure headwall.

There is a 20-foot wide access deck spanning the front of each structure. Theses decks will be designed per ACI-318-14 -to support H-20 loading.

The Inlet and Outlet Structures are pile supported on steel H-piles with tension connectors as required for uplift, as required by analysis.

5.11.7 Inlet and Outlet Structure/T-Wall Load Combinations

The table below shows the load combinations to be investigated in the design of the Inlet and Outlet Structures; the structures have been proportioned at this stage on the basis of Cases 6, 10, 11, and 12.

Table 5.11-1: Siphon Intake and Discharge Structure Load Combinations

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction + Backfill + Downdrag (no uplift)	Dead (Structure Weight) Lateral soil up to El. -1.0 Temp. construction surcharge of 200 psf on backfill (F/S)	1.6 (D+EH+EV+L_s)	Unusual
2	Construction + Backfill (no uplift) plus Wind	Dead (Structure weight) Lateral soil up to El. -1.0 on F/S Wind on Floodwall	1.6 (D+EH+EV+W)	Unusual
3	Conveyance Operation at 1,000,000 cfs, (Pervious Cutoff)	Dead (Structure) F/S Channel Stage @ El. 4.0 P/S at El. -3.0 Hydrostatic Load Lateral soil up to El -3.0 Pervious Cutoff	2.2 (D+EH+EV+Hs+Hu)	Usual
4A/4B	Water @ Design SWL, No Wind (Pervious and Impervious Cutoff)	Dead (Structure weight) F/S SWL @ El. 9.35 P/S tailwater at El. -3.0 Lateral soil up to El. -3.0 Hydrostatic Load A. Impervious B. Pervious	2.2 (D+EH+EV+Hs+Hu)	Usual
5A/5B	Water @ Design SWL + Wind + Wave (Pervious and Impervious Cutoff)	Dead (Structure) F/S SWL @ El. 9.35 P/S tailwater at El. -3.0 Wind applied above wave height Wave load on T-Wall Lateral soil up to El. -3.0	1.6 (D+EH+EV+Hs+Hw+W+Hu)	Unusual
6	Water @ Design SWL + Wind + Wave+ Debris on T-Wall	Dead (Structure) F/S SWL @ El. 9.35 P/S tailwater at El. -3.0 50 psf wind pressure on wall above 9.35 Wave load on T-Wall Lateral soil up to El. -3.0 Debris Impact at El. 9.35	1.3 (D+EH+Hs+Hu+Hw+W+I)	Extreme
7	Water @ Design SWL + Wind + Wave (Pervious and Impervious Cutoff) at pumped down canal stage	Dead (Structure) F/S SWL @ El. 9.35 P/S tailwater at El. -6.0 Wind applied above wave height Wave load on T-Wall Lateral soil up to El. -3	1.3 (D+EH+EV+Hs+Hw+W+Hu)	Extreme
8	Water @ TOW = 15.85	Dead (Structure weight) F/S SWL @ El. 15.85 P/S tailwater at El. -3.0 Impervious cutoff (uplift, water @ El. -3.0) Lateral soil up to El. -1.0 on F/S	1.6 (D+EH+EV+Hs+Hu)	Unusual
9	Water @ TOW = 15.85	Dead (Structure weight) F/S SWL @ El. 15.85 P/S tailwater at El. -6.0 Impervious cutoff (uplift, water @ El. -3.0) Lateral soil up to El. -1.0 on F/S	1.3 (D+EH+EV+Hs+Hu)	Extreme

No.	Load Case Name	Description	Factored Load Combination	Load Category
10	Water to TOW @ El. 15.85 + Debris on Floodwall	Dead (Structure) F/S SWL @ El. 15.85 P/S tailwater at El. -3.0 Lateral soil up to El. -1.0 on F/S Debris at top of wall	1.2 (D+EH)+1.3 (Hs)+I	Extreme
11	Maintenance Dewatering 1 Bay using Stoplogs	Dead (Structure) F/S SWL @ El. 4.0 P/S Slab @ -13.5 Lateral soil up to El. +4.0 on outside of structure Lateral hydrostatic pressure from water @14.5 on FS	1.6 (D+EH+Hs)	Unusual
12	Maintenance Dewatering 1 Bay using Stoplogs Check Sidewalls	Dead (Structure) F/S SWL @ El. 4.0 P/S Slab @ -13.5 Lateral soil up to El. -3 on outside	1.6 (D+EH+Hs)	Unusual
13	Maintenance Dewatering (Check Internal Walls)	Hydrostatic pressure on the wall up to El. 4.0 on one side w/ no water on the other side, slab EL. -9.0	1.6 Hs	Unusual

The table below shows the load combinations investigated in the design of the Tie-In T-Walls and Retaining Wing Wall Structures. All cases are investigated for the T-Walls, only cases 1, 4, and 7 were investigated for the Retaining Wing Walls:

Table 5.11-2: Siphon Tie-In Floodwall Load Combinations

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction + Backfill + Downdrag (no uplift)	- Dead (Structure Weight) - Lateral soil up to El. -3.0 - Temp. construction surcharge of 200 psf on backfill	1.6 (D+EH+EV+L_s)	Unusual
2	Construction + Downdrag +Backfill (no uplift) plus Wind	- Dead (Structure weight) - Lateral soil up to El. -1.0 on F/S, -3.0 on P/S - Wind	1.6 (D+EH+EV+W)	Unusual
3	Conveyance Operation at 1,000,000 cfs, (Pervious Cutoff)	- Dead (Structure) - F/S Channel Stage @ El. 4.0 - P/S at El. -3.0 - Hydrostatic Load - Lateral soil up to El -3.0	2.2 (D+EH+EV+Hs+Hu)	Usual
4A/4B	Water @ Design SWL, No Wind (Pervious and Impervious Cutoff)	- Dead (Structure weight) - F/S SWL @ El. 9.35 - P/S tailwater at El. -3.0 - Lateral soil up to El. -3.0 - Hydrostatic Load - A. Impervious B. Pervious	2.2 (D+EH+EV+Hs+Hu)	Usual
5A/5B	Water @ Design SWL + Wind + Wave (Pervious and Impervious Cutoff)	- Dead (Structure) - F/S SWL @ El. 9.35 - P/S tailwater at El. -3.0 - Wind applied above wave height - Wave load on T-Wall - Lateral soil up to El. 4.0 on F/S, -1.0 on P/S	1.6 (D+EH+EV+Hs+Hw+W+Hu)	Unusual
No.	Load Case Name	Description	Factored Load Combination	Load Category

6	Water @ Design SWL + Wind + Wave+ Debris on T-Wall	- Dead (Structure) - F/S SWL @ El. 9.35 - P/S tailwater at El. 0.0 - 50 psf wind pressure on wall above 9.35 - Wave load on T-Wall - Lateral soil up to El. -3.0 - Debris Impact at El. 9.35	1.3 (D+EH+Hs+Hu+Hw+W+I)	Extreme
7	Water @ TOW = 15.85, Pervious Cutoff (Resiliency Check)	- Dead (Structure weight) - F/S SWL @ El. 15.85 - P/S tailwater at El. -3.0 - Impervious cutoff - Lateral soil up to El. -3.0	1.6 (D+EH+ Hs+Hu+EH)	Unusual
8	Water to TOW @ El. 15.85 + Debris on T-Wall (Resiliency Check)	- Dead (Structure) - F/S SWL @ El. 15.85 - P/S tailwater at El. -3.0 - Lateral soil up to El. -3.0 - Debris at top of wall, 15.85	1.2 (D+EH)+1.3 (Hs)+I	Extreme

5.11.8 Gates and Trash Racks

The Inlet and Outlet Structures each have six (6) stoplog closures at the entrance, one for each channel, to facilitate maintenance. At the rear headwall the Inlet Structure has six (6) 8-foot sluice gates, one for each Inverted Siphon pipe. All sluice gates will be rising stem, cast iron and meet AWWA C560. The sluice gates will have flush bottom closures to eliminate the recess required for a standard gate closure which could prevent the gate from being fully closed should debris collect in the recess and will be operated with portable actuators.

Both structures also feature steel bar screen trash racks at the entrance to the structure preventing debris and trash in the canal from entering the Inverted Siphon piping. A floating vegetation arrestor will be installed approximately 300 feet upstream of the Inlet Structure. It is worth noting, however, that even should the arrestor fail and allow typical floating vegetation to accumulate at the bar screen, the canal flow volume will still pass beneath and enter the Inverted Siphon which is designed to remain submerged even with the canal at low water level.

5.12 Drainage Structure

5.12.1 General Description

The drainage structure is part of the CPRA Mid Barataria Sediment Diversion project and will become part of the New Orleans to Venice (NOV) hurricane risk reduction project once installed in the NOV-NF-W-05a.1. The NOV-NF-W-05a.1 levee has 4 hydraulic reaches, NOV 5a – NOV 5d with the drainage structure being in hydraulic reach NOV 5c, adjacent to the sediment diversion channel.

The proposed sediment diversion channel will flow from the Mississippi River to Barataria Bay and will bisect the proposed USACE NOV 5a.1 levee which will be constructed inland from the existing non-federal back levee and the existing drainage ditch which runs parallel to the existing back levee creating an area that will be impounded with water. The drainage structure will allow water to drain out of the impounded area through the culvert to the interior side of the NOV 5a.1 levee to the existing Timber Canal, through the inverted siphon, and onto the Wilkinson Pump Station before being pumped out into Barataria Bay.

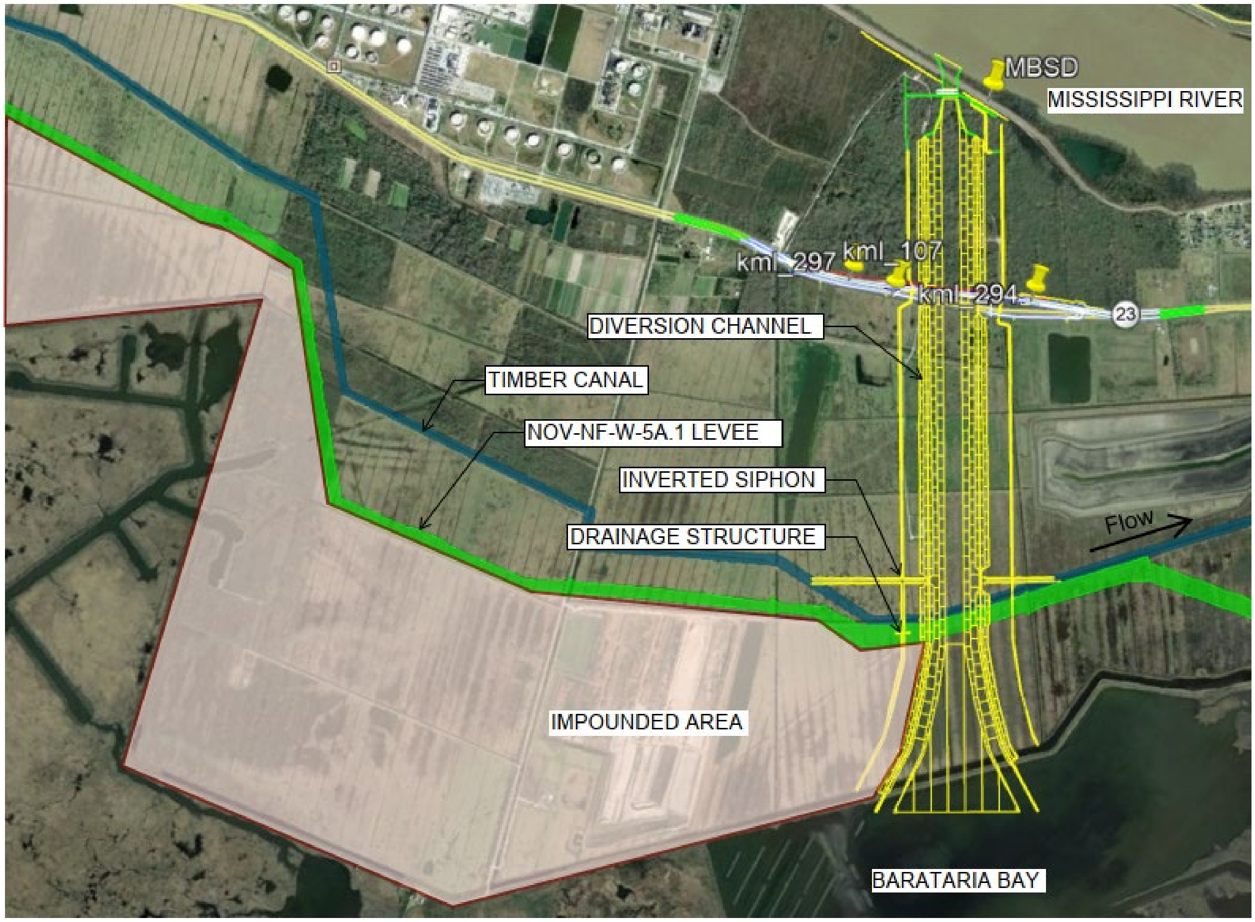


Figure 5.12-1: Impounded Area

5.12.2 Operation

The drainage structure is a pile founded reinforced concrete gated structure that will tie into the NOV 5a.1 levee with inverted T-walls to provide continuous flood protection up to a 50-Year storm event with top of wall being EL 15.85. When the threat of an approaching hurricane becomes apparent the sluice gates will be closed in both culvert bays to protect against storm surge. The sluice gate operators will be mounted at the top of the drainage structure gate well and will be accessible from the protected side access bridge via fixed ladder.

The culverts will be protected from debris by employing fixed metal bar screens at the opening of the culvert bays. The bar screens will be manually cleaned by use of an excavator via the access bridges on the drainage structure. A bar screen and access bridge will be provided on both sides of the drainage structure with access ramps leading down from the proposed NOV 5a.1 levee.

In the event of a heavy rain event water levels in the interior drainage canals could become higher than that of the impounded side therefore back flowing to the impounded area resulting in unwanted flooding. Flap gates will be employed at the interior side of both culvert bays to prevent back flow to the impounded area. The flap gates will provide one-way flow through the drainage structure. The trash racks on the interior side of the drainage structure will prevent large debris from jamming the flap gate which could prevent full closure resulting in back flow.

Maintenance bulkhead panels will be installed in recessed slots at each end of the drainage structure to perform dewatering of the drainage structure for routine inspection and maintenance. Dewatering of each bay independently will be possible by providing separate slots in each bay. The bulkhead panels will be stored on-site on a dedicated pad separate from the structure. The bulkhead panels will be lifted and lowered in the slots from the access bridge by truck mounted crane.

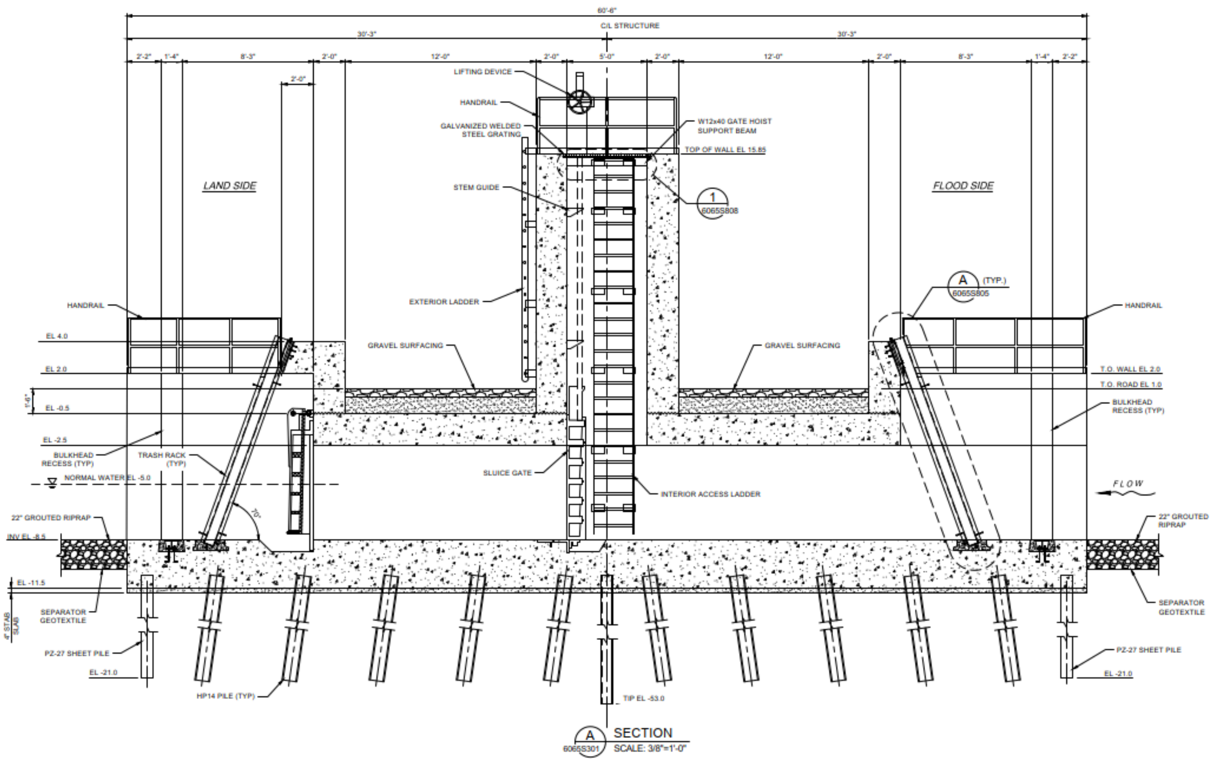


Figure 5.12-2: Drainage Structure Cross Section

5.12.3 Design Criteria

The drainage structure has been designed in accordance with the following engineering standards:
EM 1110-2-2906, Design of Pile Foundations
EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
EM 1110-2-2902 Conduits, Culverts, and Pipes
ACI 318-14, Building Code Requirements for Structural Concrete
AISC-15, American Steel Institute of Steel Construction
ASCE 7-16, Minimum Design Loads for Buildings and Other Structures

The following parameters were used in the design of the drainage structure and associated inverted tie-in T-walls:

1. Drainage structure invert shall be EL -9.0, matching that of the drainage canals leading to the Timber Canal.
2. Top of drainage structure and top of tie-in T-wall shall be designed and built to 50-Year design grade EL 15.85. Water to top of wall is considered an Extreme design case.
3. 500 plf debris impact shall be applied at water surface. Vessel impact not required for this reach.

4. Culvert cross sectional area shall be equal to or exceed that which has been developed from the hydraulic design model of qty 4 – 48 inch diameter pipes. The culvert consists of 2 – 84 inch x 72 inch bays providing 84 square feet of drainage area, greater than the required hydraulic design cross sectional area.
5. Sluice gates shall be provided for storm surge protection.
6. Bar screens shall be provided to protect the culvert from floating debris.
7. Flap gates shall be provided on the protected side to prevent back flow to the impounded area.
8. Bulkhead panels shall be installed in recessed slots for performing dewatering of the structure for routine inspection and maintenance.
9. The drainage structure shall resist lateral and overturning forces due to hurricane wind and storm surge. The structure shall also resist buoyant forces when fully dewatered using tension connectors on the pile heads. Buoyant forces shall be checked for both pervious and impervious seepage cutoff walls.
10. Normal water level in the drainage canal is EL -6.0, top of canal bank is EL 3.0, and natural ground elevation is EL -3.0. With the structure being built in the NOV levee with soil up to EL 1.0 it is assumed for the structure's design that ground water level is EL 0.0 to be consistent with other structures in the project.

Table 5.12-1: Drainage Structure Load Combinations

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction – No Backfill	- Dead (Self Weight) - Temporary Constructuion Surcharge - Wind (P/S) - No Backfill, No Uplift	1.6 (D+W+Ls)	Unusual
2	Construction with Backfill	- Dead (Self Weight) - Temporary Constructuion Surcharge - With Backfill - No Uplift	1.6 (D+EH+EV+Ls)	Unusual
3	**Gates Open** Water at Top of Culvert (Impervious)	- Dead (self wt.) - Gates Open - Water at Top of Culvert, EL 0.0 - Groundwater EL -5.0 (normal water elev. in canal) - Maintenance vehicle on bridge, HS20 - Impervious sheet pile cutoff	2.2 (D+Lv+EH+EV+Hs+Hu)	Usual
4	**Maint. Dewatering 1 Bay Closed** Water at Top of Culvert (Pervious)	- Dead (self wt.) - 1 Gate open, 1 Bay dewaterd using stop logs - Water at Top of Culvert, EL 0.0 - Groundwater EL -5.0 (normal water elev. in canal) - Maintenance vehicle on bridge, 640 psf Lane Load - Pervious sheet pile cutoff	1.6 (D+Lv+EH+EV+Hs+Hu)	Unusual
5	**Fully Dewatered** Water at Top of Culvert (Pervious)	- Dead (self wt.) - 2 bays dewaterd using stoplogs - Water at Top of Culvert, EL 0.0 - Groundwater EL -5.0 (normal water elev. in canal) Maintenance vehicle on bridge, 640 psf Lane Load - Pervious sheet pile cutoff	1.6 (D+Lv+EH+EV+Hs+Hu)	Unusual
No.	Load Case Name	Description	Factored Load Combination	Load Category

6	Water at Design SWL (Impervious and Pervious)	- Dead (self wt.) - Gates Closed - SWL @ EL 9.35 - P/S Groundwater EL 0.0 - Impervious and pervious sheet pile cutoff	2.2 (D+EH+EV+Hs+Hu)	Usual
7	Water at Design SWL plus Wave and Wind, (Impervious and Pervious)	- Dead (self wt.) - Gates Closed - SWL @ EL 9.35 - P/S Groundwater EL 0.0 - Impervious and pervious sheet pile cutoff - Wave load on wall - Wind Load above SWL	1.6 (D+EH+EV+Hs+Hw+Hu+W)	Unusual
8	Water at Design SWL plus Wave, Wind and Debris (Impervious and Pervious)	- Dead (self wt.) - Gates Closed - SWL @ EL 9.35 - P/S Groundwater EL 0.0 - Impervious and pervious sheet pile cutoff - Wave load on wall - Wind Load above SWL - 500 lb/ft debris load on wall	1.3 (D+EH+EV+Hs+Hw+Hu+W+Id)	Extreme
9	**Resiliency Check** Water to TOW EL15.85, Debris (Impervious and Pervious)	- Dead (self wt.) - Gates Closed - TOW SWL @ EL 15.85 - P/S Groundwater EL 0.0 - Impervious and pervious sheet pile cutoff - 500 lb/ft debris loadon wall	1.3 (D+EH+EV+Hs+Hu+Id)	Extreme
10	**Reverse Head** NOV 5a Overtopped, P/S Flooded EL 8.4 (Pervious)	- Dead (self wt.) - Gates Closed - NOV 5a back levee overtopped - P/S Flooded to EL 8.4 - F/S Groundwater EL 0.0 - Pervious sheet pile cutoff	1.3 (D+EH+EV+Hs+Hu)	Extreme
10	EQ (OBE)	- Dead (self wt.) - Gates Open - EL -5.0 (normal water elev. in canal) - Operating Basis Earthequake (OBE)	1.5 (D+EH+EV+EQ)	Unusual
11	EQ (MBE)	- Dead (self wt.) - Gates Open - With Backfill - Maximum Design Earthequake (MDE)	1.0 (D+EH+EV) +1.25 (EQ)	Extreme

1. D = Dead Load, Ls = Construction Surcharge, Lv = Vehicular Live Load, EH = Lateral Earth Load, EV vertical Earth Load, Hs = Hydrostatic, Hd = Wave, Hu = Hydrostatic Uplift, Id= Debris impact, W = Wind, OBE/MDE = Seismic
2. Load combinations for the design of the tie-in T-walls are similar.

5.12.4 Drainage Structure Design

The design of the drainage structure is similar to that proposed by USACE to be installed in the NOV-NF-W-05a.1 levee near the Wilkinson Pump Station. A 3D finite element model of the drainage structure was produced using SAP2000 to develop the base reactions using service loads. These base reactions were input into CPGA to analyze the proposed pile foundation. The 3D finite element model will also be used to develop maximum shear forces and bending moments in the structure from which the concrete and reinforcing will be designed.

The tie-in T-wall design employed conventional spreadsheet calculation and structural design using Microsoft Excel. The results of the design of both the drainage structure and the tie-in T-walls will be provided in a separate design report.

5.13 T-Walls under Hwy 23 Bridge

5.13.1 General Description

The Hwy 23 bridge is located approximately at Station 65+00 of the conveyance channel alignment and is approximately 2,250 feet west of the guide levee tie-in for the transition T-Wall. To protect from hurricane surge, T-Walls are proposed below the bridge instead of earthen levee. The T-Walls are located on both the north and south sides of the Conveyance Channel. The proposed T-Wall will connect to the guide levee tie-ins. South side T-Walls are in a straight alignment with levee tie-in, but north side T-Wall alignment is offset 15 feet towards the channel side to avoid the conflict between T-wall's batter piles (land side) and Hwy 23 Bridge bent piles. The Conveyance Channel T-Walls are located at a potential in-the dry construction zone. There is no need for braced construction to construct these T-Walls. The top of the base slab for the all Conveyance Channel T-Walls is at EL 3.0. Levee tie-ins for north and south walls are without settlement by using wicks drain and pre-loading. See analysis and soil report by Eustis engineering.

5.13.2 Design Features

5.13.2.1 Base Slab and Stem

The base slab for the conveyance channel T-Walls is at EL 3.0 and the T-Wall monoliths extends 430 feet from the east guide levee tie-in to the west guide levee tie-in on the north and south side of the Conveyance Channel. There are sixteen identical T-Walls on both the north side and south sides of the Conveyance Channel. For this phase the top of slab for all conveyance channel T-Walls is at EL 3 and top of wall is EL 15.85. The T-Walls are back-filled with clay to EL 4.0 on both sides of the stem wall. Wall stem height is 12 feet – 10 inches and is the same for all monoliths. Base slab width and thickness is 15 feet and 3 feet – 6 inches, respectively. A continuous cut-off sheet pile curtain wall is embedded 9 inches into the base slabs. All monoliths are pile supported with pile tips set to mitigate differential settlement among monoliths. Settlement calculations are not performed in the 60% design phase. See Appendix D for pile layout, tip elevations, sizes and other design features. Batter piles are battered at 1:12 slope to avoid the interference with bridge batter piles.

5.13.2.2 Cut-off Wall Sheet Pile

The cut-off wall of sheet piling is provided to limit seepage to a tip elevation at EL -30.0, and the embedment criteria is specified in the Geotechnical Report Section 4. Cutoff sheet pile will extend via a sheet pile transition wall into the levee embankment. Cut-off sheet pile will be extended 30 feet beyond the T-Wall at the guide levee tie-in for the T-Wall monoliths. The top of the sheet pile at these locations is set to match with the guide levee tie-in crown elevation.

5.13.3 Design Criteria and Loading Conditions

The load cases as described in the MBSD Design Criteria (**Appendix A**) are used as a guide for creating the load cases evaluated in the analysis, which were considered most likely to control the design. Engineering judgment is used in selecting the load cases by comparing the magnitude of the applied loads and the allowable overstress. Only the basic load cases are evaluated. The basic load cases selected for the analysis are as stated in the table below.

The analysis evaluated the pervious and impervious cut-off wall uplift conditions. The following table shows the selected load cases. The hydraulic grade and design grades are from the MBSD Design Criteria **Appendix A**.

Table 5.13-1: Hwy 23 T-Wall Design Load Case Summary

No.	Load Case Name	Description	Factored Load Combination	Load Category
1	Construction (w/ Backfill) + Downdrag (no uplift)	- Dead - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall)	1.6 (D+EH+EV+ES+Ds)	Unusual
2	Construction (w/ Backfill) + Downdrag + Wind (no uplift)	- Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Temporary construction surcharge of 200 psf - Downdrag (on Wall) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+ES+Ds+W)	Unusual
3a	Water @ Design SWL No Wind (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading Impervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
3b	Water @ Design SWL No Wind (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading Pervious cutoff	2.2 (D+EH+ EV+Hs+Hu)	Usual
4a	Water @ Design SWL + Wind (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual
4b	Water @ Design SWL + Wind (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+W)	Unusual

No.	Load Case Name	Description	Factored Load Combination	Load Category
5a	Water @ Design SWL + Wave (Impervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual
5b	Water @ Design SWL + Wave (Pervious)	- C/S @ El. 9.35 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wave load (50 yr future)	1.6 (D+EH+EV+Hs+Hu+Hw)	Unusual
6a	Water @ Design SWL + Wind + Debris Impact (Impervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf) [1] - Debris Impact (0.5 kip/ft)	1.6 (D+EH+Hs+Hu+W+I)	Unusual
6b	Water @ Design SWL + Wind + Debris Impact (Pervious)	- R/S @ El. 14.85 - B/S @ El. 0.0 - Dead (including bridge wt) - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wind (150 mph, min. 50 psf) [1] - Debris Impact (0.5 kip/ft)	1.6 (D+EH+Hs+Hu+W+I)	Unusual
7a	Design Flow 75K cfs (River @ 1,000,000 cfs) + Wave + Wind (Impervious)	- C/S @ El. 6.9 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wave load (50 yr future) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+Hw+W)	Unusual

No.	Load Case Name	Description	Factored Load Combination	Load Category
7b	Design Flow 75K cfs (River @ 1,000,000 cfs) + Wave + Wind (Pervious)	- C/S @ El. 6.9 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Wave load (50 yr future) - Wind (150 mph, min. 50 psf)	1.6 (D+EH+EV+Hs+Hu+Hw+W)	Unusual
8a	Reverse Head + Wind (Impervious)	- C/S @ El. -2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
8b	Reverse Head + Wind (Pervious)	- C/S @ El. -2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Wind (150 mph, min. 50 psf)	1.6 (D+EH+ EV+Hs+Hu+W)	Unusual
9a	Water to TOW (Impervious) **Resiliency Check**	- C/S @ El. 15.85 - L/S @ El. 1.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
9b	Water to TOW (Pervious) **Resiliency Check**	- C/S @ El. 15.85 - L/S @ El. 1.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff	1.6 (D+EH+EV+Hs+Hu)	Unusual
10a	Low Flow Maintenance Operation (5000 cfs) (Impervious)	- C/S @ El. 2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Impervious cutoff - Vehical Live Load	1.6 (D+EH+EV+Hs+Hu+L)	Unusual
10b	Low Flow Maintenance Operation (5000 cfs) (Pervious)	- C/S @ El. 2.0 - L/S @ El. 0.0 - Dead - Lateral and Vertical Earth Pressure - Hydrostatic Loading - Pervious cutoff - Vehical Live Load	1.6 (D+EH+EV+Hs+Hu+L)	Unusual

Notes: 1) No Unbalanced loads.
2) Debris impact is 500 Lbs/LF at the surface of the water.

3) D= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

5.13.4 Analysis and Design Summaries

Analysis of the 3-dimensional structure is performed using a combination of hand calculations, excel spreadsheets and GROUP2016. The hand calculations, provided in the **Appendix D**, consider the self-weight of the T-Wall monolith, the water weight and pressure, the soil weight and pressure, and uplift forces. There are no unbalanced loads shown in the geotechnical stability analysis at EL 3.0.

The vertical, lateral and moment forces for each are individually calculated and are added together to create the load combinations. The load combinations are then entered in GROUP2016 to analyze the pile group and to determine the individual pile demands. Soil layers and parameters entered in GROUP2016 are provided by Eustis Engineering. Once the calculated loads, pile properties, and soil parameters are entered into GROUP2016, the results are used to determine the capacities and deflection of the piles. The calculation of the pile capacities is done by using the pile capacity curves for HP14 piles, provided by Eustis Engineering. The pile design capacities are determined based on a factor of safety of 2, assuming static load tests will be conducted during construction. The deflection of the piles is also checked by using the allowable deflection values stated in the HSDRRS Design Guidelines.

Hand calculations were also performed to check the design of the stem wall and the base slab of the inverted T-Wall monolith in accordance with the MBSD Design Criteria. The stem and base slab of the T-Wall monoliths are sized by checking only the shear strength of the concrete to determine the necessary thickness. Shear is checked using EM 1110-2-2104 (Design of Concrete Hydraulic Structures). Moment calculations for the stem and base slab were performed and steel reinforcement was chosen based on requirements. The stem of the T-Walls is designed using the pressure calculations of the TOW load case. The base slab is designed by analyzing the weight of slab, weight of soil, weight of water, uplift and the pile reactions from the governing pile load from GROUP2016. Factored concrete design loads shown in Table 5.13-1 are used to confirm the adequacy of the stem wall and slab thickness.

The HWY 23 T-Wall's stem is 2 feet – 6 inches thick and 12 feet – 10 inches tall. The base slab has a top slab elevation of EL 3.0 and is 3 feet – 6 inches thick. The HP14x89 steel piles spaced at 7.5 feet o.c. have a tip EL -110.0 and -65.0 for batter and plumb pile respectively. The piles are embedded into the base slab 14 inches (one pile depth) to create a fixed connection to reduce the deflection. See **Appendix D** for pile layout, tip elevations, sizes and other design features.

6. CIVIL DESIGN

6.1 General

This section summarizes the Civil Designs included in the 60% Submittal for both the three-component diversion system and the secondary features. Applicable civil design criteria and references are listed in **Appendix A.**

6.2 Site Work and Grading

In addition to the three-component diversion system, site work and grading will be performed throughout the site to maintain access to the ancillary buildings, inverted drainage siphon, and back levee. Access roads consist mainly of a stone aggregate surfacing with a compacted granular sub-base and geotextile fabric or geogrid where appropriate. Positive drainage will be maintained with surface cross slopes between 1% to 2.5% and side slopes at 3H:1V. Runoff will be collected via drain inlets, pipes, and swales, then routed to nearby drainage ditches for transport to the Timber Canal.

The ancillary buildings area, located on the south side of the MBSD between Hwy 23 and the Mississippi River, requires the installation of new utilities such as water, electric, communications, and sewer. Electric and water lines will also be installed to provide service to the inverted drainage siphon, which will be located where the Timber Canal crosses the MBSD Conveyance Channel. Fencing will be installed along the perimeter of the MBSD right-of-way, with gates located at several locations to provide access to authorized personnel during operations and maintenance.

6.3 Conveyance Channel and Levees

The Conveyance Channel is an open channel with a bottom width of 300 feet and 4H:1V side slopes. Both the channel bottom and side slopes will be armored. On the north and south sides of the Conveyance Channel, earthen levees at design grade EL 15.85 act as both guide levees and hurricane protection levees between the MBSD headworks and the USACE NOV-NF-05a.1 levee. The levees will have a 10-foot wide gravel access road at the crown, with side slopes at 4H:1V, and they will be constructed with a sequence of overbuild and wick drains to mitigate settlement. Armoring will extend from the conveyance channel side slope and up a portion of the levee side slope. The non-armored portion of the levee will be covered by reinforced turf.

Where the levee alignments intersect LA Hwy 23 and the inverted drainage siphon, floodwalls will be constructed. Transitions between the earthen levee sections and floodwalls will be designed per the standard USACE details which include sheet pile tie-ins and either concrete or riprap slope protection.

On the basin side of the NOV-NF-05a.1 levee, the Conveyance Channel levees will only serve as guide levees since they are outside of the hurricane protection system.

6.4 Back Levee

Approximately 2,500 feet of the existing back levee will be removed for the construction of the MBSD outfall. The MBSD guide levees will tie-in with sheet pile to portions of that existing back levee that will remain in place on either side of the MBSD.

6.5 Outfall Transition Feature

The outfall transition feature begins at baseline Station 119+00 and is 300 feet wide at EL -25. It then transitions 2,667 feet to EL -4.0 with an overall width of 2,580 feet. The end of the outfall consists of an 88-foot long AZ 52-700 vertical sheet pile toe wall. Riprap armor extends 100 feet beyond the toe wall. The outfall levee flares 9 degrees from Station 119+00 to Station 127+50, 14 degrees from Station 127+50 to Station 136+20, and 28 degrees from Station 136+20 to the end.

The outfall channel bottom consists of 24-inch thick 30-lb stone over 12 inches of #57 stone, the channel slopes consist of 24-inch thick 55-lb stone over 12 inches of #57 stone from EL -25 to EL-4.0, and the levee slopes consist of 24-inch thick 130-lb stone over 12 inches of #57 stone from EL-4.0 to EL 8.2.

The outfall to back levee transition consists of embedded PZC-13 both into the guide levee and into the existing back levee. From the back levee tie-in out to the basin, the sheet pile wall consists of a braced sheet pile wall as described in **Section 5.10**.

6.6 Armoring

6.6.1 Introduction

As covered in the Design Criteria, the Basis of Design Report (BODR), and the Conveyance Channel Revetment Study appended to the BODR, the most feasible revetment material selected to protect the wetted earthen surface of the diversion is rock riprap. The following sections describe how the riprap protection system design for the Intake, Conveyance Channel, the Transition section, and the Outfall was further developed during the 60% Design phase. The development process included both numerical modeling of the hydraulics as well as scaled physical modeling of the diversion features to determine the stability of various riprap sizes under select scenarios.

Table 6.6-1 provides a summary of the MBSD armoring requirements, listed by reach/component.

Table 6.6-1: Summary of Armoring Requirements by Reach/Component

Diversion Reach / Component		Stone Gradation	Stone Thickness	Remarks
Mississippi River	Intake	130 lb	5 ft	Area beyond cofferdam in river, deposited directly on soil. Smaller stone required by calculation, but 130 lb selected due to ease of constructability and equivalent cost. 5 ft thickness due to placement in river, without filter or foundation.
	Intake (Over ex. Revetment)	130 lb	3 ft	Area is beyond cofferdam in river, deposited on existing articulated concrete mat revetment. Smaller stone required by calculation, but 130 lb selected due to ease of constructability and equivalent cost. 3 ft thickness due to placement in river.
	Levee	130 lb	2 ft	Smaller stone required by calculation, but 130 lb selected due to ease of constructability and equivalent cost. Only 2 ft thickness required due to placement in-the-dry.

Diversion Reach / Component		Stone Gradation	Stone Thickness	Remarks
Transition	First 100-ft	130 lb	2 ft	Numerical modeling showed very high velocities immediately downstream of U-frame. Physical model showed no movement with 130 lb stone. Further analysis required to evaluate need for extension of concrete slab or if 130 lb stone will be sufficient.
	Beyond 100-ft	30 lb	1.5 ft	Rapid reduction in velocities beyond first 100 ft downstream of U-Frame enable smaller stone to be used..
Conveyance Channel	Basic Section	10 lb	1 ft	Low velocities coupled with significant depth of flow require only small armoring stone.
	Highway 23 Piers	10 lb	1 ft	Standard bridge pier foundation design does not consider effect of armoring. Piles to be designed for 20 ft of scour.
Outfall	Base of Hurricane Levee	130 lb	2 ft	Storm surge effects dominate design for 1000-ft into the channel. Alternative geotechnical methods to be investigated for placement of armoring in very soft marsh soil.
	Base of Channel Side Slopes	55 lb	2 ft	Storm surge effects dominate design for 1000-ft into the channel. Alternative geotechnical methods to be investigated for placement of armoring in very soft marsh soil.
	Bottom of Channel	30 lb	2 ft	Storm surge effects dominate design for 1000-ft into the channel. Alternative geotechnical methods to be investigated for placement of armoring in very soft marsh soil.

6.6.2 Intake Armoring

6.6.2.1 General Description

The intake armoring consists of riprap and filter layers located upstream of the first protection cells, extended downstream of the farthest reach of intake u-frame cofferdam, and from the Mississippi River Levee (MRL) into the river beyond the extent of the existing Myrtle Grove revetment. Approximate area covered by intake armoring riprap is 8.1 acres (in-the-wet construction) and 7.0 acres (in-the-dry construction). Additionally, the existing Portland cement concrete (PCC) slope paving will be replaced on the MRL where disrupted by construction. Purpose of the intake armoring is to prevent erosion as follows:

- Intake channel scour due to increased water velocity during diversion operation
- River bank erosion due to normal river flow at disruption of existing Myrtle Grove revetment

Armoring analysis and proportioning for the intake channel considers the main intake channel, constructed in the dry, to EL -25; and the adjacent river banks/bottom influenced by diversion flows, where armoring is constructed in-the-wet. EM 1110-2-1601 was selected from the various approaches in the Design Criteria for relative conservatism of predicted results and for familiarity of USACE New Orleans District reviewers with the EM method within their waterways.

6.6.2.2 Design Features

Intake armoring limits were proportioned to protect the MBSD intake channel inscribed in the river bank, and to transition the riprap armoring back into the existing USACE articulated concrete mat (ACM) revetment upstream and downstream of the intake. Riprap overlap from point of ACM disruption is prescribed by USACE District practice at 80 feet minimum.

Construction methodology differs between work inside of the cofferdam and outside of the cofferdam, affecting selection of riprap scheme employed.

6.6.2.2.1 Armoring Placed Outside of Cofferdam (In-the-Wet)

Constructability dominates this case.
Reliable placement of light riprap gradations in flowing water, at the MBSD project depths (up to 50 feet), has not been demonstrated as possible with surface dump methods. USACE District experience in the Mississippi River reflects loss of fine (<4 lb. particle) stone material within riprap, which tends to drift during in-the-wet surface dump placement, often subsequently found hundreds of feet downstream. Heavy stone rock dikes and riprap are routinely placed by USACE directly on river banks without filter layers. With filter layers omitted, increased stone layer thickness is reported as successful in reducing water turbulence at the interface with underlying banks, such that erosion of fines is not widespread. Monitored and maintained, revetments constructed by these techniques have held the river bank location static for decades.

Using the required ACM lap distance, riprap limits were established from the extent of protection cells, cofferdam, and intake channel excavation. A working gradation was selected for similarity to the USACE B-Stone, then thickness proportioned from maximum stone size with an underwater placement factor. The Grade Stone B material used by the USACE is most similar in the D_{50} range to LaDOTD 130lb Class Riprap. See **Figure 6.6-1** superimposing LaDOTD 130 lb Class Riprap gradation on Grade Stone B gradation plot.

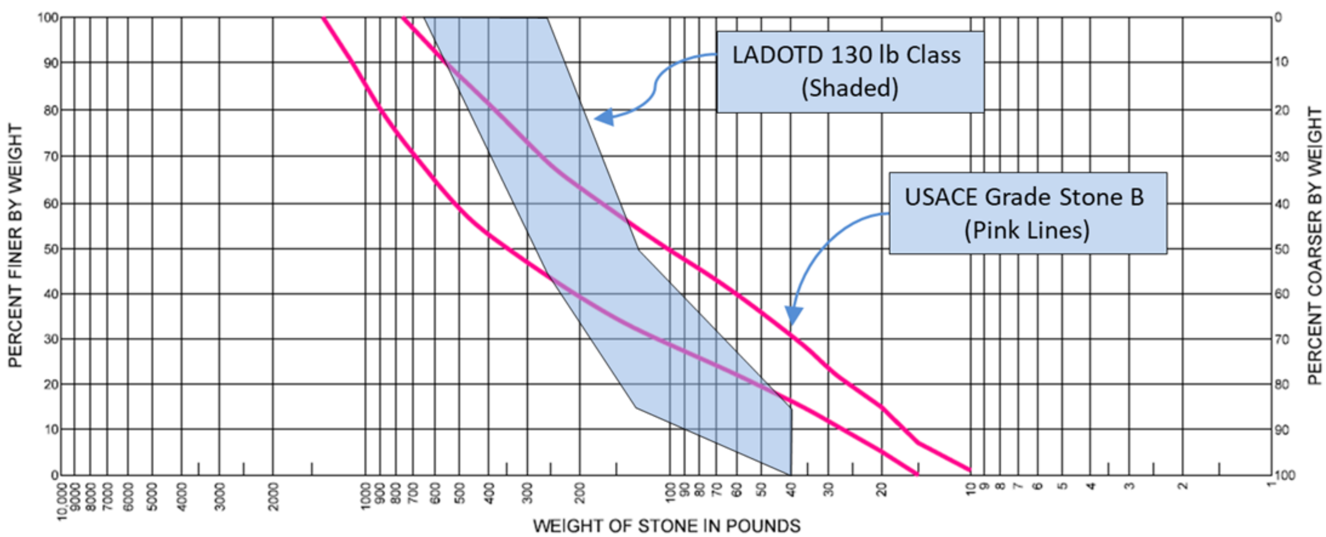


Figure 6.6-1: LaDOTD 130 lb Class Riprap and USACE Grade Stone B Gradation Plots

The existing ACM revetment must be cleanly cut and removed where change in bank elevation is required for the intake channel, prior to placement of riprap armoring.

6.6.2.2.2 Armoring Placed Inside of Cofferdam (In-the-Dry)

Riprap placed within the cofferdam functions no differently from the riprap outside of the cofferdam, but the section thickness may be decreased due to more favorable construction conditions. Geotextile fabric and crushed stone are used as filter/foundation for the armoring layer. Existing ACM within the cofferdam is to be removed.

6.6.2.2.3 MRL Armoring

Slope paving is to be placed on the re-constructed MRL in the same geometry as the existing slope paving. A 6-inch PCC with appropriately placed contraction & expansion joints is required.

Riprap armoring terminates 30 feet from the toe of the levee to allow vehicular access on levee toe side. Depth Averaged Velocity (DAV) was analyzed at this distance from levee to demonstrate resistance to model-predicted velocity by grass turf (up to 6.0 ft/s is acceptable when grass is established in clayey soil, EM 1110-2-1601).

6.6.2.3 Design Criteria and Loading Conditions

Armoring is designed to stabilize a channel or embankment by resisting:

- Tractive force-induced movement of revetment material
- Piping erosion of underlying fines
- Undermining by scour at the toe
- General revetment slump (underlying bank slope failure)

Minimum riprap gradation to resist tractive forces was selected for several locations using DAV for the controlling flow case (1,250,000 cfs MR flow; 95,000 cfs diversion flow). DAVs and near-bed velocity contours were provided from the April 2021 preliminary hydraulic model by FTN. Both the Isbash formula and EM 1110-2-1601 Eq. 3-3 were employed, and the results were computed for DAV at three selected locations (points 4 & 5 are over intake slab), and for the maximum velocity predicted on the armored section (9.0 ft/s, near tip of downstream guide wall). Model output is shown in the following figures. Calculations show the 130 lb class riprap is satisfactory for the most conservative case at maximum velocity.

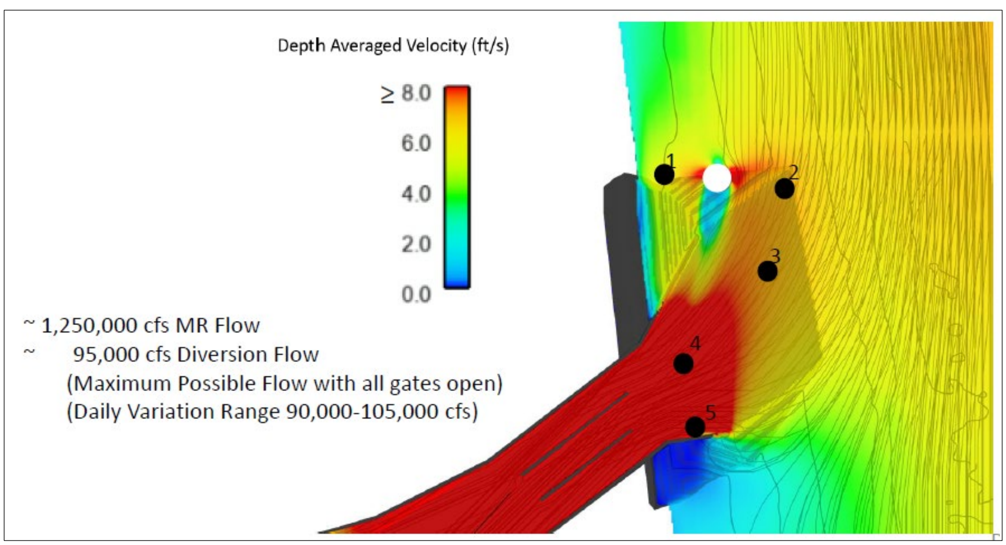


Figure 6.6-2: Depth-Average Velocities, 95k cfs MBSD / 1.25M cfs MR, from April 2021 Model

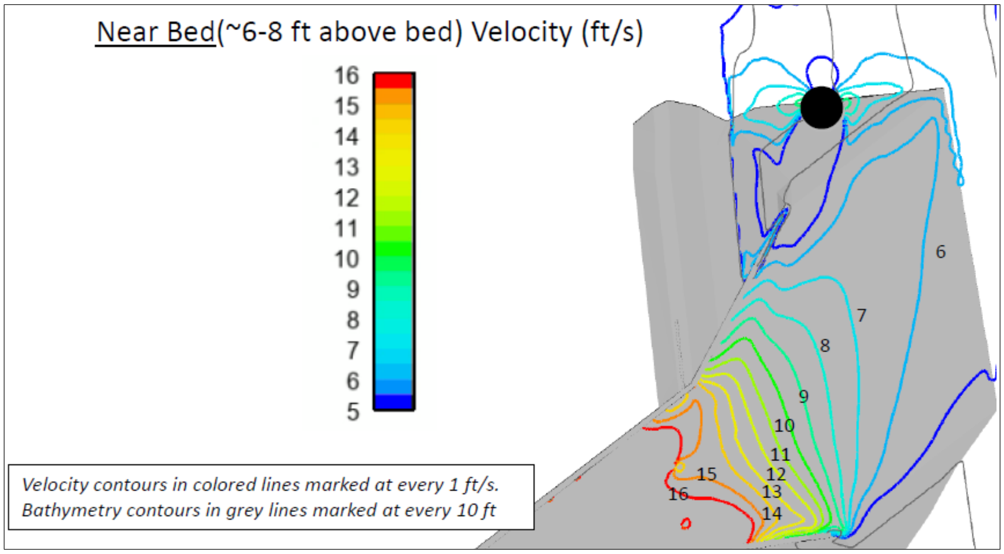


Figure 6.6-3: Near-Bed Velocities, 95k cfs MBSD / 1.25M cfs MR, from April 2021 Model

Layer thickness is calculated to accommodate the largest stone diameter, or a multiple above the median stone diameter, and is increased for in-the-wet placement to account for uncertainty.

6.6.2.4 Storm Surge and Wave Impacts on Intake Armoring Requirements

During the diversion non-operational season, the river water levels and wind generated waves at the intake location may control the armoring requirements. The diversion non-operational season also coincides with the U.S. hurricane season, and there is potential for storms surge and storm generated waves to increase. An analysis has been conducted to determine these potential impacts. A 50-Year design condition was designated for the analysis. The 100-Year conditions were also evaluated. The 50-Year and 100-Year storm conditions were provided by the USACE and are summarized in **Table 6.6-2** below.

Table 6.6-2: Summary of Surge and Wave Design Conditions for the Intake Armoring

Return Period (yrs)	50	100
Surge (ft, NAVD88)	12.7	14.5
Hs(ft)	2.3	3.8
T (sec)	2.5	3.8

The analysis consists of two areas. The flat intake area that is at EL -25 and the sloped sides of the intake which extend from EL -25 to EL -10 with a 1:4 slope. The areas are designated in **Figure 6.6.4**. For the flat area, the approach presented by Schiereck has been applied (Schiereck, 2012). For the submerged side-slopes, the approach of van de Meer (1991) was used.

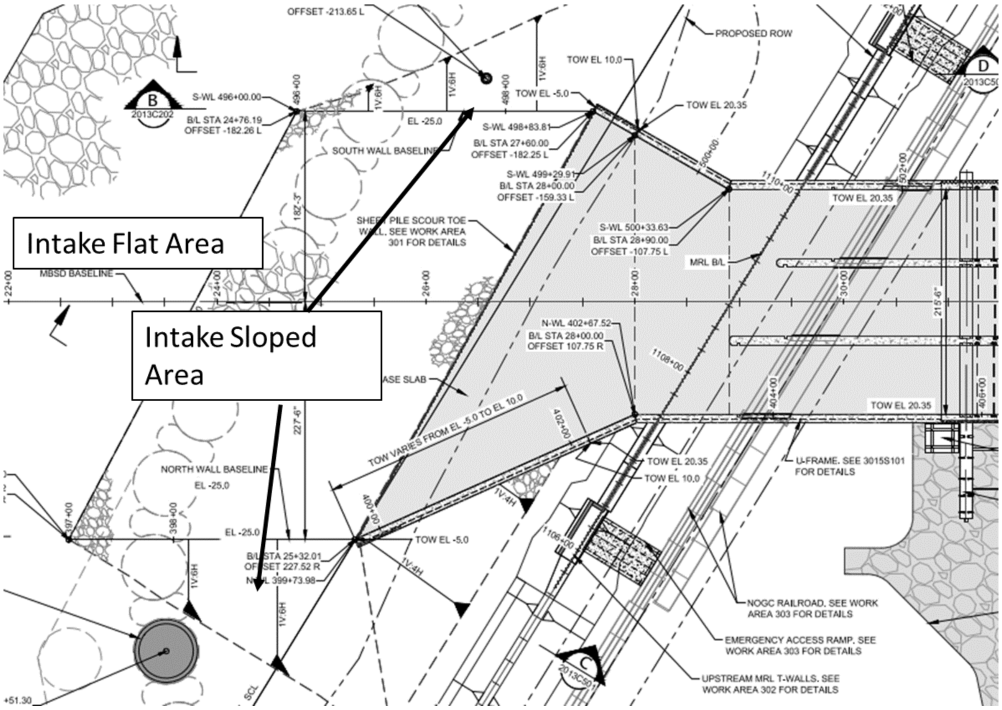


Figure 6.6-4: Armored sections of the Diversion Intake

The results are summarized in **Table 6.6-3** for the flat section of the intake and in **Table 6.6-4** for the sloped sections.

Table 6.6-3: Summary of result for the flat section of the intake

Design Condition (yr)	Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)
50	2.3	3.8	<0.25
100	3.8	3.8	<0.25

Table 6.6-4: Summary of result for the sloped sections of the intake

Design Condition (yr)	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)
50	37.7	2.3	2.5	13.2
100	39.5	3.8	3.8	16.5

These armoring riprap sizes have been compared to those developed using the design flow conditions and it was determined that the riprap sizes required from the design flow conditions control the riprap sizes. Further details can be found in the BODR **Appendix H.11**.

6.6.2.5 Analysis and Design Summaries

Using the DAV velocity at each modeled point with the bank slope, gradation characteristics, thickness coefficients, and channel characteristic inputs, the EM 1110-2-1601 Equation 3-3 and Isbash Equation were used to calculate required gradation to resist tractive forces. See the following table summarizing inputs, calculated values, and resulting minimum LaDOTD riprap gradation class.

Table 6.6-5: Calculated Minimum Riprap Gradations from DAVs

EM 1110-2-1601 Equation 3-3 and Isbash Inputs				
Point No:	**1**	**2**	**3**	**Max V**
Sf	1.1	1.1	1.1	1.1
Cs	0.3	0.3	0.3	0.3
Cv	1.25	1.25	1.25	1.25
Ct	1	1	1	1
d (ft)	**9**	**34**	**34**	**33**
γ_s (lbs/ft^3)	155	155	155	155
γ_w (lbs/ft^3)	62.4	62.4	62.4	62.4
V (ft/s)	**5.5**	**6.9**	**7**	**9**
theta (1V:5H)	**14**	**14**	**14**	**0**
phi	40	40	40	40
K_1	0.93	0.93	0.93	1.00
Low Turb. Isbash C	1.2	1.2	1.2	1.2
High Turb. Isbash C	0.86	0.86	0.86	0.86
Calculated Values				
EM Eq 3-3 D_{30} (ft)	**0.15**	**0.19**	**0.19**	**0.33**
Assumed D_{85}/D_{15}	4.0	4.0	4.0	4.0
EM Eq 3-3 D_{50} (ft)	**0.23**	**0.30**	**0.31**	**0.53**
Isbash D_{50} (ft) (Low Turb.)	**0.22**	**0.35**	**0.36**	**0.59**
Isbash D_{50} (ft) (High Turb.)	**0.43**	**0.67**	**0.69**	**1.15**
Min DOTD Class for Highest D_{50}	**10 lb**	**30 lb**	**30 lb**	**130 lb**

Given constructability requirement for 130 lb class riprap, construction and cost efficiency gained from singular gradation, and calculated minimum classes equal or lesser in size, 130 lb class riprap was chosen for the entire exposed armoring layer.

Layer thickness for in-the-wet placement on soil is maintained at 5 feet, since placement accuracy is considered low in flowing river water, and filter layers are omitted. In-the-wet placement on existing ACM is maintained at 3 feet for reduced placement accuracy but good foundation support. Armoring layer thickness in-the-dry is established at the maximum D_{100}, 2.0 feet, with 1.0 foot underlying stone filter/foundation layer and geotextile.

6.6.3 Transition from Gates to Channel

As illustrated on Sheets 5013C201 and 6013C401 and shown in **Figure 6.6-5**, the transition section is situated between the intake gates and the beginning of the Conveyance Channel. The 38-foot long by 215.5-foot wide section of U-frame downstream of the gate monolith that remains flat at EL -25.0.

The transition consists of pile founded T-walls which flare at varying angles and tie into the full conveyance levee section. The flare stations, angles, and cross-sectional areas are shown in **Table 6.6-6.** The cross-sectional area assumes a constant WSE of 6.5 feet.

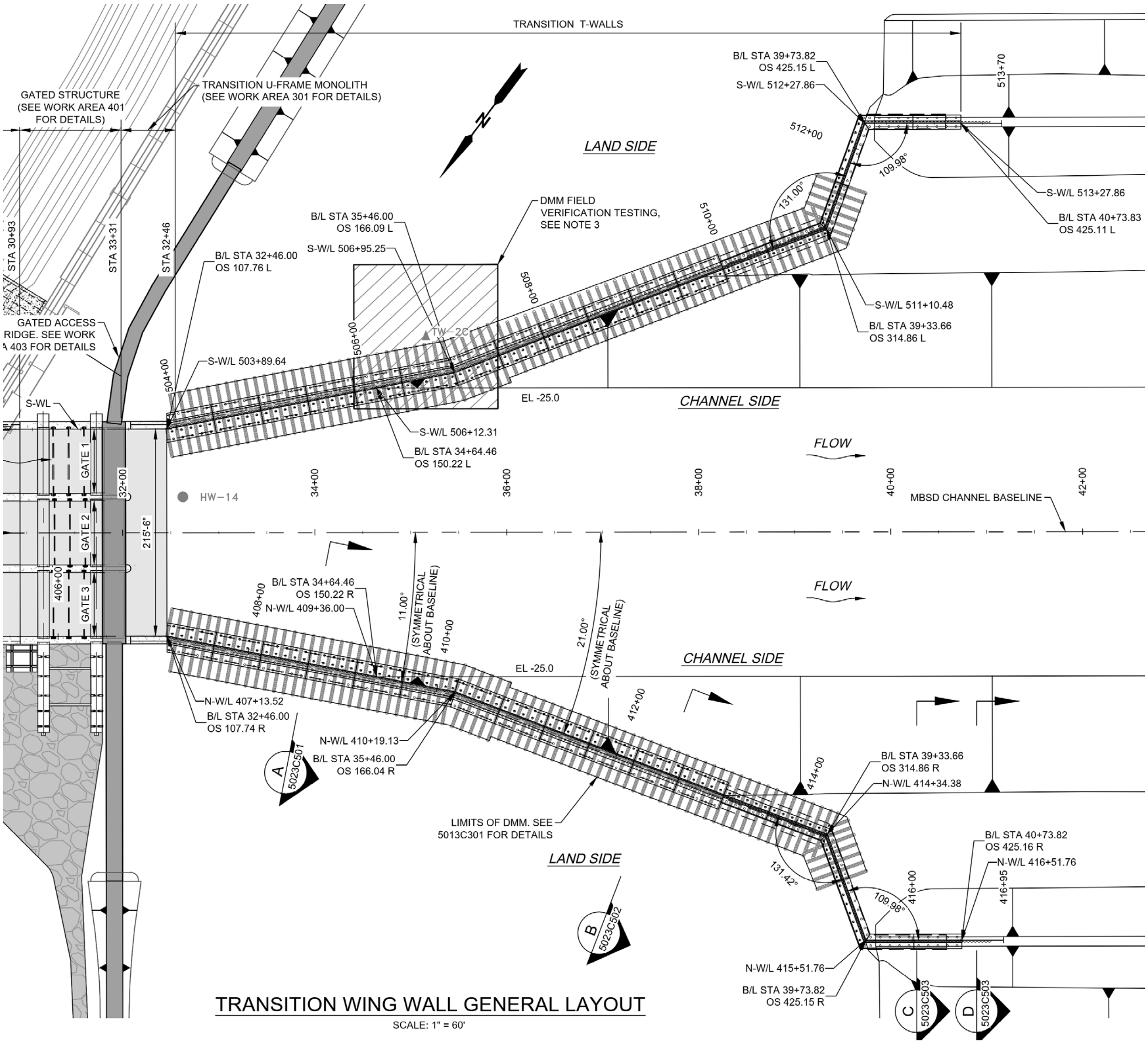


Figure 6.6-5: Transition Layout

Table 6.6-6: Cross-Sectional Areas at Various Locations along the Transition

Station	Position Description	Elevation (ft, NAVD88)	Cross-Sectional Area* (ft^2)
32+46.00	End of U-Frame Monolith	-25.0	6,788
35+46.00	End of first 11° flared section	-25.0	10,396
39+33.66	End of second 21° flared section	-25.0	13,338
39+73.82	End of third 70° flared section	-25.0	13,851
40+73.82	End of horizontal section Beginning of typical section	-25.0	13,803

*Assuming a Constant WSE = 6.5-ft, NAVD88

6.6.3.1 Numerical Hydraulic Modeling

Table 6.6-7 presents the results of four cases that were run in the hydraulic model, two with all the gates open and two with one of the gates closed. These represent potential scenarios; however, the target design condition is a diversion flow of 75,000 cfs when the corresponding river flow is at one million (1 M) cfs at the USACE Carrollton gauge. The locations A, B, C and D within the Transition section are depicted in **Figure 6.6-6**; they are at distances of 0 feet, 175 feet, 530 feet, and 750 feet, respectively, downstream of the gate monolith.

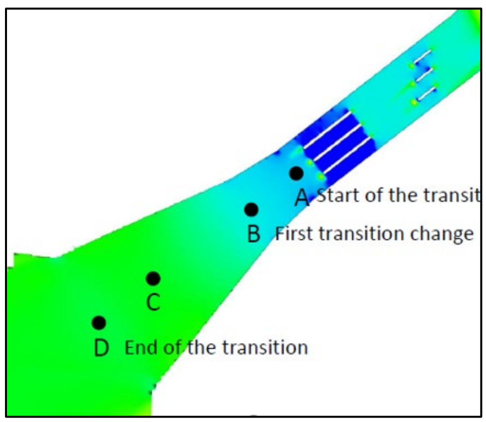


Figure 6.6-6: Locations of Transition Points

Table 6.6-7: Depth Averaged Velocities and Water Surface Elevations for Four Cases

River Flow (cfs)	Diversion Flow (cfs)	Gate Condition	Depth-Averaged Velocity (ft/s)				Water Surface Elevation (ft, NAVD88)			
			A (0 ft)	B (175 ft)	C (530 ft)	D (750 ft)	A	B	C	D
1.25 M	93,000	All gates open	10.5	9.7	7.2	6.8	7.1	7.5	7.9	7.9
	75,000	One gate closed	9.8	8.5	6.2	6	7	7.2	7.5	7.6
1 M	82,000	All gates open	9.2	8.3	6.4	6.3	6.2	6.5	6.9	6.9
	62,000	One gate closed	8.7	8.4	6.3	5.5	6.3	6.3	6.4	6.6

As **Table 6.6-7** shows, the two runs with the river at 1 M cfs bracket the target diversion flow of 75,000 cfs (one is at 62,000 cfs and the other at 82,000 cfs). An exact match to the 75,000 cfs target was not run because the Mid-Breton Sediment Diversion will be constructed upstream of the MBSD, but downstream of the Carrollton gauge. The Mid-Breton diversion is being designed to divert 75,000 cfs from the river, resulting in an estimated river flow at the MBSD of 925,000 cfs when the Carrollton gauge is at 1 M cfs. At that river flow, the MBSD diversion flow is anticipated to be approximately 75,000 cfs.

The results shown in **Table 6.6-7** indicate the depth-averaged velocities and the water surface elevations at the various locations for each of the cases. To determine the size of riprap required to withstand the corresponding hydraulic forces, the variables needed to apply Equation 3-3 are velocity and depth of flow. The water surface elevations must thus be added to the depth of the channel bottom. Thus, the flow depths range from 47.1 feet to 31.4 feet, corresponding to the combination of bottom and water surface elevations. **Table 6.6-8** was generated by applying Equation 3-3 using the derived numbers.

Table 6.6-8: Required Stone Weights for Various Flows & Gate Conditions

Flows and Gate Conditions			Required Stone W_{50} Weight (lbs)			
River Flow (cfs)	Diversion Flow (cfs)	Gate Condition	Point A (0 ft)	Point B (175 ft)	Point C (530 ft)	Point D (750 ft)
1.25 M	93,000	All gates open	35	20	2.9	1.9
	75,000	One gate closed	21	7.6	0.9	0.7
1 M	82,000	All gates open	13	6.3	1.2	1.1
	62,000	One gate closed	8.4	6.9	1.1	0.4

As **Table 6.6-8** shows, the rock size required to withstand the fluid forces rapidly diminishes as the flow moves downstream after exiting the gates. Interpolating to the target 75,000 cfs between the diversion flows of 82,000 cfs and 62,000 cfs yields a W_{50} stone weight of 11.4 lb at Point A, immediately downstream of the flat U-frame monolith. Based upon that analysis, an LaDOTD Class 10 lb stone does not quite meet the flow resistance requirements; therefore, an LaDOTD Class 30 lb stone would be selected, which will provide a factor of safety (FOS) of 2.6 at Point A and significantly higher FOS's downstream.

However, the cases described above are not the only conditions that the Transition section may encounter. The operation of the gates has a significant effect on the flow regime in the Transition section. Two additional scenarios were run to determine the velocities that might be reached with partially opened gates:

1 bay open 5 feet, 3 bays fully open. River flow 1.25 M cfs. Diversion flow of 75,000 cfs.
[River stage at 10-ft, NAVD88, Conveyance Channel End WSE at 6.6-ft NAVD88 to achieve flow.]

4 bays open 5 feet, River flow 1.25 M cfs. Diversion flow of 23,000 cfs.
[River stage at 10-ft, NAVD88, Conveyance Channel End WSE at 1-ft NAVD88 to achieve flow.]

While an unlikely scenario to be maintained for a significant length of time, the greatest velocity in the Transition section occurs under Scenario 2 - all four gates opened only 5 feet, with the river at 1.25 M cfs, and the diversion flow of 23,000 cfs. Under this scenario, the velocities under the gate can reach the 20 – 25 ft/s range, as shown in **Figure 6.6-7**.

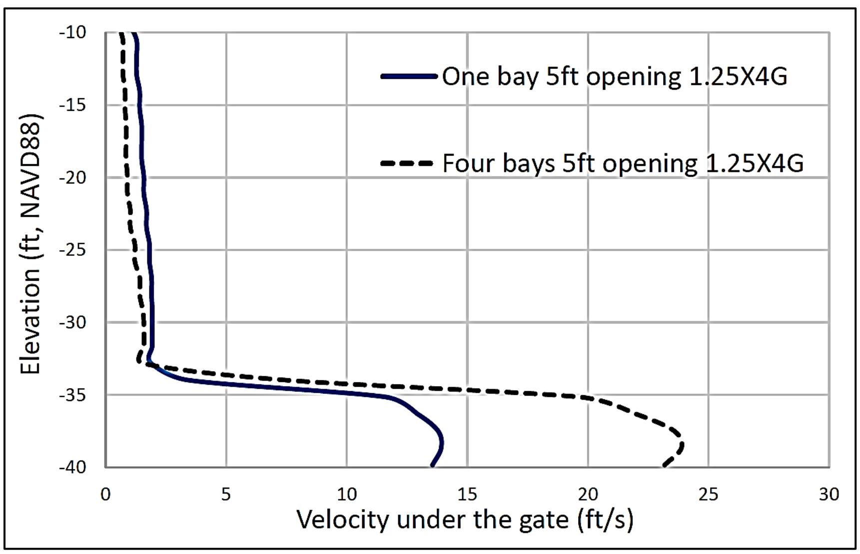


Figure 6.6-7: Velocity Under a Partially Open Gate or Gates

Additional hydraulic analyses show the velocities at 10 feet, 40 feet, 70 feet, and 100 feet past the gate opening into the transition section, with all gates partially open 5feet. The data from these analyses along with a depiction of where the various data collection points are located are plotted in **Figure 6.6-8**.

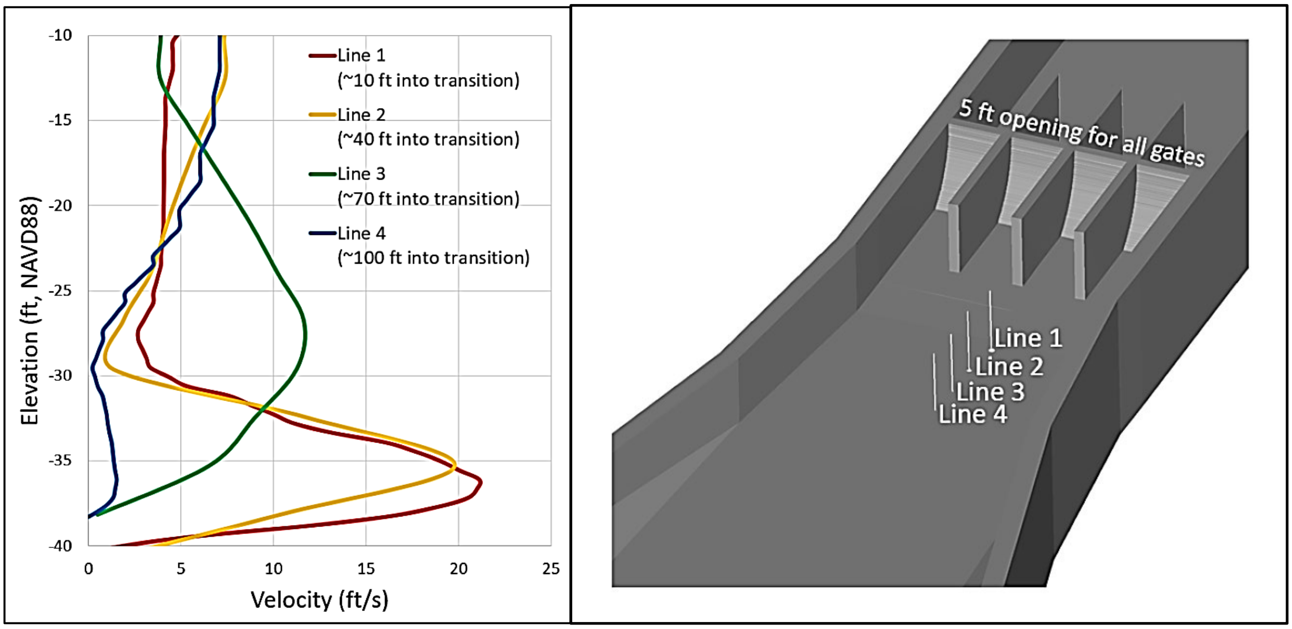


Figure 6.6-8: Velocities at Various Locations for Partially Opened Gates

As **Figure 6.6-8** shows, the maximum flow velocities up to 40 feet away from the gates range from 20 – 22 ft/s; they slow to 12 ft/s 70 feet from the gates; and they are less than 10 ft/s 100 ft into the transition. The data is presented in **Table 6.6-9** along with the corresponding riprap size and weight that would be required to resist such velocities.

Table 6.6-9: Stone Sizes Required for Partially Opened Gates, Assuming 32-ft Water Depth

Distance from Gate	Velocity (ft/s)	D_{50} Size (ft)	W_{50} Weight (lb)
10-ft	22	5	10,500
40-ft	20	4	5,000
70-ft	12	1.1	111
100-ft	7	0.3	2

As **Table 6.6-9** shows, very large, Derrick stone-sized riprap of 4 feet to 5 feet in diameter and weighing 5,000 lbs to over 10,000 lbs would be required to withstand the forces generated by water flowing at 20 – 22 ft/s. These sizes are not feasible for use in the transition section because the layer thickness of a 5-foot diameter stone placed in-the-wet would be 7.5 feet, which is approximately one-quarter of the depth of flow, meaning that a significant portion of the diversion flow would be routed through the stone pore space instead of over a stone lining.

At a distance of 70 feet from the gates, the velocity drops to 12 ft/s. Applying Equation 3-3 with that velocity and a water depth of 32 feet yields a D_{50} size of 1.1 feet and a W_{50} weight of 111 lbs. The closest riprap size is the LaDOTD 130 lb class, which has a D_{50} size of 1.17 feet and a W_{50} weight of 130 lbs. **Figure 6.6-9** presents the gradation curves for the LaDOTD 130 lb Class riprap. The required layer thickness calculated from Equation 3-3 is 1.75 feet. Assuming the riprap is installed in-the-wet, then the required thickness becomes 1.5 x 1.75 feet = 2.63 feet, which is rounded to 3 feet. Thus, outside of the unusual situation of the gates all being partially open 5 feet, it appears that the transition section, up to the 70-foot mark from the gates can be protected by a 3-foot thick layer of 130 lb stone.

Beyond the 70-foot distance from the gates, the velocity continues to drop, indicating that smaller riprap can be used in the remainder of the Transition section. The following section describes the physical modeling done in the Transition section, which further refines the findings.

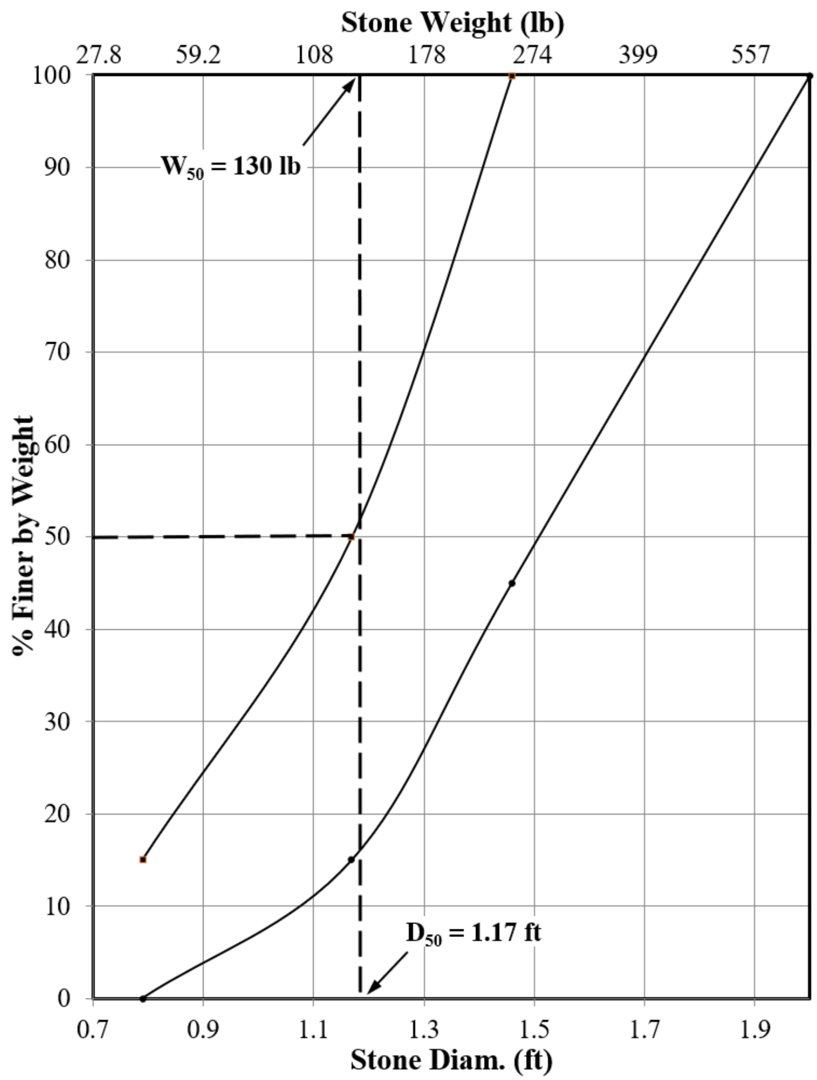


Figure 6.6-9: Gradation Curves for LaDOTD 130 LB Class Riprap

6.6.3.2 Physical Hydraulic Modeling

Current with the mathematical hydraulic modeling, additional physical modeling was also conducted to confirm and refine the findings. A 1:65 scale model of the Transition section was constructed with a live bed lined with scaled riprap simulating the LaDOTD 130 lb class. The most adverse operating condition occurs at a high river flow when the gates are partially opened with a low downstream water level.

Riprap stability in the Transition section was tested for two operating scenarios:

> Scenario 1 consisted of one gate closed and the other three gates partially opened. Testing was completed under five simulated conditions: with the three gates open 1 feet, 2 feet, 4 feet, 6 feet, and 10 feet from the bottom. The water level downstream of the gates was set at 0 feet NAVD88, representing a low water level in the conveyance channel on startup.
>
> Scenario 2 was designed to replicate a condition predicted in the hydraulic CFD models; all four gates were open 5 feet, and the diversion flow was set at 23,000 cfs based on worst case conditions (river stage at 10 feet NAVD88, conveyance channel end WSE at 1 foot NAVD88).

Under Scenario 1, no riprap movement was observed when the three gates were open 1 foot and 2 feet. When the three gates were open 4 feet and 6 feet, minor riprap movement was observed, i.e., a few individual stones moved. When the gates were opened 10 feet a small scour pocket developed downstream of gate 2; however, it stabilized and did not grow. The scour depth was limited to a few stone diameters. While the physical test was a steady state condition, in the actual diversion these conditions would have a short duration. As flow is added to the conveyance channel, the water level will start to rise, reducing the head across the gate and the flow through the gate.

Under Scenario 2, no riprap movement was observed. Observations made by injecting a dye indicator in the physical model showed faster jet breakup downstream of the gates than the CFD model predicted. A high frequency (1200 Hz) Acoustic Doppler velocimetry probe was used to record instantaneous velocities at 0.05 second intervals. Velocities were measured at 10 feet, 40 feet, 70 feet and 100 feet downstream of the gate monolith. Immediately downstream of the gate monolith, the instantaneous peak velocity measured was approximately 21 ft/s, while the peak thirty second average velocity was approximately 8.5 ft/s.

The data from the simulated run with the river at 1.25 M cfs, a diversion flow of 23,000 cfs, and all four gates open 5 feet are plotted in **Figure 6.6-10**.

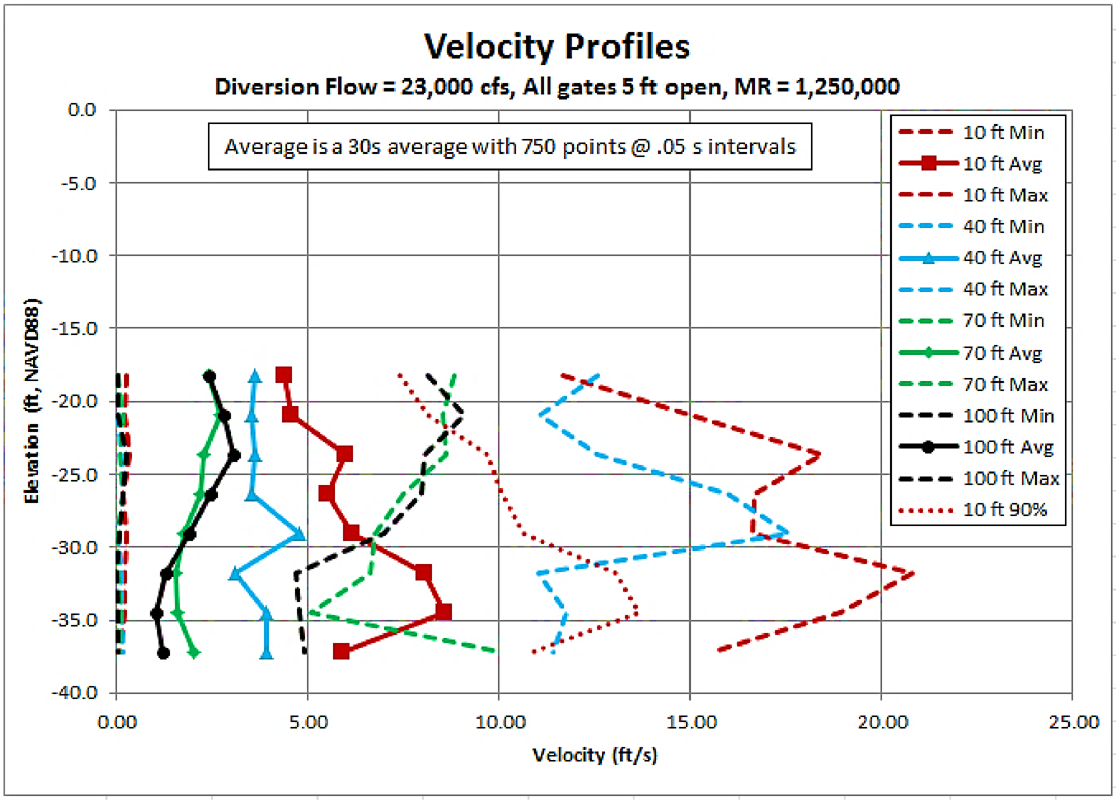


Figure 6.6-10: Velocity Profiles in the Transition Section - All Gates Open 4-ft, River at 1.25 M cfs, and Diversion Flow of 23,000 cfs

As **Figure 6.6-10** shows, the maximum velocity at 10 feet from the gates can reach 20 ft/s or more, albeit under transient conditions that only last a few seconds before the energy dissipates. The average velocity at 10 feet from the gates is less than 10 ft/s. The maximum velocity at 40 feet from the gates exceeds 15 ft/s, while the average velocity at that location is less than 5 ft/s. At 70 feet from the gates, the velocity can reach 10 ft/s, with an average velocity of 2 – 2.5 ft/s. At 100 feet from the gates, the maximum velocity is always less than 10 ft/s, while the average velocity ranges from 1.5 – 3 ft/s.

Thus, the results of the physical modeling were in general agreement with the numerical CFD modeling in that the instantaneous peak velocity can reach levels above 20 ft/s under certain unique conditions. However, the effect is very short-lived and occurs only immediately adjacent to the gate outlet. Without sustaining the velocities for any significant period, the transient hydraulic forces do not appear sufficient to move the 130 lb stone.

As noted above, at the 70-foot distance, the velocity can still reach 10 ft/s. Applying Equation 3-3 with a velocity of 10 ft/s and a water depth of 32 feet yields a required D_{50} stone size of 0.70 feet with a W_{50} weight of 28.3 lbs. To provide adequate erosion protection under these conditions the LaDOTD 30 lb stone class was selected. It has a D_{50} stone size of 0.72 feet with a W_{50} weight of 30 lbs, as shown on **Figure 6.6-11**. The requisite thickness is 1.22 feet; multiplying by 1.5 for in-the-wet placement results in a layer thickness of 1.83 feet, which is rounded to 2 feet.

Based on both the numerical and physical modeling results, the Transition section armoring will be a 3-ft thick layer of 130 lb stone up to a distance of 70-ft from the discharge U-Frame. For the remainder of the transition section, the protection will be a 2-ft layer of 30 lb stone. Additional physical modeling and analyses is being performed to refine the required revetment configuration. The need for an additional 50-feet to 75-feet of concrete U-Frame immediately downstream of the gates to address the potential very high velocities will be addressed. Further refinement of the riprap revetment will be addressed as well.

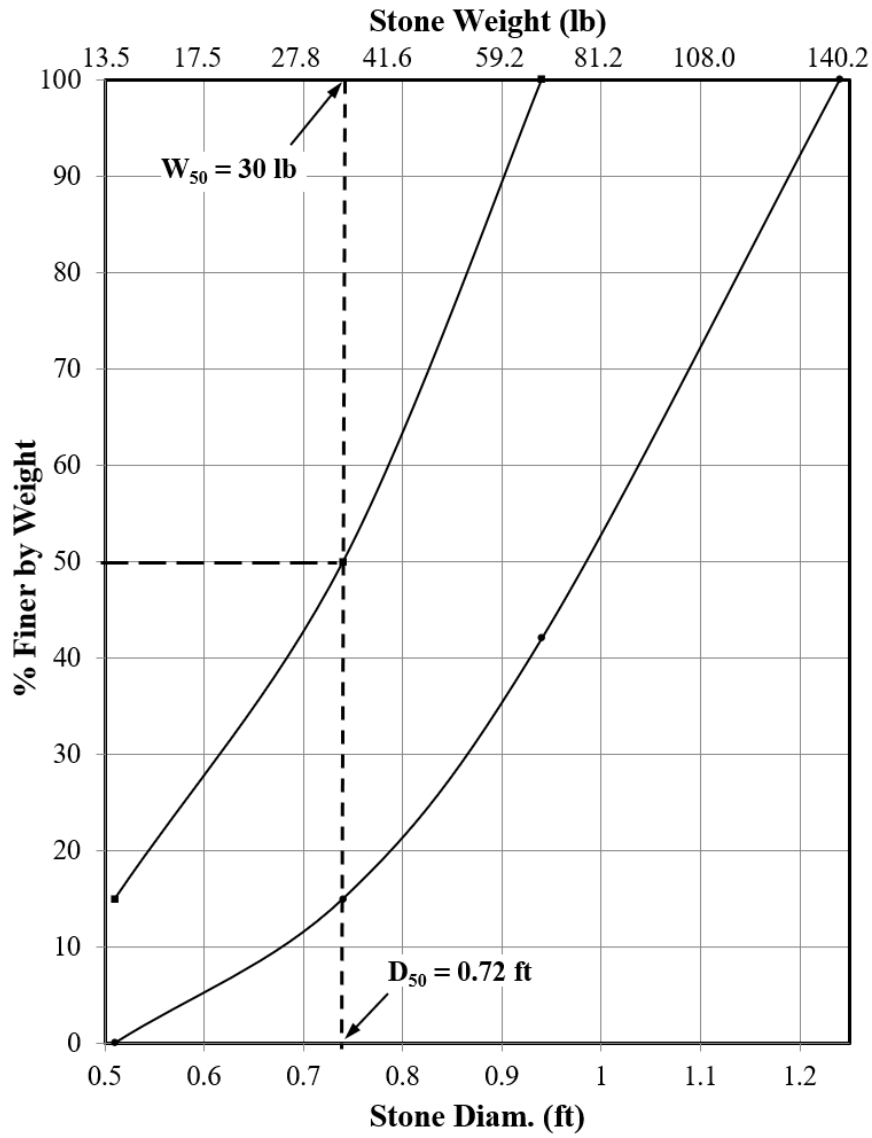


Figure 6.6-11: Gradation Curves for LaDOTD 30 LB Class Riprap

6.6.4 Conveyance Channel

6.6.4.1 Numerical Hydraulic Modeling

Based on the MBSD target flow of 75,000 cfs and the cross-sectional area of the basic Conveyance Channel, the average velocity under normal conditions is approximately 7 ft/s. This is greater than the bare soil can withstand without eroding, so a revetment material is required for erosion protection. As described in the BODR and the Revetment Study, the riprap selected to protect the wetted surface of the Conveyance Channel during normal flow conditions was 10 lb stone per the LaDOTD classification system.

To briefly reiterate how that value was derived, the hydraulic conditions determined by the Computational Fluid Dynamic (CFD) modeling within the Conveyance Channel (as discussed in **Section 3** above) are displayed graphically in **Figure 6.6-12**.

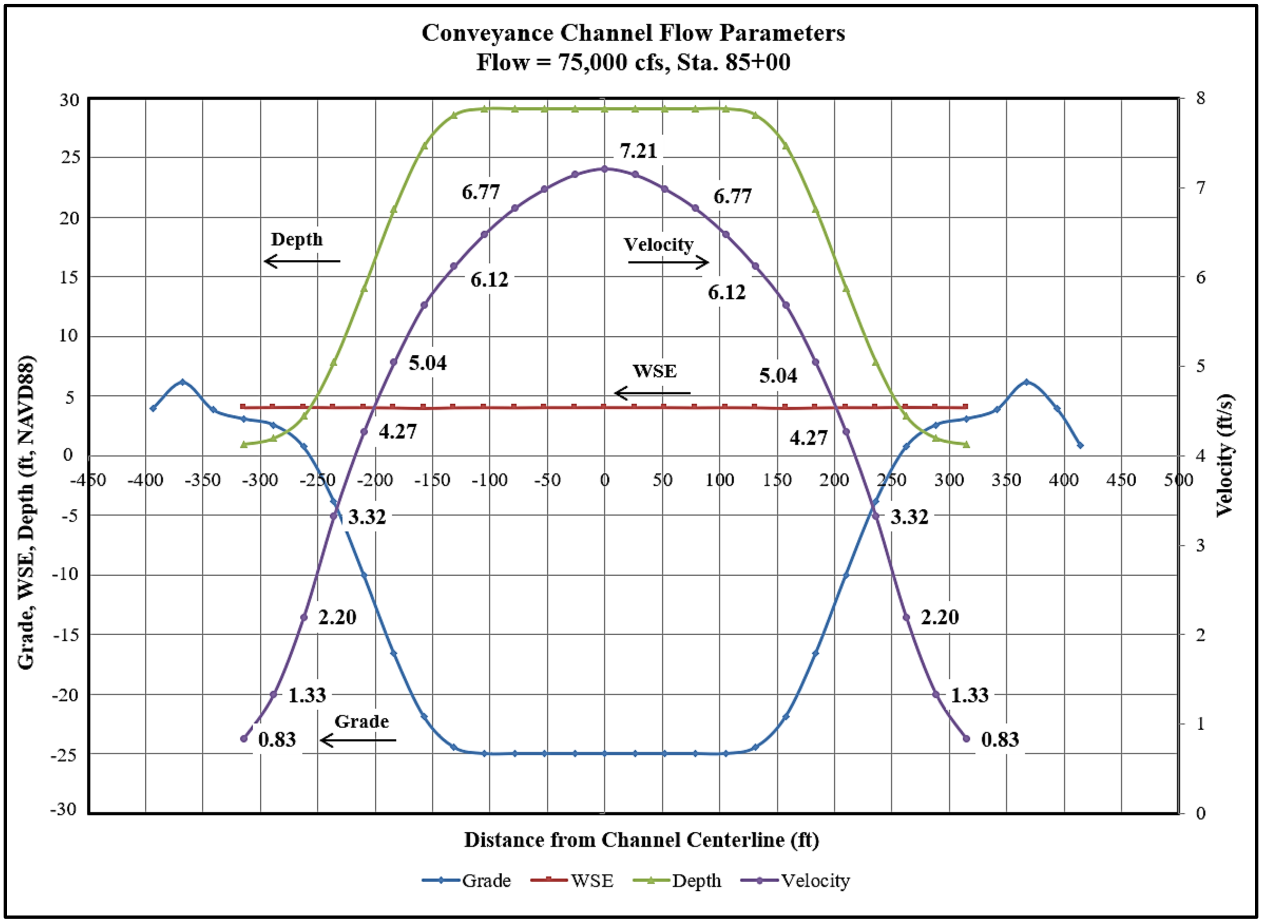


Figure 6.6-12: Water Surface Elevation, Depth & Velocity at Mid-Channel (Sta 85+00) for Normal Flow Conditions (75,000 cfs) [Velocities are Depth-Averaged Velocities]

As shown, the peak depth-averaged velocity within the channel is 7.21 ft/s, which occurs at a corresponding water depth of 29.25 feet. The values of those parameters were entered into Equation 3-3 from the USACE EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*, which is incorporated into the following spreadsheet, presented as **Figure 6.6.13**.

Design of Rip-Rap Revetment						
Basic Input Data	Velocity of water	v =	7.21	ft/s		
	Unit Weight of Stone	γs =	155	lb/ft³	← 155 lb/ft.³ is the conservative value assumed by the USACE	
	Unit Weight of Water	γw =	62.4	lb/ft³		
	Specific Gravity of Stone	sg =	2.48	-----		

Method	Equation	Value	Additional Parameters	D_{50}	Weight	Layer Thick*
EM-1601 (USACE) 1994	$D_{30} = S_f C_s C_v C_t d \left((\gamma_w / (\gamma_s - \gamma_w))^{0.5} (v / (K_1 g d)^{0.5}) \right)^{2.5}$					
	Sf = Safety factor	1.2				
	Cs = Stability coefficient	0.3	angular rock			
	Cv = Vertical velocity distrib. coeff.	1	straight channel			
	C_T = Blanket thickness coeff.	1	assume stnd uniform. ratio			
	d = Local depth of flow, ft	29.25				
	γw = Unit weight of water, lb/ft³	62.4				
	γs = Unit weight of stone, lb/ft³	155				
	v = velocity, ft/s	7.21				
	g = gravitational constant, ft/s²	32.2				
	Side slope H:V	10	$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$			
	θ = angle side slope w/ horizontal	6				
	Φ = angle of riprap repose	40	assume stnd angle			
	K1 = Side slope correction factor	0.988				
	D30 =	0.175 ft 2.10 in	D50 =	0.318 ft	2.6 lb	1.00 ft

*Layer Thickness = Greater of : 1½ x D_{50}, D_{50} + ½-ft, or 1-ft:		Assume D30/D50 = 0.55
1½ x D_{50} =	0.48 ft	
D_{50} + ½-ft =	0.82 ft	
1-ft	**1.00 ft**	

Figure 6.6-13: Design of Riprap for velocity = 7.21 ft/s and depth = 29.25-ft

Using 155 lb/ft³ as the unit weight of stone (the USACE recommended value, which is relatively conservative), yields a D_{50} size stone of 0.318 feet, which would weigh approximately 2.6 lbs. Therefore, the 2 lb LaDOTD stone classification is just slightly too small, placing the required stone in the 10 lb Class. As shown on the following gradation curve for 10 lb stone, the minimum limit of the D_{50} size is 0.51 feet, which is approximately 60% larger than the required 0.318 feet, which increases the Safety Factor from 1.2 assumed in the above calculation (using the 2.6 lb stone) to 1.9 (with the 10 lb stone).

The layer thickness is based off the greater of the upper limits of the 10 lb stone, either: 1) The D_{100} which is 0.88 feet, or 2) 1.5 times the upper limit of the D_{50} which is 1.5 x 0.65 feet = 0.98 feet. Thus, the required layer thickness for in-the-dry construction is approximately 1 foot. If the riprap is to be placed in-the-wet, then per USACE criteria, an additional 50% is added to the required thickness, resulting in an approximately 1.5-feet thick layer.

Figure 6.6-14 presents the gradation curves for the LaDOTD 10 lb Class riprap.

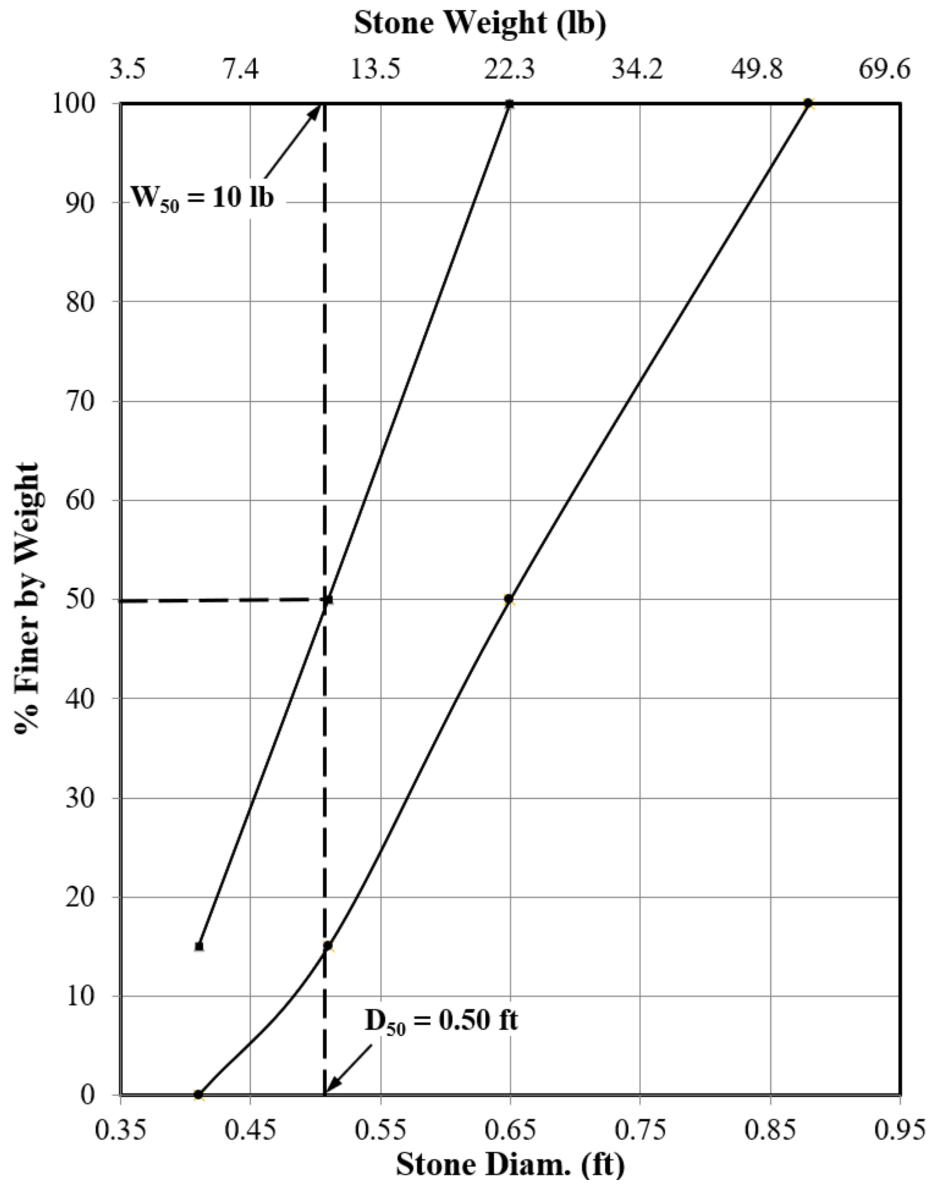


Figure 6.6-14: Gradation Curves for LaDOTD 10 LB Class Riprap

6.6.4.2 Physical Hydraulic Modeling

Physical models of the Conveyance Channel, Transition section, and Outfall have been constructed and data has been collected in an on-going effort to determine riprap stability, supplementing the hydraulic modeling and stability calculations. The model of the Conveyance Channel was constructed at a 1:65 scale, with a live bed, and scaled riprap. Local velocity measurements were made at various points throughout the model and observations of riprap stability or movement were made. The riprap used in the channel model was scaled to simulate the 1.5-feet thick layer of LaDOTD 10 lb stone indicated by the design approach described above.

The stability or movement of the model riprap generally correlates to the behavior of the actual full-scale riprap. However, if the model riprap is stable, the factor of safety cannot be determined directly because it is unknown whether it is in a state of incipient motion. Increasing the flow velocity to the point at which motion is observed can confirm stability at lower velocities but does not precisely quantify the factor of safety. The conditions most likely to destabilize the riprap are for a high Conveyance Channel flow and a low tailwater condition in Barataria Bay.

To confirm the stability of the 10 lb riprap for the Conveyance Channel, the test throughput (and thus water velocity) was increased from the design flow of 75,000 cfs to 104,000 cfs (a 39% increase). This condition is more adverse than what is physically possible, representing a high river level and tailwater level in Barataria Bay below EL 0.0. The flow is more than the discharge capacity of the diversion and the tailwater level would not be that low with 104,000 cfs discharging into it. None-the-less, no riprap movement was observed under these test conditions, indicating that the 10 lb riprap is very stable for the conditions likely to occur within the Conveyance Channel.

6.6.4.3 Storm Event Flow Conditions

During the diversion non-operational season, the river water levels and wind generated waves at the intake location may control the armoring requirements. The diversion non-operational season also coincides with the US hurricane season, and there is a potential for storms surge and storm generated waves to increase in the basin adjacent to the outfall. The surge and waves will propagate into the channel and may impact the armoring requirements for outfall and channel features. An analysis was completed to determine the armoring requirements for the storm events.

The 50-Year and 100-Year design conditions were available from previous work documented in the 2017 update to the USACE document: Elevations for Design of Hurricane Protection Levees and Structures Report. The report provides data for sections along the non-Federal NOV levee as well as many other levee systems. The 50-Year and 100-Year surge and wave conditions for the levee section NOV-NF-W-05c, whose location coincides with the conveyance channel outfall, were obtained from the report. The data for “existing conditions” is summarized below in **Table 6.6-10**.

Table 6.6-10: Summary of Surge and Wave Design Conditions

Return Period (yrs)	50	100
Surge (ft, NAVD88)	7.1	9.3
Hs (ft)	2.1	4.1
T (sec)	4.1	4.8

It is recognized that the design surge and wave may not occur simultaneously, and therefore the design condition for evaluating the armoring stone sizes may be governed by other conditions. To evaluate this possibility, an additional range of surge levels were considered using the 50-Year (and 100-Year) design wave. These design surge and waves were propagated into the channel using a 2D wave Model CMS-Wave. To evaluate the impact of non-coincident design surge and wave conditions on armoring size estimates, a range of possible surges were considered using the 50-Year (and 100-Year) design wave. The analysis applied the design wave conditions on the offshore boundary of the CMS-Wave model grid and propagated the waves into the channel. The results are summarized in **Table 6.6-11**.

Table 6.6-11: Summary of Wave Conditions for Range of Surge Elevations for 50-Year Design Conditions

Surge (Ft, NAVD88)	Hs(ft) Outfall	Hs (ft) 1100 feet into Channel	Hs (Ft) 2500 feet into Channel
7.1	2.10	1.07	0.53
6.1	1.97	0.98	0.49
5.1	1.77	0.89	0.44
4.1	1.57	0.79	0.39
2.1	1.12	0.56	0.28

As the surge is decreased, the waves at the outfall decrease due to bottom frictions and additional sheltering in the vicinity of the outfall. The 50% and 75% decrease in wave height with distance into the channel was consistent for all cases considered.

The storm conditions were evaluated for 6 components of the channel and outfall ramp. The components and the analysis method are listed in **Table 6.6-12**.

Table 6.6-12: Diversion Components and Analysis Method

Component	Analysis Method
Hurricane Guide Levees	Van der Meer (1988) non-overtopped breakwaters
Channel Side Slopes	Van der Meer (1991) submerged breakwaters
Channel Bottom	Schiereck (2012) waves over flat surfaces
Channel Berm	Schiereck (2012) waves over flat surfaces
Outfall Ramp	Schiereck (2012) waves over flat surfaces
Outfall Ramp Side Slopes	Van der Meer (1991) submerged breakwaters

The results for each diversion component are summarized in the following tables. These armoring riprap sizes have been compared to those developed using the design flow conditions, and it was determined that the riprap sizes required from the design flow conditions control the riprap sizes for some of the channel and outfall features.

Table 6.6-13: Summary of Wave-based Riprap Size for Levee for 50-Year Design Conditions

Water Depth(ft)	Wave Height Hs (feet)	Wave Period (seconds)	Deepwater Wave Length (ft)	Wave Breaking	Riprap Size (lbs)	Applicable Range in Channel (measured from the outfall) (ft)
3.1	2.1	4.1	86	N	96.1	0 – 1100
3.1	1.07	4.1	86	N	17.3	1100 - 2500
3.1	0.53	4.1	86	N	1.9	2500 - 3500

Table 6.6-14: Summary of Wave-based Riprap Size for Levee for 100-Year Design Conditions

Water Depth(ft)	Wave Height Hs (feet)	Wave Period (seconds)	Deepwater Wave Length (ft)	Wave Breaking	Riprap Size (lbs)	Applicable Range in Channel (measured from the outfall) (ft)
5.1	2.9	4.8	118	N	251.6	0 – 1100
5.1	1.05	4.8	118	N	42.9	1100 - 2500
5.1	0.73	4.8	118	N	4.9	2500 - 3500

Table 6.6-15: Summary of Wave-based Riprap Size for Channel Slope for 100-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (ft)
2.1	4.1	33.1	0 – 1400
1.05	4.1	8.3	1400 - 2800
0.52	4.1	2.1	2800 - 3800

Table 6.6-16: Summary of Wave-based Riprap Size for Channel Slope for 100-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (Ft)
2.9	4.8	68.3	0 – 1400
1.45	4.8	17.0	1400 - 2800
0.73	4.8	4.3	2800 - 3800

Table 6.6-17: Summary of Wave-based Riprap Size for Channel Bottom for 50-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (ft)
2.1	4.1	<0.25	0 – 1400
1.05	4.1	<0.25	1400 - 2800
0.52	4.1	<0.25	2800 - 3800

Table 6.6-18: Summary of Wave-based Riprap Size for Channel Bottom for 100-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (ft)
2.9	4.8	<0.25	0 – 1400
1.45	4.8	<0.25	1400 - 2800
0.73	4.8	<0.25	2800 - 3800

Table 6.6-19: Summary of Wave-based Riprap Size for Channel Berm for 50-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (ft)
2.1	4.1	0.5	0 – 1400
1.05	4.1	<0.25	1400 - 2800
0.52	4.1	<0.25	2800 - 3800

Table 6.6-20: Summary of Wave-based Riprap Size for Channel Berm for 100-Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)	Applicable Range in Channel (measured form the outfall) (ft)
2.9	4.8	52.0	0 – 1400
1.45	4.8	0.63	1400 - 2800
0.73	4.8	<0.25	2800 - 3800

Table 6.6-21: Summary of Wave-based Riprap Size for Outfall Ramp for 50-Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)
Beginning	32.1	2.1	4.1	<0.25
Mid-way	23.6	2.1	4.1	<0.25
End	11.1	2.1	4.1	<0.25

Table 6.6-22: Summary of Wave-based Riprap Size for Outfall Ramp for 100-Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)
Beginning	34.3	2.9	4.8	<0.25
Mid-way	25.8	2.9	4.8	<0.25
End	13.3	2.9	4.8	0.5

Table 6.6-23: Summary of Wave-based Riprap Size for Outfall Ramp Side Slopes for 50-Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)
Base	32.1	2.1	4.1	33.1
Mid-way	23.6	2.1	4.1	25.6

__Table 6.6-24: Summary of Wave-based Riprap Size for Outfall Ramp Side Slopes for 100-Year Design Conditions__

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Riprap Size (lbs)
Base	34.3	2.9	4.8	68.3
Mid-way	25.8	2.9	4.8	49.3

6.6.5 Outfall from Channel to Wetlands

6.6.5.1 Numerical Hydraulic Modeling

As illustrated on Sheets 6043C101 and 6053C102, the Outfall section extends for 2,000 feet from the end of the Conveyance hannel out into the marsh. As with the Transition section, the Outfall section both widens in lateral extent and the bottom elevation increases as it extends from the Conveyance Channel to the marsh. The top width changes at an angle of 23° from horizontal on each side, starting at 750 feet wide on the Conveyance Channel end and widening to 2,000 feet at the marsh end. At the same time the bottom starts at EL -25.0 at the upstream end and rises to EL -4.0 at the terminus, following a gradual 1.05% slope.

The Outfall section's cross-sectional area is affected in opposite directions by the elevation of the bottom and the widening of the sides. The cross-section is reduced by the raising of the bottom, while it is increased by the widening of the channel. These geometric changes create corresponding variations in the flow velocity as discussed in **Section 3**. The varying velocities must be considered when designing the protective revetment. Overall, there is a significant reduction in the flow velocity as water moves through the Outfall section. This indicates that the channel widening has a greater effect in increasing the cross-sectional area than the upward shift of the bottom has in reducing the area.

Six numerical hydraulic modeling cases were run at various combinations of river flows, diversion flows, and outfall conditions. **Figures 6.6-15** and **6.6-16** present the modeling results. The cases are described in **Figure 6.6-17** which presents the legend for the plots. The OTF end line represents the end of the Outfall, where it transitions to open marsh; the CC end line represents the end of the Conveyance Channel, which is the beginning of the Outfall.

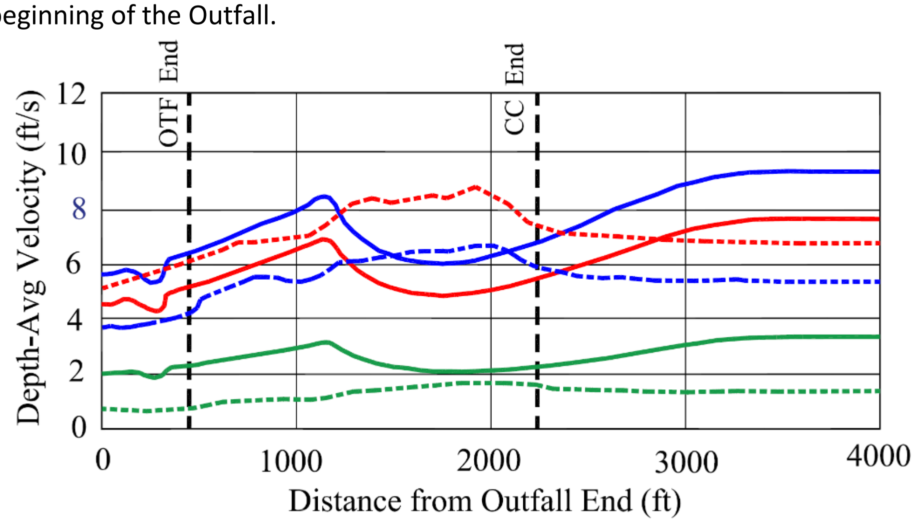


Figure 6.6-15: Depth-Averaged Velocities in the Outfall section for various model cases

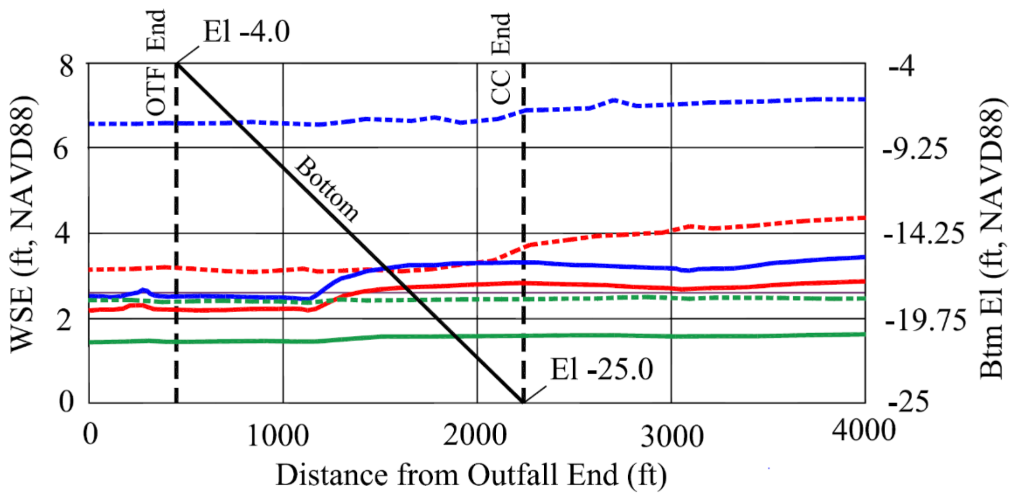


Figure 6.6-16: Water Surface Elevations in the Outfall section for various model cases

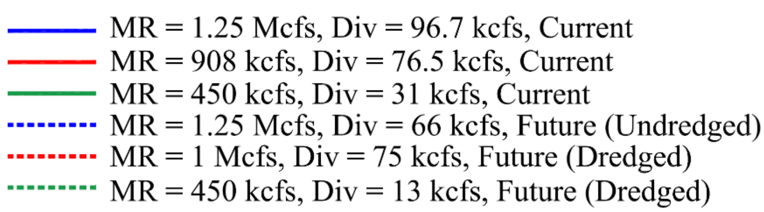


Figure 6.6-17: Legend describing the various model cases

The flow velocities within the Outfall section for the various cases range from 1.8 ft/s to 8.5 ft/s. The higher velocities occur at peaks in the velocity profile near the entry into the Outfall from the Conveyance Channel or in the middle of the Outfall section. Overall, the average velocities are less than that within the Conveyance Channel, for which the LaDOTD 10 lb Class riprap is recommended for erosion protection. However, the depth of water in the Outfall section is less than it is throughout the Conveyance Channel because, as shown on **Figure 6.6-16**, the bottom of the Outfall section rises from EL -25.0 at the channel end to EL -4.0 at the Outfall end. Other conditions being equal, it is the depth-averaged velocity and the flow depth that are the two salient variables entered into Equation 3-3 to calculate the required riprap size and weight.

Table 6.6-25 lists the conditions modeled in the Outfall section, as plotted above, along with the maximum depth-averaged velocity and associated water surface elevations and water depths. The final three columns present the resulting calculations from Equation 3-3 to determine the riprap requirements.

Table 6.6-25: Outfall Scenarios modeled and resulting maximum riprap requirements.

River Flow (cfs)	Diversion Flow (cfs)	Outfall Condition	Max Depth-Avg Velocity (ft/s)	Assoc. WSE (ft)	Assoc. Water Depth (ft)	Calc. D_{50} (ft)	Calc. W_{50} (lb)	Calc. Thick (ft)
1.25 M	96.7 k	Current	8.2	3.0	21.5	0.47	8.61	1.00
908 k	76.5 k	Current	7.0	2.5	21.0	0.32	2.68	1.00
450 k	31 k	Current	3.0	1.8	21.8	0.04	0.005	1.00
1.25 M	66 k	Future Undredged	6.5	6.5	12.4	0.30	2.28	1.00
1 M	75 k	Future Dredged	8.5	3.0	21.0	0.52	11.48	1.02
450 k	13 k	Future Dredged	1.8	2.5	20.5	0.01	.0001	1.00

As **Table 6.6-25** shows, using the maximum velocity values and associated water depths, the riprap requirements range from negligible to 11.5 lb stone (under the future dredged conditions with a river flow of 1 Mcfs and a diversion flow of 75,000 cfs). As **Figures 6.6-16** and **6.6-17** show, the combination of relatively high velocity and low water level occur in only this one scenario that represents the flow in a future dredged condition. The LaDOTD 10 lb Class riprap is sufficient for all cases except that one. However, to maintain an adequate factor of safety Class 30 lb stone is specified.

Significant geotechnical concerns exist with placing loose stone (rock riprap) in the weak marsh soils that exist in the outfall region. Bearing capacity failures will occur due to the weight of the stone and the weak strength provided by the foundation soils. Alternatives to address the very soft ground conditions are currently being explored. The protection within the Outfall section will be a 2-foot thick layer of 30 lb stone, over a 1-ft thick layer of No. 57 stone that is underlain by a high strength geotextile. Prefabricated, rock-filled geotextile mattresses are also being considered to provide bearing support to the overlying rock section. This solution may provide economy by preventing losses of material into the foundation soils. Another option may be to fill geotubes with the requisite 10 lb stone and install them as units in-the-wet. Alternatives are currently being investigated for an optimum configuration in collaboration with the CMAR to ensure constructability is carefully considered.

The backside of the Outfall extension will not require armoring for “normal” operating conditions because the velocities are less than 1.5 ft/s. However, armoring may be required to address the effect of surge and waves generated during hurricane conditions.

The armoring analysis was applied to the basin floor adjacent to the end of the outfall ramp. This area is assumed to have an average elevation of EL -4.0. The water depths and wave conditions for the 50-Year and 100-Year design conditions are identical to those calculated for the end of the ramp. These correspond to the last row of results provided in Tables CWRT-15 and CWRT-16.

6.7 Dredged Material Placement Areas (DMPA)

The DT performed a conceptual analysis of several dredged material placement areas during the preliminary phases of the MBSD Project. As part of the 60% Design Phase, CPRA asked the DT to further assess three general placement areas for dredged material placement. These general areas are referred to as Barataria Basin (**Section 6.7.1.1**), Fastlands (**Section 6.7.1.2**), and Existing Borrow Pits (**Section 6.7.1.3**). Alternative placement areas and methodologies within these three general areas were assessed in detail. **Section 6.7.2** details the 60% dredged material placement area (DMPA) design. In addition to numerous alternatives analyses, the DT collected topographical, bathymetric, magnetometer, and hazard investigation surveys along with geotechnical investigation and analysis.

6.7.1 Preliminary DMPA Location Alternatives Analysis

Based on the Excess Material Quantities 60% Snapshot (**Figure 6.7-1**), the DT was tasked with finding suitable areas to place a neat line volume of approximately 3.8 million cubic yards of excess dredged material from the excavation of the headworks, conveyance channel, and outfall transition feature (OTF) of the Mid Barataria Sediment Diversion. This quantity was derived from a summation of the unusable (scrape-down) cut and the excess usable cut. Scrape down material was deemed unusable for levee construction due to a high organic material content. When considering each of these location alternatives, pros and cons of topics like economics and construction feasibility were considered along with the inputs from CPRA, the CMAR, and other members of the DT.

	Location	Cut (Usable) (CY)	Cut (Unusable) (CY)	Fill (CY)	Excess Usable Cut (CY)
1	**Headworks**	**1,418,272**	**74,969**	**605,181**	**813,091**
1.1	Strip 12" of Top Soil	-	74,969	-	-
1.2	Excavate Riverside of MRL	238,871	-	-	-
1.3	Excavate - MRL to Transition	634,120	-	-	-
1.4	Excavate - Transition	509,381	-	-	-
1.5	Excavate - Final Grade	35,900	-	-	-
1.6	Backfill Basin	-	-	605,181	-
2	**Channel/Levee**	**2,921,217**	**376,258**	**1,153,800**	**1,767,417**
2.1	Strip 12" Channel limits only	-	323,258	-	-
2.2	Excavate - Ditches	-	53,000	-	-
2.3	Excavate - Channel	2,795,620	-	-	-
2.4	Excavate - Siphon Below Channel	66,000	-	-	-
2.5	NOV5A Removal	59,597	-	-	-
2.6	Fill- Design Grade	-	-	533,500	-
2	Fill0Overbuild	-	-	362,000	-
2.8	Fill- Settlement	-	-	214,500	-
2.9	Backfill Siphon	-	-	43,800	-
3	**Outfall**	**1,019,537**	**-**	**239,100**	**780,437**
3.1	Excavate - Outfall Channel	1,019,537	-	-	-
3.2	Fill - Design Grade	-	-	138,800	-
3.3	Fill - Overbuild	-	-	40,300	-
3.4	Fill - Settlement	-	-	60,000	-
4	**Hwy 23**	**19,566**	**70,226**	**94,376**	**(74,810)**
4.1	Muck Excavation	-	70,226	-	-
4.2	Excavate	19,566	-	-	-
4.3	Fill	-	-	94,376	-
5	**NOGC RR**	**-**	**12,467**	**26,349**	**(26,349)**
5.1	Muck Excavation - Main Line	-	2,346	-	-
5.2	Muck Excavation - Temp Line	-	10,121	-	-
5.3	Fill - Main Line	-	-	5,526	-
5.4	Fill - Temp Line	-	-	20,823	-
6	**Reservation Area**	**-**	**-**	**32,896**	**(32,896)**
6.1	Fill - Site	-	-	30,789	-
6.2	Fill - Building Pad	-	-	881	-
6.3	Fill - Boat Dock	-	-	1,226	-
Total (No Factors Applied)		**5,378,592**	**533,920**	**2,151,702**	**3,226,890**

Figure 6.7-1: Excess Material Quantities 60% Snapshot (AECOM – July, 2020)

6.7.1.1 Barataria Basin Placement

During the 15% BOD and 30% phases, the DT performed a conceptual analysis of several Beneficial Use of Materials (BUM) alternatives. A detailed description is included in the 30% Design Documentation Report (DDR). **Figure 6.7-2** shows the location of these BUM alternatives.

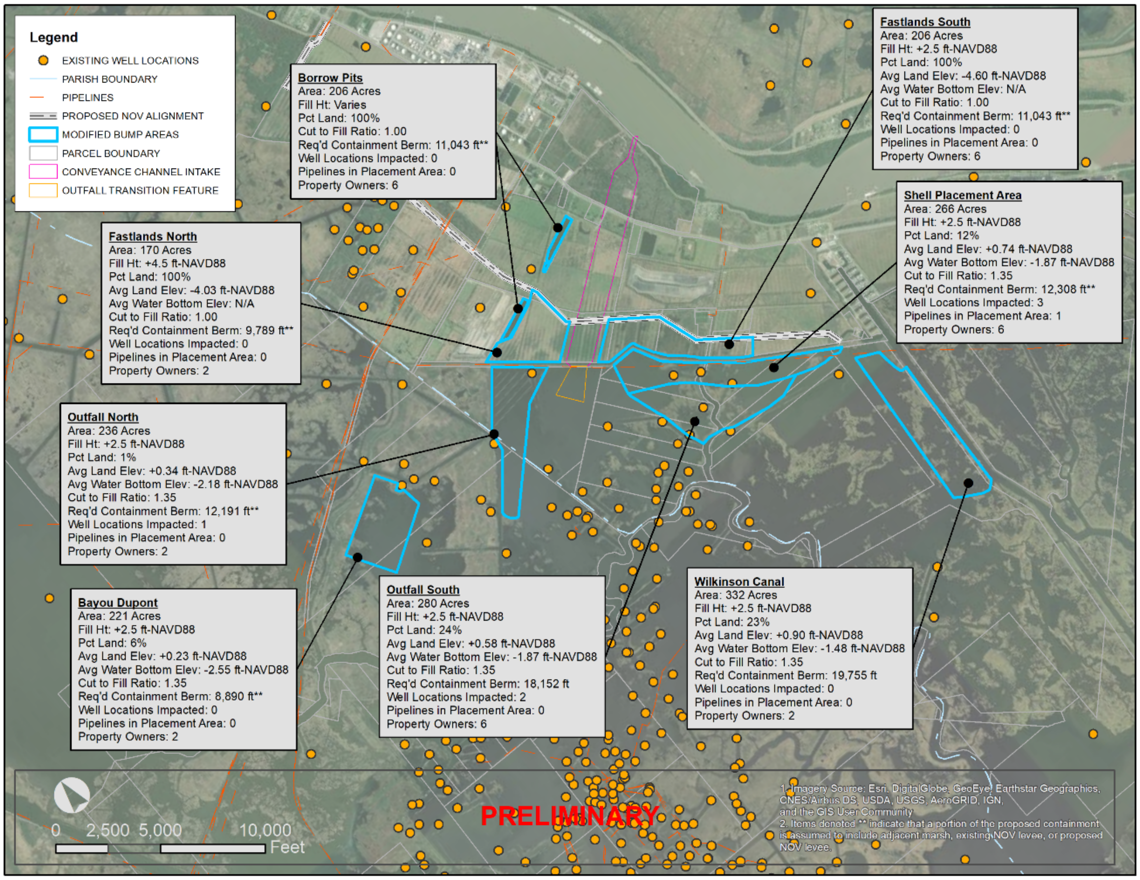


Figure 6.7-2: Beneficial Use of Material Placement Alternatives (Sept. 2019)

The two selected areas of investigation in Barataria Basin were named the MBSD Outfall North Marsh Creation Alternative and the Shell Pipeline Marsh Creation Alternative (**Figure 6.7-3**). Topographic and bathymetric surveys, magnetometer surveys, and anomaly investigations were conducted in these areas. These areas accounted for approximately five hundred acres for dredged material placement. Based on preliminary geotechnical investigations, however, the stability of the existing back levee came into question.

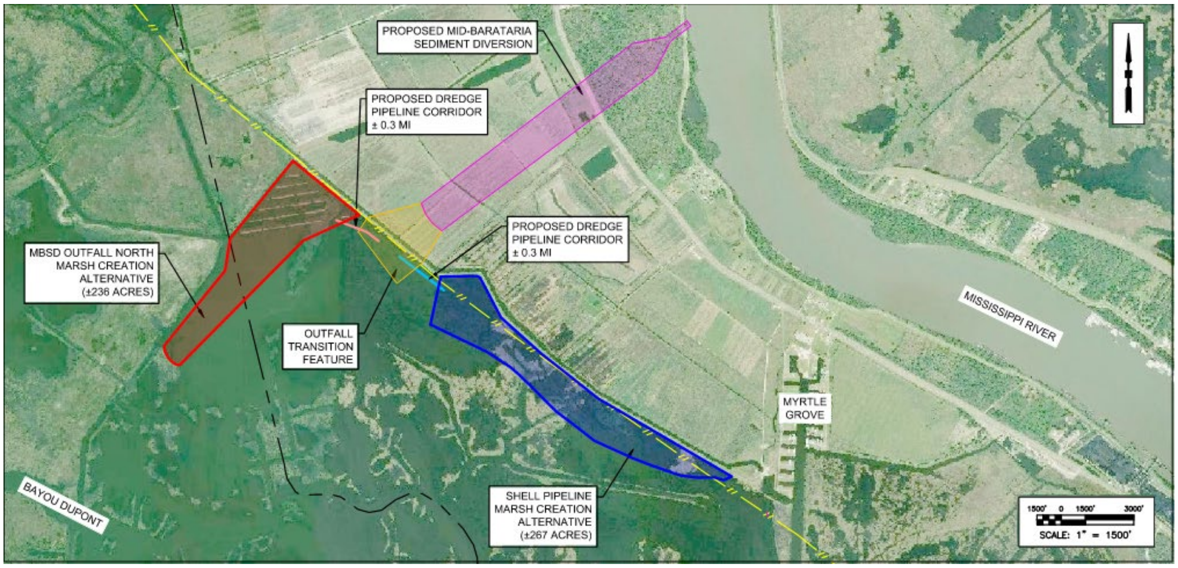


Figure 6.7-3: Barataria Basin Dredged Material Alternative Placement Areas

6.7.1.2 Fastlands Placement

The area of land located between the proposed NOV-NF-W-05a.1 levee and the existing back levee is considered the fastlands. Dredged material placement in the fastlands was considered for several reasons:

- Would bring area between existing back levee and proposed NOV-NF-W-05a.1 levee back to historic elevations
- Would provide protection for proposed NOV-NF-W-05a.1 levee
- Reduce need to maintain integrity of existing back levee

Several design alternatives were investigated for the fastlands. The fastlands were divided into four main areas as shown in **Figure 6.7-4**: Northwest Polder Area, Midway Cattle Ranch, LLC Operational Area, Fastlands North, and Fastlands South. The Fastlands North and Fastlands South Areas were investigated as DMPAs for the MBSD excess material. Two placement methods were considered: mechanical and hydraulic. Due to the construction of the MBSD and proposed activities in the Fastlands North and Fastlands South Areas, the Northwest Polder and Midway Cattle Ranch, LLC Operational Areas would be impounded. Four alternatives were analyzed for the Northwest Polder Area: (1) forced drainage improvements, (2) bottomland hardwood forest hydraulic restoration, (3) intertidal marsh creation, and (4) NOV-NF-W-05a.1 levee vicinity restoration. The Midway Cattle Ranch, LLC Operational Area was excluded in considerations for dredge placement or other alternatives.

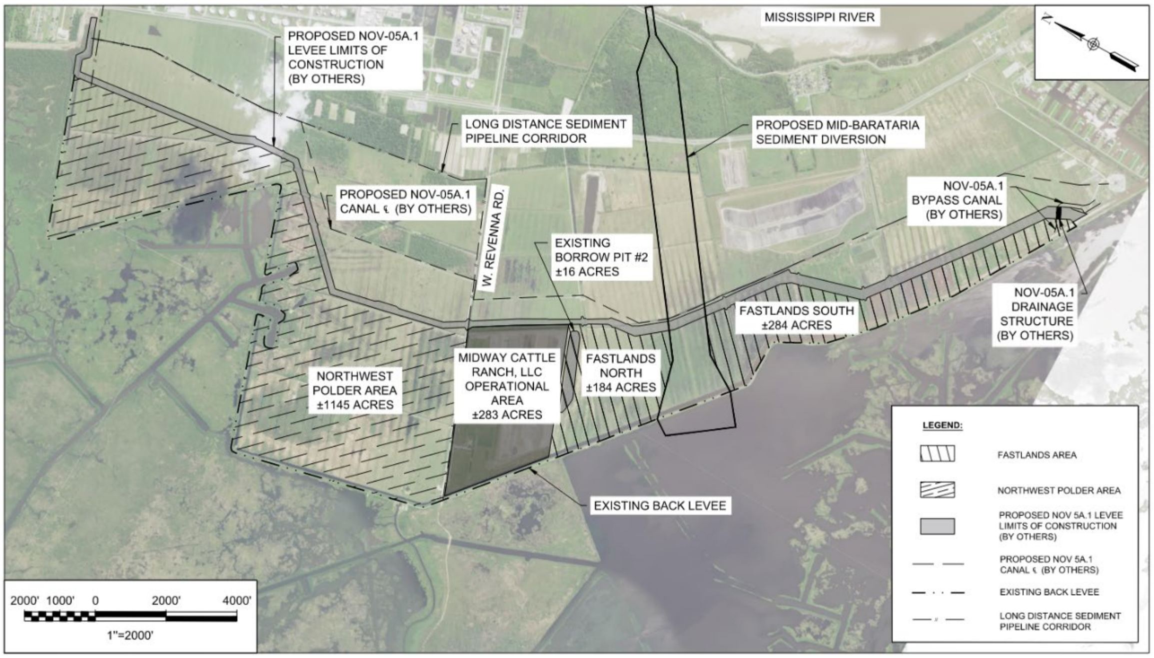


Figure 6.7-4: Fastlands Dredged Material Placement Areas Alternative

6.7.1.2.1 Fastlands – Mechanical-Fill Alternative

The DT investigated two separate methods for material placement within the fastlands. The first of these alternatives was mechanical placement. Discussions with the CMAR contractor indicated that excess material placed in the DMPAs by mechanical methods would be more feasibly used for haul roads and containment. Also, the maximum feasible distance to spread mechanically deposited material with a bulldozer is approximately 200 to 300 feet. Based on these recommendations, design for the DMPA in the fastlands area includes hydraulic fill slurry containment by use of a haul road or sidecast containment dike. The haul road was designed to be constructed using excess usable material and mechanically placed using dump trucks and shaped using bulldozers and excavators. It was assumed that off-highway dump trucks would be used; therefore, haul roads were designed with a 27-foot wide crown for two-way travel. The maximum mechanical placement quantity in **Table 6.7-1** (below) reflected designed haul road lengths, along with the spreading of 4 feet of fill over a 200-foot width adjacent to the haul road alignment along the basin side of the road. If necessary, approximately 1 million additional cubic yards of material could have been mechanically placed in the existing back levee canal.

6.7.1.2.2 Fastlands – Hydraulic Fill Alternative

It was assumed that much of the placement of material into the proposed fastlands area DMPA would be done hydraulically. Even if portions of the conveyance channel were dredged by mechanical methodologies (e.g. clamshell bucket dredge and excavators), the material may have been hydraulically pumped into the DMPAs as a slurry by depositing the material in scow barges or hopper barges and pumping hydraulically to the DMPAs from the barge. Everything west of the NOV-NF-W-05a.1 levee would be excavated hydraulically. This methodology would require containment and dewatering of the slurry.

Table 6.7-1: Fastlands Material Placement Capacities for Dredged Material Placement

AREA	ACRES	Haul Road length (ft)	Maximum Mechanical Placement (CY)	Maximum Hydraulic Fill Capacity (+2.0' NAVD88) (CY)
Fastlands North	183	6,468	256,317	1,782,946
Fastlands South	271	11,101	439,929	2,746,805
Borrow Pit #2	16	N/A	318,460	318,460
Total	470	17,569	1,014,705	4,848,211

6.7.1.2.3 Northwest Polder – Forced Drainage Alternative

The first alternative analyzed for the Northwest Polder was to utilize forced drainage improvements. The forced drainage improvements alternative design goal was to allow for drainage of the area without significantly altering the landscape of the NW Polder. This would have been accomplished similarly to existing conditions by using the back levee canal for drainage. There were two proposed alternatives for drainage: (1) hydraulic connection of the back levee canal to the new NOV-NF-W-05a.1 canal with culverts through the new NOV-NF-W-05a.1 levee and (2) forced drainage of the back levee canal directly to the Barataria Basin with a pump station. Hydraulic connection to the new NOV-NF-W-05a.1 canal included the use of culverts through the canal using existing drainage features within the NW Polder. An H&H study would have been required to properly size the culverts. Forced drainage via pump station(s) was another alternative that was analyzed. An H&H study would have been required for this alternative as well to properly size the pump(s).

The forced drainage improvements alternative may have needed improvements and continued maintenance of the existing back levee to maintain protection of the NW Polder area. Dredged material from the MBSD could have been beneficially used to strengthen the back levee.

There would have been costs associated with the new drainage features, however this alternative was the most economic. The alternative also did not require significant improvements or modifications to West Ravenna Road.

6.7.1.2.4 Northwest Polder – Bottomland Hardwood Alternative

The second alternative analyzed for the Northwest Polder was to create bottomland hardwoods by gapping the existing back levee and planting mechanically excavated terraces with trees. The design goal for this alternative was to convert the existing NW Polder area to a forested wetland that could be sustained in the natural tidal conditions within the upper Barataria Basin. Currently the existing natural ground within the Northwest Polder is at an elevation that ranges from EL -4 to EL -2 which puts the entire impounded area below sea level. The goal of the bottomland hardwood forest alternative was to elevate portions of the Northwest Polder to an elevation that could sustain a forested wetland without the back levee after the forest had matured.

The project area would require the existing back levee to remain in services until the trees were mature enough to be flooded naturally. Prior to degrading or gapping of the back levee, this alternative would have had to install water control structures to both flood and drain the impounded area. The inflow water control structure was proposed with gates in the northernmost portion of the Northwest Polder, near the Naomi Siphon discharge area. Drainage of the project area was proposed similarly to the forced drainage improvements scenario described in **Section 6.7.1.2.3** with either a gravity drainage control structure that discharges through the NOV-NF-W-05a.1 levee towards the Wilkinson Pump Station or discharge through a new pump station over the back levee into the Barataria Basin. **Figure 6.7-5** shows the alternative layout with water control structure locations.

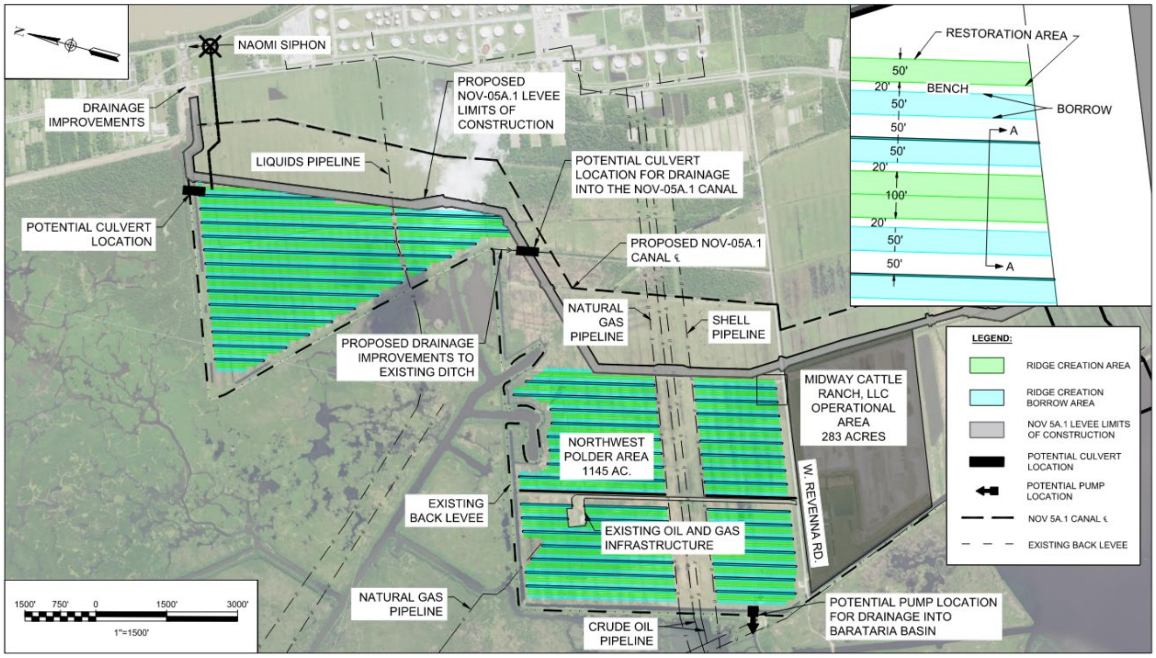


Figure 6.7-5: Bottomland Hardwood Forest Alternative

The alternative proposed to create a series of terraces within the Northwest Polder utilizing long reach amphibious excavators with a 60-foot boom. Terraces were proposed to be built to a target elevation of EL 0.0 NAVD88. No excavation or fill was proposed within any pipeline right of way. **Figure 6.7-6** shows a typical section of the proposed terraces and borrow areas.

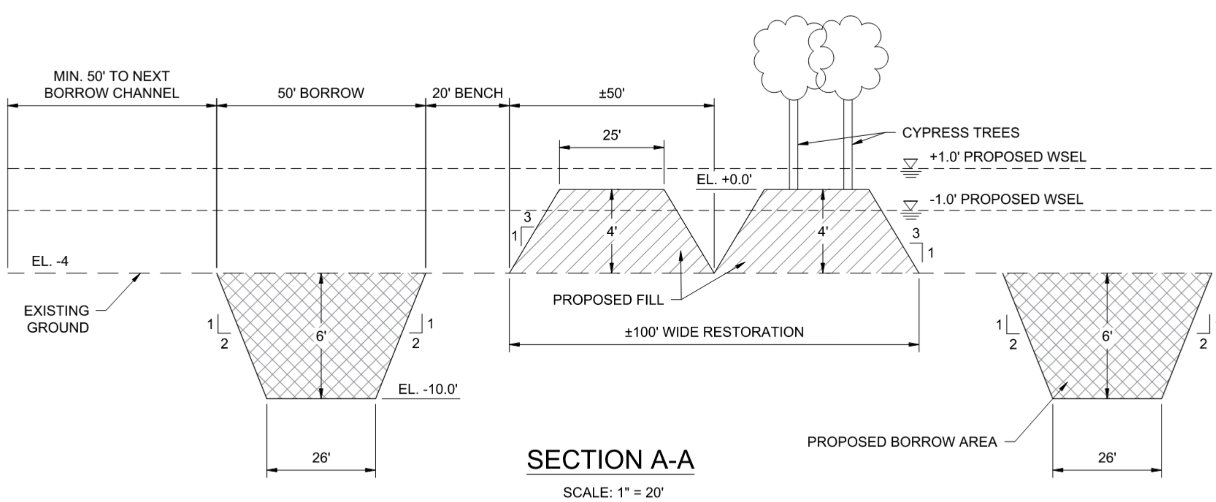


Figure 6.7-6: Bottomland Hardwood Forest (Typical Section)

6.7.1.2.5 Northwest Polder – Intertidal Marsh Alternative

The intertidal marsh creation alternative design goal was to build the Northwest Polder area up with dredged material to an elevation that would support healthy marsh once the area was hydraulically connected to the Barataria Basin by degradation or gapping of the existing back levee. Improvements and lifts of West Ravenna Road would be required to allow for hydraulic connectivity to Barataria Basin without flooding the Midway Cattle Ranch, LLC Operational Area.

The Northwest Polder area is approximately 1138 acres at an approximate average elevation of EL -3. **Table 6.7-2** presents the amount of material that would be required to fill the Northwest Polder area to given target elevations.

Table 6.7-2: CY Fill vs. Target Elevation for the Northwest Polder Area

CY Fill vs. Target El	
	NW Polder
Target EL = 0'	6,556,918
Target EL = +1'	8,400,117
Target EL = +2'	10,250,949

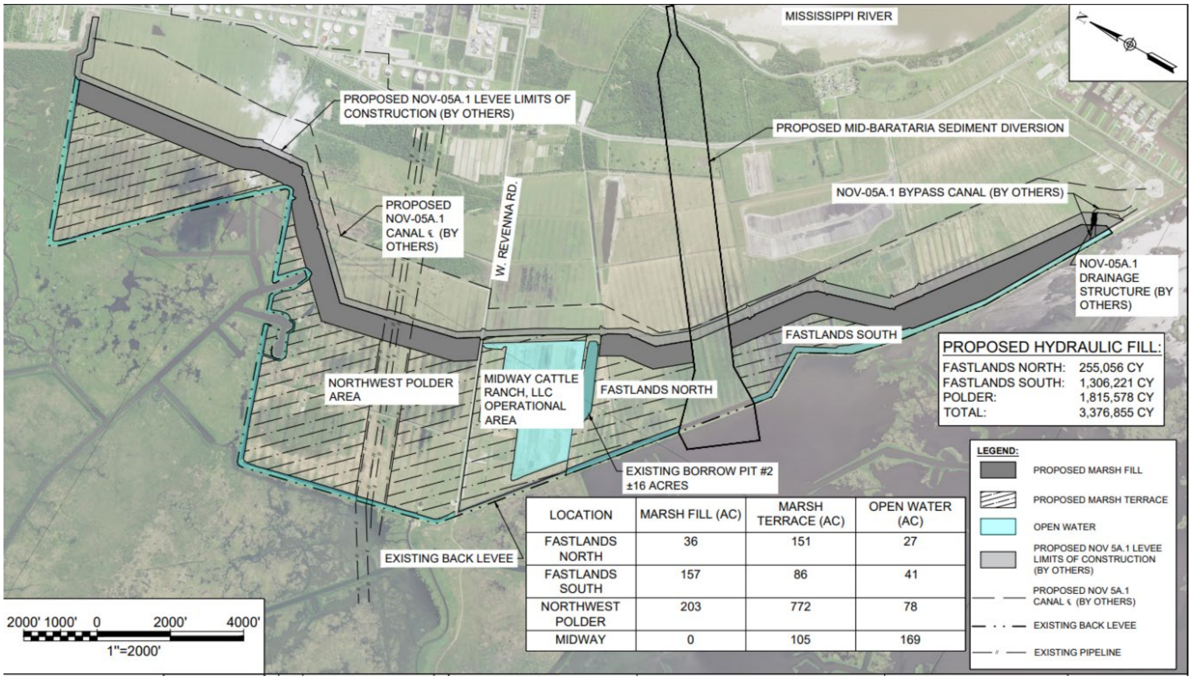
A potential mining source for this fill material is the existing Alliance Anchorage borrow area in the Mississippi River. The Alliance Anchorage borrow area spans from approximately RM 65 to RM 63.7 and encompasses a 120-acre area. This borrow area contains approximately 6.57 million cubic yards of material if dredged to an elevation of EL -90 NAVD88. Two additional borrow areas exist near the location of the LDSP. The Will's Point borrow area is located from approximately RM 66.9 - 66.4 and contains about 3 million cubic yards of material if dredged to an elevation of -90 ft NAVD88. The Alliance South borrow area spans from approximately RM 61.2 - 59.9 and contains about 3.9 million cubic yards of material if dredged to an elevation of -90 ft NAVD88. The Mississippi River Long Distance Sediment Pipeline (LDSP) project (state project number BA-0043 EB) borrow area modeling report indicates that it would take approximately one to three years for the Alliance Anchorage borrow area to refill to 90 percent capacity after it is mined (Moffatt Nichol, *LDSP 30% Design Report, Appendix H – Delft3D Borrow Area Modeling.* December 16, 2011 Draft). According to the report, if two or more of the borrow areas were dredged simultaneously, the rate of infill would decrease for those borrow areas that are downstream. The LDSP dredge pipeline corridor could be utilized for this alternative.

With the Midway Cattle Ranch, LLC operational area bordered to the north by the Northwest Polder area and the south by the Fastlands area, this alternative would have isolated the Midway Cattle Ranch, LLC operational area and would require the operational area to become an independent drainage system. The drainage of this area would need to be routed to the new NOV-NF-W-05a.1 canal or directly to the Barataria Basin.

6.7.1.2.6 Northwest Polder – NOV-NF-W-05a.1 Levee Vicinity Restoration Alternative

The NOV-NF-W-05a.1 levee vicinity restoration design, as shown in **Figure 6.7-7,** was suggested by CPRA for presentation to the US Army Corps of Engineers (USACE). The goal of this design alternative was to provide protection for the NOV-NF-W-05a.1 levee while eliminating maintenance of the existing back levee and drainage in the impounded polders. A marsh creation area was planned on the flood side of the proposed NOV-NF-W-05a.1 levee, with source material coming from the excess MBSD dredged material. Earthen containment dikes were planned for construction using either haul roads or side cast material.

Terraces would have then been constructed using side-cast material, and the existing back levee degraded upon project completion for tidal exchange.



LOCATION	MARSH FILL (AC)	MARSH TERRACE (AC)	OPEN WATER (AC)
FASTLANDS NORTH	36	151	27
FASTLANDS SOUTH	157	86	41
NORTHWEST POLDER	203	772	78
MIDWAY	0	105	169

Figure 6.7-7: NOV-NF-W-05a.1 Levee Vicinity Restoration Design Alternative

6.7.1.3 Existing Borrow Pit Placement

Three existing borrow pits (P600, P601, and P602) exist in the vicinity of the proposed Mid Barataria Sediment Diversion. The Plaquemines Parish Government (PPG) has expressed a desire to fill these pits. As shown in **Figure 6.7-8**, the three pits account for approximately 36 acres of fill. PPG's desire is that the post-construction grade of these pits is similar to existing ground, which is at approximately at EL -3.0 NAVD88. Discussions with the CMAR indicate that the existing borrow pits may be filled with excess unusable material, such as scrape-down material with high organic content.

It was brought to the attention of the DT that the existing borrow pits are currently being filled by others using material taken from the widening and deepening of the existing timber canal. This canal is one of the main drainage canals in the Northwest Polder and will be a large portion of the proposed drainage canal associated with the NOV-NF-W-05a.1 levee. As of June of 2021, the P602 borrow pit had been filled in. The remaining quantity of material that will be placed in the existing pits from this construction is unknown. Consolidation of material placed in the existing borrow pits are dependent on the source material and fill methods. Dewatering and slope stability will need to be taken into consideration during the filling operation.

Neither slope stability analysis nor consolidation tests have been performed by the design team for these existing borrow pits. A topographic and bathymetric survey of the existing borrow pits was performed by All South Consulting Engineering in May of 2019. The results of these surveys were used to create a volume surface to obtain a fill capacity.

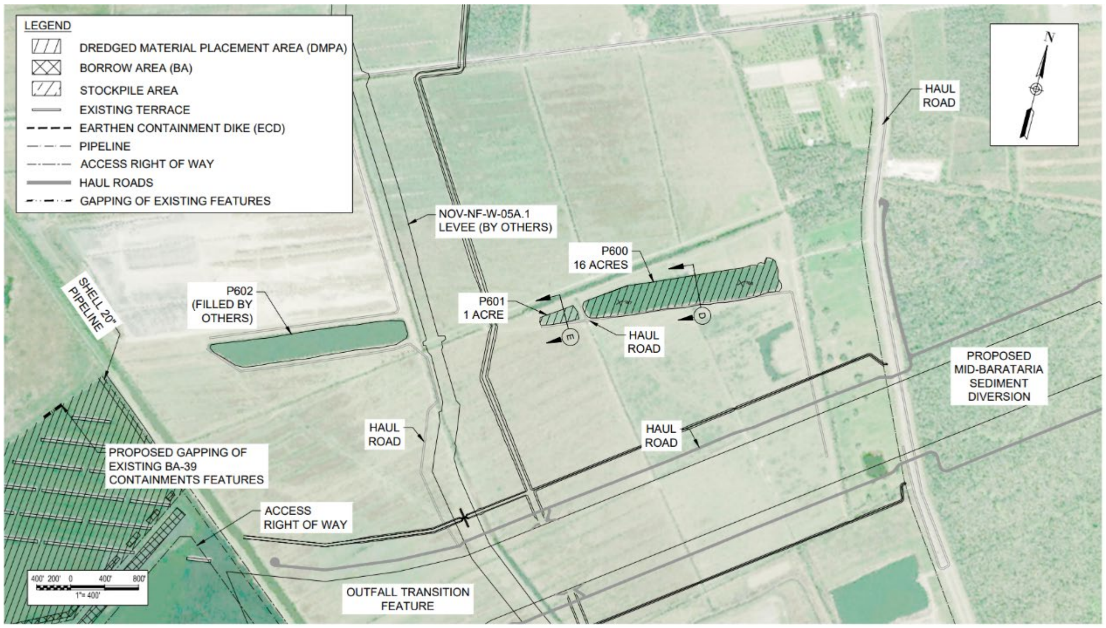


Figure 6.7-8: Existing Borrow Pits Plan View

6.7.1.4 Stockpile Placement (Alternate Bid Item)

Two potential stockpile areas lie adjacent to the MBSD alignment, as shown in **Figure 6.7-9**. One of these two areas may receive excess dredged material, which will then be donated to the landowner(s). Assuming a 5-foot fill height, the neatline fill volumes for each of the stockpile areas are shown in **Table 6.7-3.**

Table 6.7-3: Stockpile Area Capacity Quantities

ECD LENGTH & DMPA FILL CAPACITY QUANTITIES (ALTERNATIVE 1)							
Dredged Material Placement Area	Acreage (AC)	Neat Line Fill Volume (CY)	6'' Construction Settlement Volume (CY)	Total Fill Volume Capacity (CY)	Fill:Cut Ratio	Cut Volume (CY)	ECD (LF)
Alliance Stockpile	90	725,441	0	725,441	1	725,441	0
Midway Stockpile	124	1,000,000	0	1,000,000	1	1,000,000	0
Subtotal	**90 or 124**	**726K or 1M**	**0**	**726K or 1M**		**726K or 1M**	**0**

Access for these areas will be provided by haul roads, and the material will be mechanically placed. Placement of this material will be an Alternate Bid item, which is not included in the Base Bid. In addition to the two stockpile areas, the CMAR may stockpile additional material within the permanent MBSD ROW limits for future levee lifts.

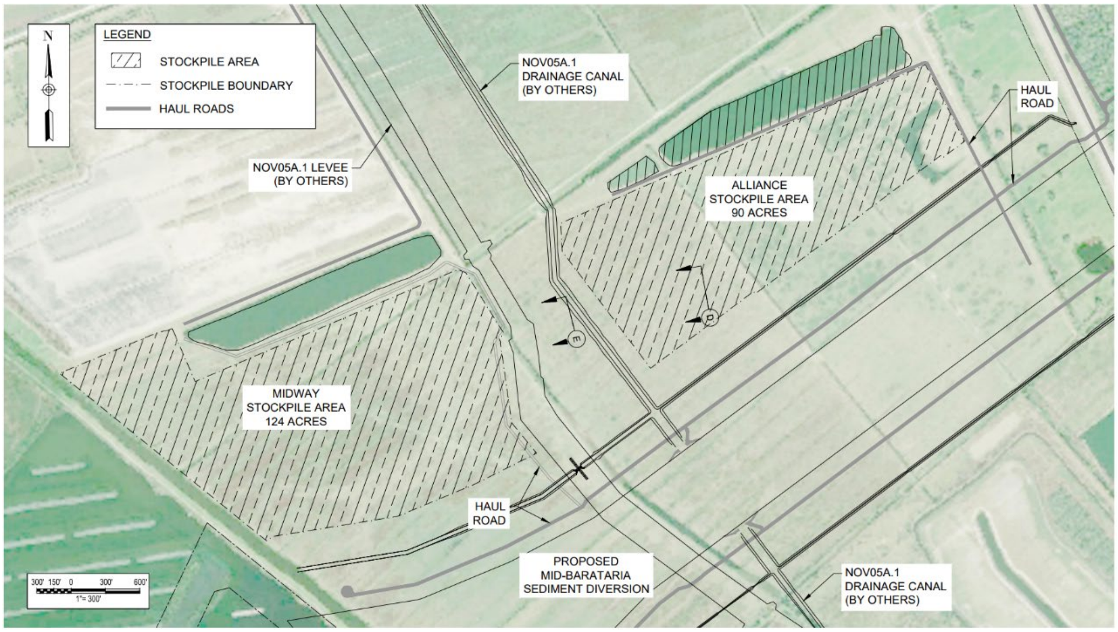


Figure 6.7-9: Dredged Material Stockpile Areas

6.7.2 DMPA 60% Design

During 60% design, DMPA alternatives were presented to the CPRA and the USACE. Coordination with these entities led to the selection of a combination of Existing Borrow Pit dredged material placement, the option for Stockpile Area dredged material placement, and Barataria Basin dredged material placement. Consultation with CPRA and USACE removed the Fastlands material placement design alternatives due to cost, coordination concerns, and construction feasibility. Coordination with the PPG led to the decision to fill in the remainder of the existing borrow pits that aren't filled by the NOV-NF-W-05a.1 construction. Additionally, discussions with landowners adjacent to the MBSD alignment led to the option to place up to 1 million yards of excess material onto either the Alliance or Midway Stockpile Area. The Stockpile area placement option is still in the preliminary design phase, and will be further detailed as design progresses. Once these options have been exhausted for placement, the Barataria Basin DMPAs will likely receive the remainder of the excess material.

The two Barataria Basin placement areas are named Outfall North and Outfall South, due to their vicinity to the OTF of the MBSD (**Figure 6.7-10**). After consultation with CPRA, filling Outfall North with excess material is the first priority after filling the existing borrow pits and stockpiling material for additional levee lifts. The next priority would be to fill the Outfall South section from North to South and East to West. This material placement could aide in steering the diverted water and sediment westward into Barataria Basin prior to turning and flowing south. In addition, the placed excess dredged material would act as a buffer for the existing back levee from wave action in the basin. The placement preferences of CPRA are contingent on associated costs and schedule anticipated during the construction process and are subject to change with updated quantities, placement methods, etc. during the design process. The two DMPA sections are designed using best engineering practices for containment, settlement, and material placement design, but they are not subject to the CPRA marsh creation design guidelines (CPRA, *Marsh Creation Design Guidelines*, Version MCDG1.0, November 15, 2017). The goal for the dredged material placement areas is a beneficial use of excess dredged material; however, they will not need to meet any post-construction criteria for wetland habitat creation.

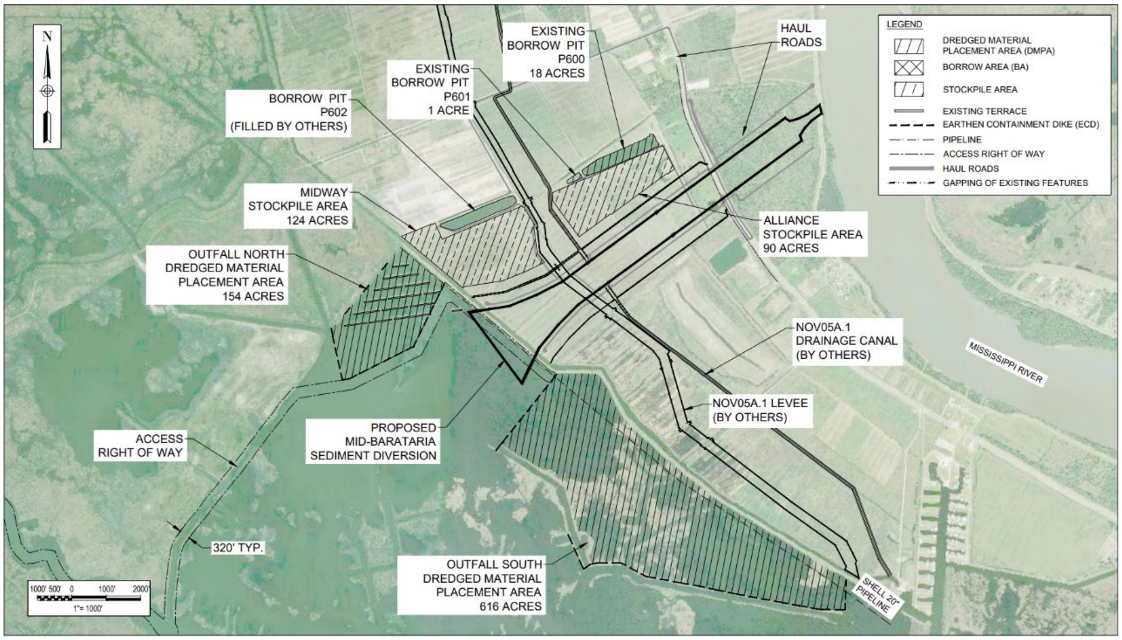


Figure 6.7-10: DMPA 60% Design Overall Plan View

The Outfall North and Outfall South DMPA locations are similar to the two Barataria Basin placement areas mentioned in **Section 6.7.1.1**. These two DMPAs were chosen for their vicinity to the MBSD and distance to transport excess dredged material. Additionally, the DT has conducted recent survey and geotechnical investigations in the areas. The Outfall North DMPA extents differ from preliminary designs due to landowner rights and changes in the planned access Right of Way (ROW). The Outfall South DMPA extents differ from preliminary designs to account for a reduction in volume capacity of the Outfall North DMPA, to best use existing site features for material containment, and to aide in guiding the flow out of the MBSD while staying outside of the operational 3 ft/s velocity contour in the basin.

Volume surfaces were created for the two DMPAs using the recent topographic and bathymetric survey data. The neat line capacities for these two areas are detailed in **Table 6.7-4.** In addition, the volume capacities for the existing borrow pits and the volume for placement in one of the stockpile placement areas are included in **Table 6.7-4.** Volume capacities detailed in **Table 6.7-4** disregard any potential settlement or lost material flowing out of the gaps in the partial containment. The preliminary capacity for these two areas is much greater than the Excess Material Quantities 60% Snapshot (**Figure 6.7-1**). This is to accommodate any potential bulking factors which may be applied to the hydraulically placed material. According to USACE recommendations, initially placed dredged material may have a bulking factor of approximately 1.5. The dredged material bulking factor does not account for fill settlement, foundational soil settlement, or construction sequencing of material placement. Applying the bulking factor to the Excess Material Quantities 60% Snapshot results in a required neat line volume capacity of approximately 5.7 million cubic yards. Factors that could impact the DMPA required volume capacity include but aren't limited to:

- The potential for the existing borrow pits to be filled prior to MBSD construction
- Potential loss of material due to partial containment
- Increased storage capacity due to foundation settlement
- Lower bulking factors
- Dredged material placement methods and sequencing

Table 6.7-4: 60% DMPA Neat Line Fill Capacities

DMPA Fill Capacities and Associated Quantities (Base)				
Location		**Capacity (CY)**	**Area (AC)**	**Containment Length (FT)**
1	Existing Borrow Pits	322,506	20	-
2	Outfall North DMPA (+3.0' Fill)	951,237	154	5,909
3	Outfall South DMPA (+3.0' Fill)	3,095,434	616	13,098
Total (No Factors Applied)		4,369,177	790	19,007

DMPA Fill Capacities and Associated Quantities (Alternative 1)				
Location		**Capacity (CY)**	**Area (AC)**	**Containment Length (FT)**
	Alliance Stockpile Area	725,441	90	-
or	Midway Stockpile Area	1,000,000	124	-
Total (No Factors Applied)		726K or 1M	90 or 124	0

As shown in **Figure 6.7-10**, the Outfall North and Outfall South DMPAs are currently designed with partial containment. This design decision was made after consultation with CPRA and the CMAR. Impacts to adjacent property owners, existing site features and conditions, costs, and existing back levee stability were considered for this design decision. Both DMPAs will be contained using the existing back levee. There are stability considerations associated with using the existing back levee for containment, which are further addressed in **Section 6.7.2.1.**

The Outfall North DMPA is bounded on the South and East by the Access ROW. Existing mudline elevations at this location is approximately EL -2.5 NAVD88. For access purposes, the CMAR plans to dredge within the permitted ROW to a bottom contour of EL -9.0 NAVD88. Currently, the DT plans to use this side-cast material for containment of the Outfall North DMPA where the planned containment is parallel to the ROW. Based on suggestions from the CMAR contractor, the earthen containment dike crest shall be no further than 85 feet from the bottom extents of the Access ROW. Any additional material required for containment of the Outfall North DMPA can be borrowed from within the DMPA. Any containment of the Outfall North DMPA that is not adjacent to the planned ROW will be constructed using interior containment. The Outfall North DMPA is bounded on the Northwest by the previously constructed Bayou Dupont Marsh Creation Project (CPRA Project BA-39). This marsh creation project was filled to EL +2.0 +/- 0.3 feet NAVD88 and had containment features constructed to EL +3.0 +/- 0.5 feet NAVD88. The current design allows for the decanting of the slurry through the adjacent Bayou Dupont Marsh Creation Project area. Gapping of the shoreline or old Bayou Dupont Marsh Creation earthen containment dike may be needed to allow for decanting.

Side-cast earthen containment in the Outfall South section will primarily be sourced from interior borrow. Special containment design considerations in the Outfall South DMPA include:

- Placement of material on top of/or filling the gaps of the existing spoil bank on the southwest portion of the DMPA
- Using sheet piles or alternative containment methods for containment where the containment alignment runs across an existing canal

- A 50-foot excavation buffer around the existing Shell Pipeline

Preliminary geotechnical investigations of the existing back levee indicate low factors of safety for global stability. To help mitigate these concerns, partial containment of both DMPAs is considered to reduce the hydraulic head applied by hydraulically placed dredged material. Geotechnical investigations into slip surface failure of the existing back levee are detailed in **Section 6.7.2.1.** Additional considerations include the use of the existing back levee for construction access. This would likely require improvements to the levee. CPRA and the DT plan to further investigate the risks and responsibilities related to the existing back levee during construction.

6.7.2.1 Geotechnical Investigation

Due to current design uncertainties, the geotechnical DT has recommended a cut to fill ratio of 1.5 for hydraulically placed dredged material. This will vary considerably with construction timing and sequencing. The earthen containment features are estimated to experience lower bulking factors between 0.9 and 1.1 because they will consist of mechanically excavated, fine-grained soils. These factors for ECDs are difficult to assess independently of the mud waves and lateral spreading that occurs when the ECDs are constructed. Additionally, all settlement curves were conducted assuming all material was placed to the construction fill height at one time. The 60% geotechnical estimates for settlement are conservative. As the construction sequencing becomes clearer in subsequent phases, the bulking factor for hydraulically placed material and settlement curves will likely become more certain.

The predicted settlement (orange curve) for the DMPAs with a fill elevation of EL +3.0 NAVD88 is approximately 3 feet according to **Figure 6.7-11**. This amount of settlement will occur over a 15-Year period after material placement, considering the existing mud elevation is at approximately EL -2.0 NAVD88. The predicted settlement for 1 and 5 years post construction is approximately 2.1 and 2.8 feet, respectively. This settlement estimation was made under the assumption that all material arrived at once on top of the existing mudline. As documented in Appendix C (Geotechnical Report), the predicted total settlement vs. elevation for the DMPAs with a fill elevation of EL +2.0 NAVD88 would be approximately 21 inches using the same assumptions after a 25-Year period. Estimated settlement vs. elevation at the 1 and 5 year periods for post construction is approximately 16 and 20 inches, respectively.

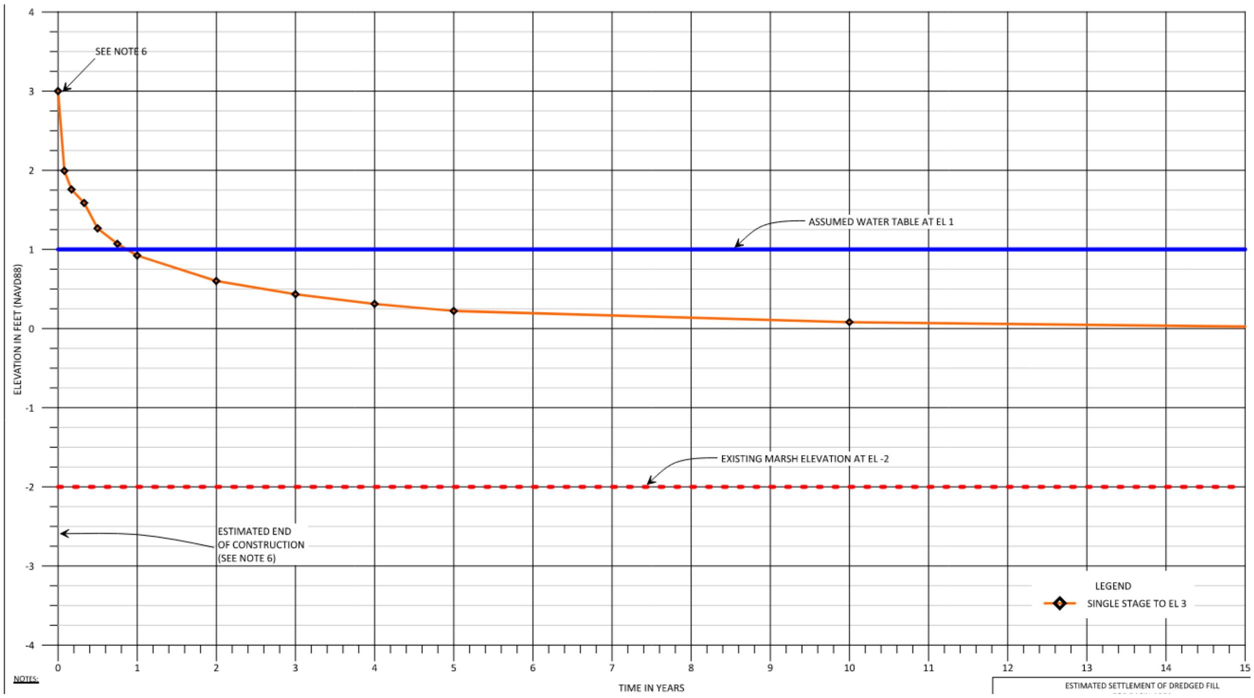


Figure 6.7-11: Time settlement curve for material fill to +3.0 ft. NAVD88

The expected settlement vs. elevation for earthen containment dikes constructed to EL +4.0 NAVD88 with 3H:1V side slopes are predicted to be about 1.5 feet of settlement after 25 years (**Appendix C**). Less settlement is anticipated than the DMPA area due to the limited width of the earthen containment dike.

Slip surfaces were analyzed using existing geotechnical data for the existing back levee. As shown in **Figure 6.7-12**, analysis of the back levee under existing conditions revealed the stability Factor of Safety is slightly above 1 (factor of safety of 1.08). As shown in **Figure 6.7-13**, results when considering DMPA fill placed to EL 3.0 in the basin reveal no impact to the existing back levee due to fill material (factor of safety of 1.06).

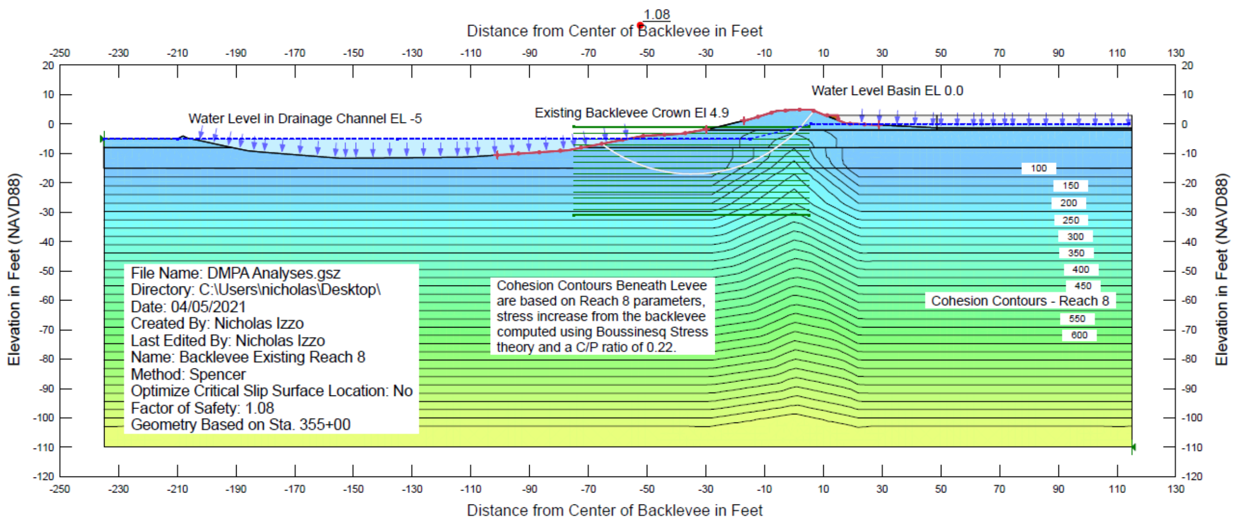


Figure 6.7-12: Existing back levee slip surface analysis for existing conditions

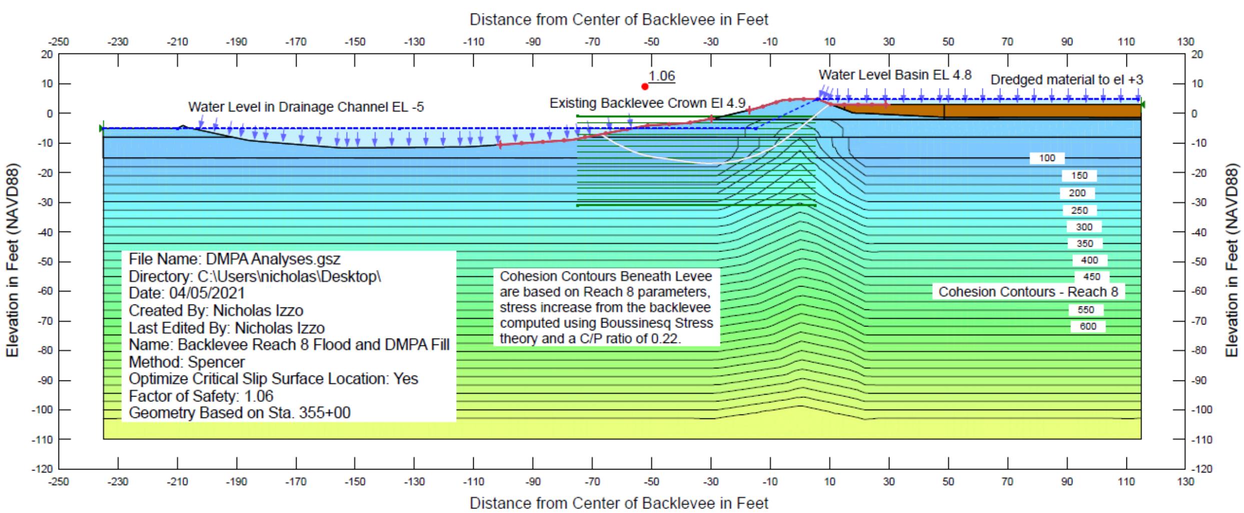


Figure 6.7-13: Existing back levee slip surface analysis for existing conditions with DMPA fill placed to el 3.0 in the basin

6.7.2.2 Survey Investigation

Two phases of topographic and bathymetric surveys were performed in Barataria Basin. The first round of surveys encompassed the original Barataria Basin dredged material placement areas, and the second round encompassed the additional area of Outfall South where there were data gaps. These surveys were used to create a volume surface for quantity calculations for the Outfall North and Outfall South DMPAs. The existing basin elevations are shown in **Figure 6.7-16**.

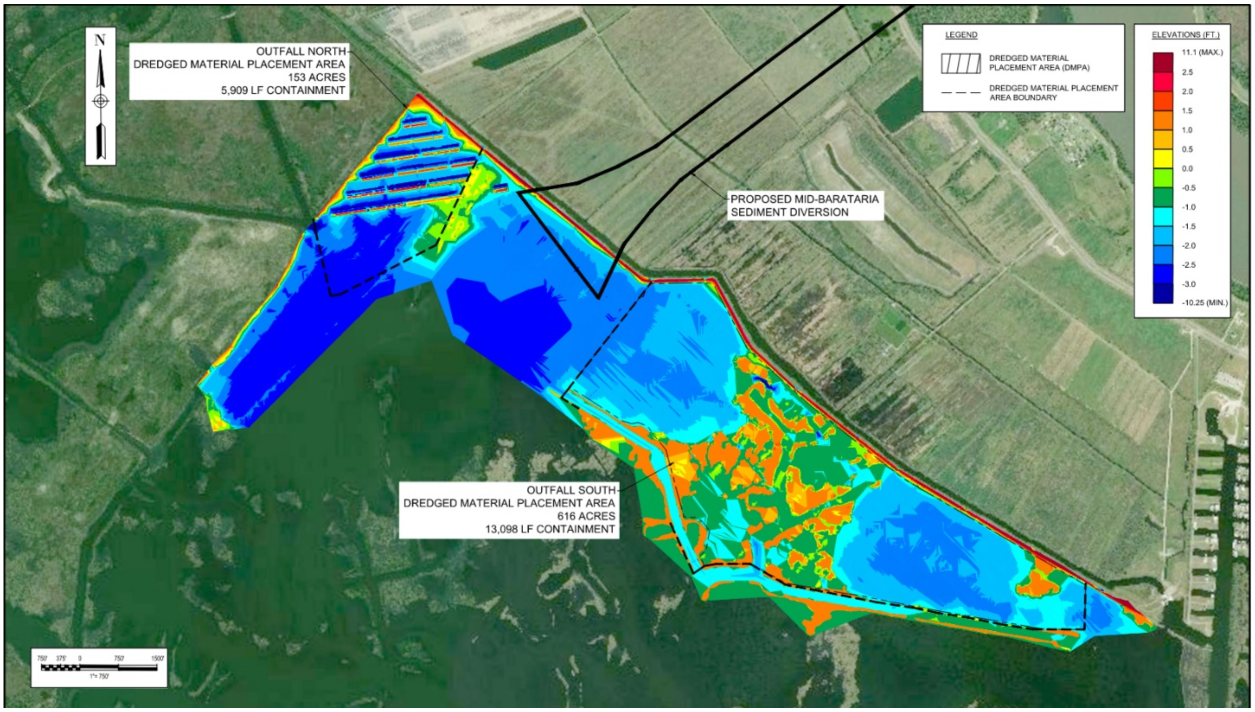


Figure 6.7-16: Dredged material placement areas contour map

Analysis of the geotechnical information, discussions of constructability with the CMAR, and the necessary DMPA volume capacity led to a design fill elevation of EL +2.0 NAVD88 with containment features for a crown elevation at EL +3.0 NAVD88. The breakdown for the available fill capacities associated with the 60% DMPA design features can be seen in **Table 6.7-5**.

Table 6.7-5: MBSD DMPA Capacities with Geotechnical Considerations

ECD LENGTH & DMPA FILL CAPACITY QUANTITIES (BASE)							
Dredged Material Placement Area	Acreage (AC)	Neat Line Fill Volume (CY)	6'' Construction Settlement Volume (CY)	Total Fill Volume Capacity (CY)	Fill:Cut Ratio	Cut Volume (CY)	ECD (LF)
P600	18.1	313,760	14,610	328,370	1	328,370	0
P601	1.4	8,746	1,147	9,892	1	9,892	0
Subtotal	**19.5**	**322,506**	**15,757**	**338,263**		**338,263**	**0**

Dredged Material Placement Area	Acreage (AC)	Neat Line Fill Volume (CY)	6'' Construction Settlement Volume (CY)	Total Fill Volume Capacity (CY)	Fill:Cut Ratio	Cut Volume (CY)	ECD (LF)
Outfall North	154	951,237	124,410	1,075,647	1.5	717,098	5,909
Outfall South	616	3,095,434	496,596	3,592,030	1.5	2,394,687	13,098
Subtotal	**770**	**4,046,671**	**621,006**	**4,667,677**		**3,111,785**	**19,007**

Dredged Material Placement Area	Acreage (AC)	Neat Line Fill Volume (CY)	6'' Construction Settlement Volume (CY)	Total Fill Volume Capacity (CY)	Fill:Cut Ratio	Cut Volume (CY)	ECD (LF)
Project Total	**789**	**4,369,177**	**636,763**	**5,005,940**		**5,005,940**	**19,007**

ECD LENGTH & DMPA FILL CAPACITY QUANTITIES (ALTERNATIVE 1)							
Dredged Material Placement Area	Acreage (AC)	Neat Line Fill Volume (CY)	6'' Construction Settlement Volume (CY)	Total Fill Volume Capacity (CY)	Fill:Cut Ratio	Cut Volume (CY)	ECD (LF)
Alliance Stockpile	90	725,441	0	725,441	1	725,441	0
Midway Stockpile	124	1,000,000	0	1,000,000	1	1,000,000	0
Subtotal	**90 or 124**	**726K or 1M**	**0**	**726K or 1M**		**726K or 1M**	**0**

Magnetometer and anomaly investigations were also conducted in Barataria Basin. The results of these investigations were taken into consideration for the 60% design and will be provided to the CMAR contractor.

6.8 Utility Relocations

The four primary utilities affected by various MBSD project features were as follows:

- Shell Pipeline – 20-inch crude oil pipeline located within the outfall transition feature
- Entergy Transmission – 115 KV transmission line located within the conveyance channel
- Entergy Distribution – 35KV distribution line located within Hwy 23 right of way and conveyance channel
- Plaquemine's Parish Water Line – 16-inch AC line located within Hwy 23 right of way

Additional utilities, CableOne and AT&T, are located on Entergy Distribution's infrastructure through an existing agreement.

Meetings with utility companies began in 2019 and were ongoing through 2021. Each utility company, with the exception of Plaquemine's Parish water line, will be designed by the utility owners and reimbursed by CPRA. The water line will be designed as a part of the LA 23 bridge relocation. Relocation agreements have been executed for design services with Shell and Entergy.

6.8.1 Shell Pipeline Company

Shell Pipeline (Shell) operates a 20-inch crude pipeline within the Barataria Basin which is located approximately 25 feet from the flood side toe of the existing back levee, and approximately 2,500 feet from the flood-side toe of the NOV-NF-W-05a.1 levee alignment. The Shell line will be affected by the dredging within the outfall transition feature and the outfall transition feature guide walls which will require a relocation of Shell's pipeline. Due to the depth of the outfall channel, the only option for relocation is a Horizontal Directional Drill (HDD). Lowering of the line or trenching in soft soils would not suffice. The limiting factor that will determine the minimum depth of the HDD is the pile tips for the braced sheetpile wall, which serves as the guide wall for the outfall. At a minimum, Shell will be required to relocate their line to a depth that is 20 feet below the braced sheet pile wall. Relocation of this line does not cross any federal projects or flood risk reduction structures. As an additional, redundant safety measure for the HDD, Shell is looking to grout the entry and exit of the HDD bore holes. Preliminary drawings prepared by Shell are included in the reference drawing set.

Shell will access the site from the Barataria Waterway utilizing the same primary route as the MBSD CMAR contractor. The access route is designed by the CMAR and has been approved by Shell for their use. Construction of the access route is currently planned by the CMAR prior to Shell's construction start. **Figure 6.8-1** shows the access route through the Barataria Basin for Shell's use.

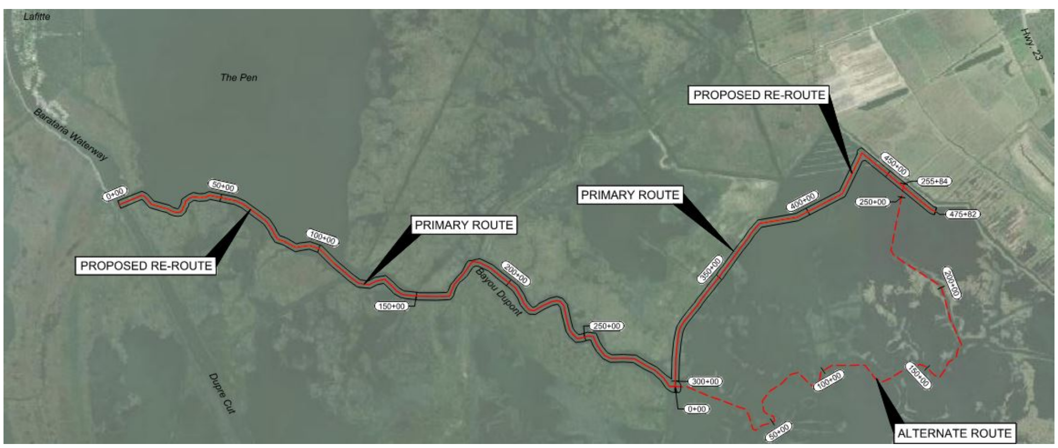


Figure 6.8-1: Barataria Basin Access Route

As a part of the Shell's design deliverables, they will provide CPRA and the MBSD Team with the following deliverables for review and acceptance into the MBSD Project.

- Construction Plans
- Permit Drawings
- HDD Calculations (including frac out calculations)
- Project Cost

6.8.2 Entergy Transmission

Entergy Transmission operates a 115 KV transmission line known as the Alliance to Happy Jack line that runs parallel to Hwy 23, outside of the DOTD right of way. The current layout of the transmission towers is within the footprint of the conveyance channel and will require relocation of the towers to a distance outside of the protected side toe of the conveyance levees. Entergy Transmission has provided the DT with their relocation plan, schedule, and cost. Based on the estimate received, it is anticipated that the relocation will take approximately 18 months to complete. **Figure 6.8-2** shows the conceptual layout for relocating Entergy's transmission line.

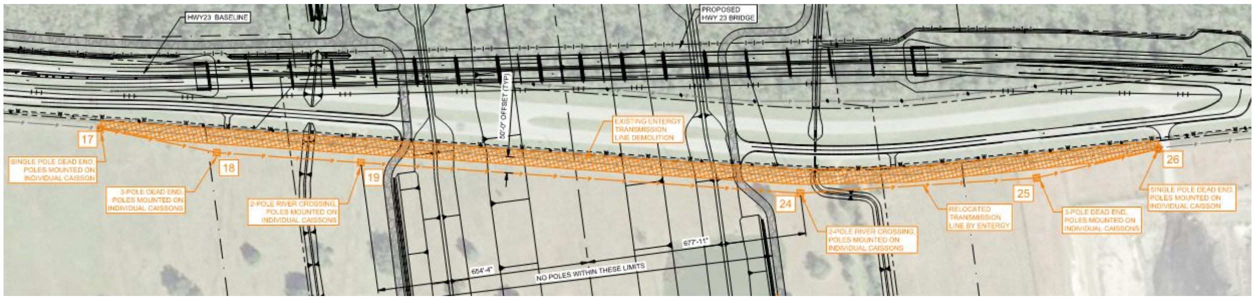


Figure 6.8-2: Entergy Transmission Relocation Layout

Entergy plans on utilizing a series of poles to connect into the existing infrastructure and build to a 2-pole river crossing. The relocation will span approximately 1,368 feet across the conveyance channel and is designed to have a minimum clearance of at least a 32 feet above the levees (currently designed with a clearance of 65 feet) and water surface (currently designed with a clearance of 43 feet with the predicted 75Kcfs operating water surface elevation of EL +5.0 at year 0). Additionally, transmission poles are designed to have a minimum clearance of 25 feet from the toe of the conveyance channel levees (currently designed at approximately 165 feet). Current transmission line dimensions are designed to account for future levee lifts along the conveyance channel. Entergy will provide CPRA and the MBSD Team with the following deliverables for review and acceptance. Preliminary drawings prepared by Entergy Transmission are included in the reference drawing set.

- Project Execution Plan
- Project Cost Estimate
- Construction Plans
- Permit Drawings

6.8.3 Entergy Distribution

Entergy Distribution has two distribution lines within LaDOTD right of way with one line east of Hwy 23 and the other line west of Hwy 23. Both lines will be impacted by the diversion's conveyance channel and the Hwy 23 bridge crossing. In addition to the distribution lines, Entergy has hanging agreements for both CableOne and AT&T which utilize the same power poles.

During early stages of design, it was anticipated that Entergy Distribution's relocation would be an HDD under the channel. Based on discussions with USACE New Orleans district, an HDD would need to follow USACE Guidelines for Installing Pipelines by Near Surface Directional Drilling Under Levees dated February 2021 which could put the HDD to depths near 300 feet below ground. Therefore, alternatives were evaluated for the relocation of Entergy's infrastructure. These alternatives included relocation along the LA 23 bridge and a reroute of Distribution's infrastructure towards the Mississippi River with an aerial crossing over the headworks. The reroute towards the headworks was the preferred alternative and moved forward into design. Preliminary drawings prepared by the DT and reviewed and approved by Entergy Distribution are included in the reference drawing set.

The relocation towards the headworks will require a phased approach for construction. Phase 1 will reroute both of Entergy's power poles from the east side of LA 23 to the west side of LA 23 to allow for construction of the LA 23 bridge. Removal of the power lines east of LA 23 and a couple of feeders for temporary power for construction services will also be included into Phase 1. **Figure 6.8-3** shows the layout for Phase 1 relocation.

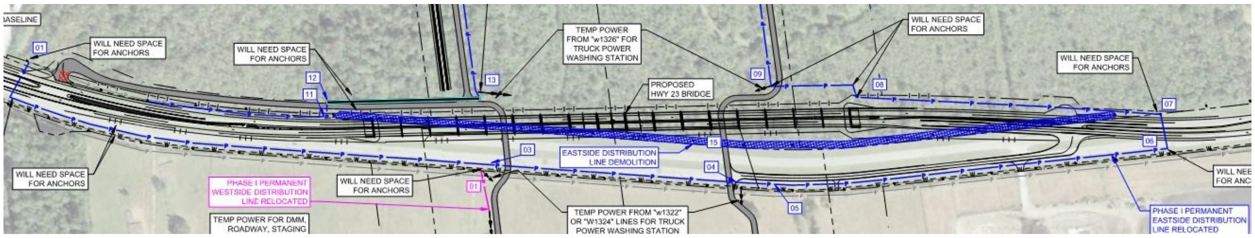


Figure 6.8-3: Entergy Distribution Phase 1 Relocation

Phase 2 of the relocation will be performed after LA 23 traffic has transferred to the new bridge and prior to the existing LA 23 highway being removed as a part of the MBSD conveyance channel construction. Relocation during Phase 2 will reroute Entergy's infrastructure from the western side of LA 23 towards the headworks. Due to construction of the headworks, Phase 2 will need to temporarily cross the future conveyance channel between the interim Mississippi River Levee and the conveyance channel borrow pit. Additionally, Phase 2 will provide power to the Barataria Basin for temporary construction needs and for permanent power to the siphon and the boat launch. **Figure 6.8-4** shows the layout for Phase 2 relocation for Entergy Distribution.

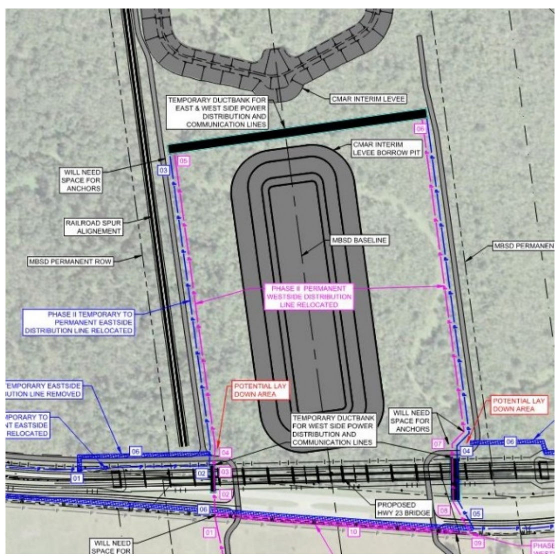


Figure 6.8-4: Entergy Distribution Phase 2 Relocation

Phase 3 is the final phase for Entergy Distribution which includes the relocation of power lines over the MBSD Headworks and the removal of the temporary underground power for Phase 2. Phase 3 relocation is shown in **Figure 6.8-5**.

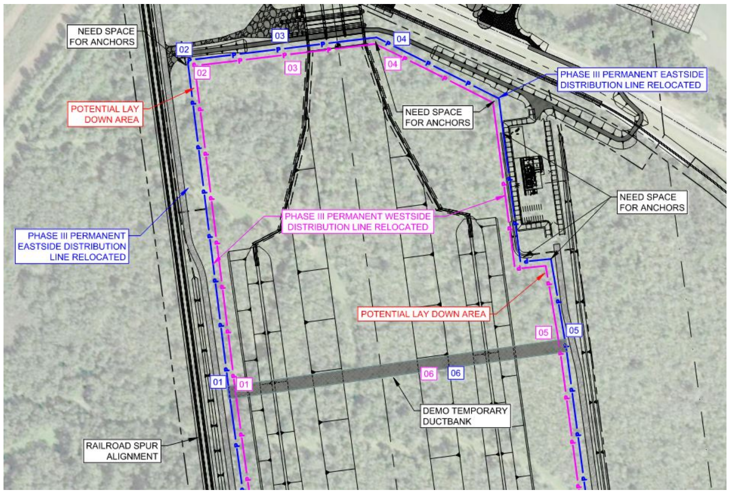


Figure 6.8-5: Entergy Distribution Phase 3 Relocation

As a part of the Entergy Distribution relocation, both CableOne and AT&T will also relocate their communication lines onto the Entergy poles utilizing an agreement between their respective companies. In areas where power is to be laid underground, CableOne and AT&T will relocate their lines underground similar to Entergy.

6.8.4 Plaquemines Parish Waterline

Plaquemines Parish Government (PPG) currently operates a 16-inch diameter AC waterline that runs on the west side of Hwy 23 within the LaDOTD right of way. Inframark operates and maintains the utilities for PPG including the waterline in conflict which will be impacted by the conveyance channel. During our kickoff meeting on February 21st, 2019 with PPG and Inframark, the DT and CPRA discussed options for relocation which included either hanging the relocated waterline under the proposed Hwy 23 Bridge or relocating it under the conveyance channel via HDD methods. Per CPRA's agreement with PPG, the waterline will be designed and constructed as part of the Hwy 23 relocation. The relocated waterline will be attached to the underside of the Hwy 23 Bridge.

The waterline will be replaced with a 20-inch diameter line that will hang under the proposed shoulder along the LA 23 bridge. Additionally, a second hanger will be installed under the LA 23 bridge to allow for a future 20-inch diameter line to be added by the parish. **Figure 6.8-6** shows a typical detail of the proposed waterline relocation along the LA 23 bridge. Additional details can be found in the 60% drawing set.

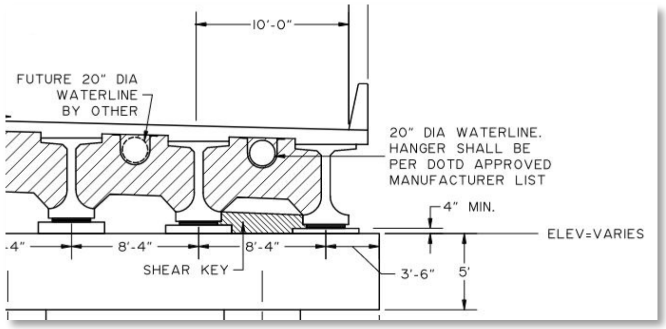


Figure 6.8-6 – Water Line Detail

PPG requested that CPRA act as the owner's agent for design, permitting, and construction of the waterline relocation. PPG will provide review and acceptance of the work and will allow CPRA's CMAR contractor to perform the work. During construction of the waterline, PPG will require permission to perform inspections and testing on the waterline.

7. HWY 23 ROADWAY AND BRIDGE

7.1 General

Hwy 23 is a north-to-south state highway, approximately 74 miles long, that serves both Plaquemines and Jefferson Parishes. It is also known as Belle Chasse Highway, Lafayette Street, and the West Bank Expressway at different locations along its length. Between Belle Chasse and Venice, the highway is the main thoroughfare along the western bank of the Mississippi River. This route provides the only access in and out of Plaquemines and lower Jefferson Parishes and is a State of Louisiana evacuation route during hurricane season. Within the area of the MBSD project, the existing highway is an at-grade, four-lane rural arterial asphalt composite roadway with 4-foot wide inside shoulders, 10-foot wide outside shoulders and a 42-foot wide depressed grass median.

Hwy 23 is within the jurisdiction of LaDOTD District 02, as shown in **Figure 7.1-1**. The red outline marks the location of the MBSD project, which is south of the Phillips66 Alliance Refinery and north of the town of Ironton. Since a portion of Hwy 23 will be intersected by the MBSD conveyance channel and guide levees, the existing highway will be removed and replaced with an elevated bridge structure. At the location of the bridge, property access is maintained by use of access roads along the west side of the highway. A construction access road along the east side of the relocated highway will be provided for construction operations during the construction of the diversion channel with the intent of transferring ownership of the road to the current owner of the property.

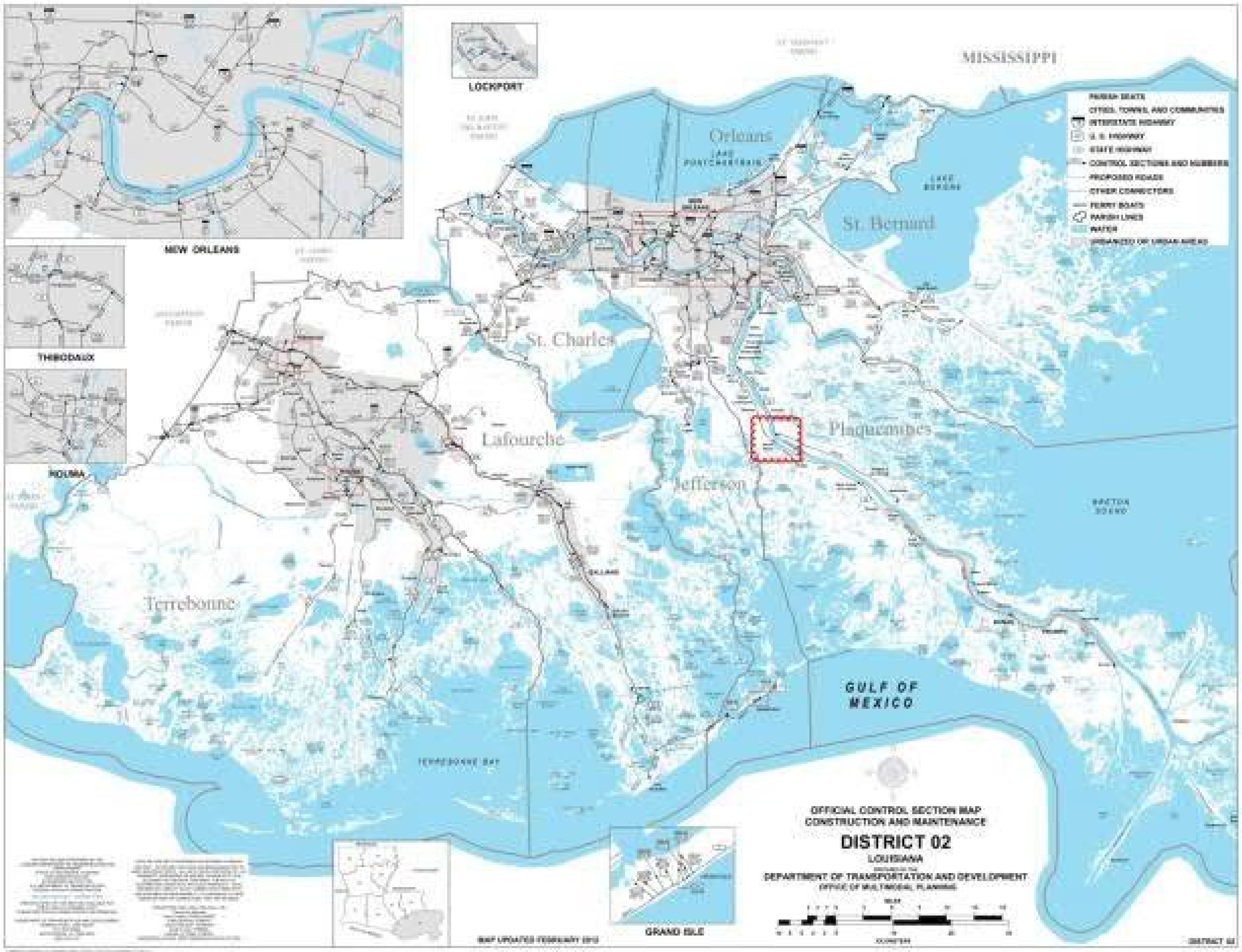


Figure 7.1-1: Hwy 23 Location Map

Because Hwy 23 is a State highway, the design of the Hwy 23 Bridge and roadway is designed in accordance with LaDOTD standards and specifications. The 90% Preliminary Plans for the proposed Hwy 23 Bridge were reviewed by LaDOTD in March 2021. The comments from that review are being addressed concurrently with the development of the 60% Final Hwy 23 Bridge Design.

7.2 Traffic Study

The DT conducted a traffic analysis report for the MBSD Project Area which included Hwy 23 and all of the intersections, commercial driveways, and median openings along Hwy 23 from Ravenna Road to the Plaquemines Parish Access Road. The study included traffic counts, peak hours, and a safety study to ensure that the proposed Hwy 23 Bridge will meet the capacity of future road demand. As a requirement to fulfill the LaDOTD Project Delivery Process, meetings with adjacent landowners have been held to seek comment on the proposed median opening relocations and access changes as a result of the relocated highway and diversion channel bridge crossing.

7.3 Roadway Design

Hwy 23 is classified as a Rural Minor Arterial with a design speed of 65 mph for the highway and 30 mph for local roads. A full listing of the applicable standards and design criteria is included in **Appendix A.** Roadway improvements include new asphalt paving that will transition from the existing divided highway typical section to the bridge typical section on either side of the channel. At-grade access roads on the north and south sides of the channel provide access to adjacent properties and the levee maintenance roads and supporting infrastructure that will be constructed along the diversion.

7.4 Bridge Design

The bridge will consist of precast prestressed girders and piles along with cast in place caps and decking. The elevation of the bridge allows for a minimum of 5 feet above vertical clearance between the top of the guide levee floodwall and the low chord of the bridge, for a minimum of 16.5 feet above vertical clearance between the crossing haul roads and the low chord of the bridge, as well as 25 feet of vertical clearance between the conveyance channel's maximum water surface elevation (EL 2.0) and the low chord of the bridge. A full listing of the applicable standards and design criteria is included in **Appendix A.** The bridge deck will also structurally accommodate two 20-inch diameter water lines owned by Plaquemines Parish which will be hung underneath the bridge superstructure. Further discussion of the waterline relocation is discussed in **Section 6**.

7.5 Maintenance of Traffic

A preliminary sequence of construction has been developed that utilizes the southbound pavement to maintain both directions of traffic during the construction of the bridge. The bridge itself will not require phased construction since traffic will be maintained west of the bridge construction. Localized shifts will be required to maintain the tie in. A full definition of the maintenance of traffic incorporating the CMAR contractor input will be shown in the 60% Hwy 23 Bridge Final Plans.

7.6 Bridge Scour Analysis

In accordance with LaDOTD policy, a Bridge Scour Analysis was performed to evaluate potential scour at the bridge piles within the limits of the conveyance channel. The report has been reviewed and accepted by CPRA and LaDOTD. As a result of the review, a 50 lb riprap will be placed to line the diversion channel within 100 feet on each side of the bridge.

8. RAILROAD (R/R) BRIDGE

8.1 General

The project includes a railroad bridge over the intake structure to allow the New Orleans Gulf Coast Railroad (NOGC) to continue rail service to current and future customers. The bridge generally follows the current horizontal alignment of the existing railroad with a slight shift away from the MRL to avoid pile penetrations of the MRL toe.

The approach bridges are comprised of embankments to an approximate EL 10 feet NAVD88, at which point they transition to pile-supported bridge spans. Typical pile bents consist of three HP 14x117 piles tipped at approximately EL -150 feet NAVD88 supporting a reinforced concrete pile cap. Pile bents are spaced at 35 feet. Every eighth pile bent will include a double row of 30" diameter pipe piles, tipped at approximately EL -105 supporting a reinforced concrete pile cap. The approach spans consist of prestressed concrete box girders supporting ballasted track.

The vertical alignment of this alternative allows the bottom of the rail to clear the top of the intake structure wall (EL 20.35 feet NAVD88). The bridge spans over the U-frame intake structure consist of steel girders supporting ballast decks. Two interior concrete piers and the outer walls of the U-frame intake structure support the steel girders. The intake walls turn perpendicular to the bridge structure where it crosses the walls to avoid conflict with the railroad ties, keeping the elevation of the bridge (and therefore the overall bridge length) as low as possible. One span includes a longitudinal deck joint so that it can be disassembled and removed in the event that a floating maintenance plant requires access to the control gates.

A second parallel bridge spans over the U-frame structure to provide flood response access across the intake structure. Access ramps from the levee crowns provide continuous access across the intake structure for wheeled vehicles. This auxiliary bridge has the same structural design as the RR bridge and can accommodate a future second track if needed, but only the spans necessary to accommodate access across the intake structure will be built initially. This configuration minimizes the initial construction cost while providing a route for flood response access along the MRL.

Temporary spur tracks are included to maintain rail service during construction.

8.2 Design Features

8.2.1 Track

The track conforms to Union Pacific standards. The track generally consists of 115# rail with 7-inch ties and a minimum of 8 inches of ballast.

8.2.2 Embankments

The top of rail elevation for the existing tracks is approximately +5 feet NAVD88. The 60% design assumes that an earthen embankment is sufficient to accomplish an elevation increase to +10 feet NAVD88. The side slopes of the embankment are 3H:1V. The embankment terminates at a reinforced concrete abutment that transitions to the pile-supported portion of the bridge.

During 60% design, the design team began to consider lightweight fill for the embankment. In the 90% design phase, the design team will determine whether lightweight fill can be used to increase the elevation of the embankment, thus shortening the length of bridge required.

8.2.3 Approach Spans and Bents

As the bridge approaches the U-frame intake structure, the bridge consists of pile bents supporting prestressed concrete box girders.

Three HP 14x117 piles tipped at approximately EL -150 feet NAVD88 support each pile bent. The piles support a 15’ L x 3’-6” W x 2’-8” D reinforced concrete pile cap. The spacing of the pile bents is 35 feet. Every eighth bent includes a double row of 30” diameter pipe piles, tipped at approximately EL -105 NAVD88 for additional resistance to longitudinal forces.

Each span of the approach bridge consists of two 7’ W x 2’-6” D prestressed concrete double box girders. The girders rest on the pile caps on elastomeric bearing pads.

The girders support a ballasted railroad deck. Reinforced concrete curbs contain 8 inches of initial ballast under 7-inch ties. The structural design accounts for a maximum depth of ballast of 30 inches. The out-to-out dimension of the curbs is 18 feet.

A few asymmetrical bents near the vehicular access bridges support two concrete spans between the two railroad bridges in addition to the railroad bridge girders. The concrete spans allow wheeled vehicle to drive from the first railroad bridge to the auxiliary bridge, providing access for flood response.

8.2.4 Spans Over Intake Structure

The bridge crosses the U-frame intake structure at a skew of approximately 58°. To keep the crest of the bridge as low as possible (and therefore the overall length of the bridge as short as possible), the outer walls of the intake turn perpendicular to the railroad bridge to avoid conflict with the railroad ties.

The spans over the U-frame intake structure consist of seven steel plate girders, a ¾-inch steel ballast plate, MC 18x42.7 steel ballast curbs, and end plates for ballast retention. The girders vary in length from 79’-6” to 88’-9 1/16” to accommodate the skew. “Steps” in the outer walls and two interior concrete piers support the bridge spans over the intake structure on elastomeric bearing pads.

The girders support a ballasted railroad track. The steel bottom plate and channels contain 8 inches of initial ballast under 7-inch ties. The structural design accounts for a maximum depth of ballast of 30 inches. The out-to-out dimension is 18 feet.

A second parallel bridge spans over the U-frame structure to provide a route for flood response across the intake structure. This auxiliary bridge has the same structural design as the RR bridge and can accommodate a future second track if needed, but only the spans necessary to accommodate access across the intake structure will be built initially. This configuration minimizes the initial construction cost while providing a route for flood response along the MRL.

The Design Team gave particular consideration to the hydraulic criteria for bridges. The CPRA approved an elevation of +20.35 feet NAVD88 for the outer wall of the U-frame structure. This elevation is based on the 50-Year hurricane design elevation. To maintain risk reduction to this elevation, the Design Team considered this elevation to be the minimum bottom of rail elevation to avoid penetrating the wall. Union Pacific guidelines for main line tracks require the low chord of a bridge to be at or above the 50-Year flood event and the subgrade (2’-3” below top of rail) to be above the 100-Year flood event. Controlling hydraulic elevations (in ft NAVD88) for the project are as follows:

- Mississippi River Standard Project Flood: +12.65
- Authorized Mississippi River Levee crown elevation: +16.4
- Future 50-Year hurricane design elevation: +20.35
- Future 100-Year hurricane design elevation: +24.85

Considering that this segment of the NOGC will serve very few industries south of the intake structure and that the 50-Year design hurricane would submerge the surrounding track on both sides of the intake, the Design Team determined that the riverine elevations were more appropriate for design. NOGC agreed that this approach meets the intent of the standards and is practical and appropriate for this project. The railroad bridge is designed so that the tracks clear the intake walls at EL +20.35. The low chord elevation remains above the Mississippi River Standard Project Flood.

8.2.5 Removable Span

On both bridges over the intake structure, the second span from the north includes a longitudinal deck joint to allow for relatively easy disassembly and removal in the event that a floating maintenance plant must access the intake control gates. The design of the removable span is identical to the other spans with the exception of this longitudinal joint, which reduces the pick weight of the span in the event of removal.

Work time required for removal is approximately 2 days. Replacement time is approximately 2-3 days. The general process for removal is:

1. Close span to rail traffic.
2. Remove rails and ties.
3. Remove ballast.
4. Unbolt deck joint connections.
5. Remove nuts from anchor bolts on bearings.
6. Install lifting devices.
7. Lift out span one side at a time.
8. Set span pieces in storage location.
9. Emplace fall protection on open ends of adjacent spans.

Replacement is the reverse of the above procedure.

The design team considered an open deck design for the removable span to expedite removal. However, open deck bridges require a different maintenance regime than ballast decks. Since removal will be a rare occurrence (on the order of once per 15 years), the design team chose to avoid introducing one span of open deck and thus complicating regular operations and maintenance. NOGC concurred with this decision.

8.2.6 Vehicular Access Bridges

Reinforced concrete access bridges provide access for wheeled vehicles from the MRL crown to the auxiliary bridge over the U-frame intake structure, providing flood response access across the diversion structure.

The geometry of the bridges is based on an SU-30 design vehicle to represent the largest likely emergency vehicles that would need to use the bridge – military cargo trucks and ambulances. The design speed is 5 mph. The design vehicle, speed, and low frequency of use allow the access bridges to be generally perpendicular to the MRL and railroad bridge and to have relatively high slopes (up to 7%) and grade breaks (up to 1.5%).

The structural design of the bridges is based on an H-20 traffic load. At the MRL, the outer monoliths of the proposed T-wall provide the foundation for the bridges, with the bridge girders resting on the T-wall slab. A pile bent parallel to the railroad bridges supports the ends of the bridges closest to the railroad bridge. Three HP 14-117 piles with tip elevation -124 ft NAVD88 support each 38’x2.5’x2’ reinforced concrete bent. The superstructure of each bridge consists of five LG-36 prestressed concrete girders supporting an 8.5” thick reinforced concrete slab.

8.2.7 Spur Tracks

The spur tracks allow continued rail operations during construction. One spur track provides the same length of track as exists in the pre-construction condition to accommodate NOGC’s current operations. The second spur track allows NOGC to deliver materials to the CMAR for this project.

8.3 Design Criteria and Loading Conditions

8.3.1 General

The railroad tracks and bridges comply with the requirements of the 2016 American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering* and pertinent requirements of the Union Pacific Railroad’s *Technical Specification for Construction of Industrial Tracks*. The access bridges comply with AASHTO LRFD Bridge Design Specifications and the LADOTD Bridge Design and Evaluation Manual (Latest Edition). The sections below provide general design criteria information, and design criteria document (included as an appendix) includes additional details.

8.3.2 Geometry

The railroad design speed is 25 mph on the main line track(s) and 15 mph on the spur tracks. Horizontal curves are designed using the chord definition with a maximum degree of curvature of 7° 30'. Vertical grades do not exceed 1.5% .

The geometry of the access bridges is based on an SU-30 design vehicle with a speed of 5 mph.

8.3.3 Loads

The Cooper E-80 train configuration governs the design loading for the railroad. A minimum 200 psf live load is applied where rail and road loadings are not applied. Wind loads are applied in accordance with the 2016 AREMA *Manual for Railway Engineering* Chapters 8 and 15.

The structural design of the access bridges is based on an H-20 traffic load.

8.3.4 Materials

Steel conforms to the requirements of ASTM A709 Grade 50W. Cast-in-place structural concrete has a minimum 28-day strength of 4,000 psi or higher. Precast concrete for box girders has a minimum 28-day strength of 6,000 psi. Reinforcing steel consists of billet steel bars conforming to requirements of ASTM A615 Grade 60. Prestressing strand shall be ½” diameter, Gr. 270 low-relaxation strand conforming to the requirements of ASTM A416.

8.4 Summary

The project includes a railroad bridge that generally follows the current railroad alignment in order to preserve rail service to current and future clients. A second, parallel bridge over the intake provides flood response access along the Mississippi River Levee alignment and can accommodate a future second track. Access bridges allow wheeled vehicles to access the flood response access along the auxiliary bridge from the MRL crown. Spur tracks provide rail service during construction. Future design phases will include additional details including structural details, grade crossing details, more detailed drainage design, and refinements to the spur track design.

9. SECONDARY SITE FEATURES

9.1 General

Building Design

Structural: The reservation site will include several buildings on pile supported slab on grade at assumed EL 11 (BFE=10) listed below:

Applicable Publications

American Association of State Highway Traffic Officials (AASHTO)
AASHTO LRFD Bridge Design specifications 4th Edition Dated 2007 (for bridge designs after 2007)
HB-17 Standard Specifications for Highway Bridges (for bridge designs prior to 2007)

American Concrete Institute (ACI)
315 Details and Detailing of Concrete Reinforcement
315R Manual of Engineering and Placing Drawings for Reinforced Concrete Structures
318 Building Code Requirements for Structural Concrete and Commentary
530 Building Code Requirements for Masonry Structures

American Institute of Timber Construction (AITC) Construction Manual

American Forest & Paper Association (AF&PA)
NDS National Design Specification for Wood Construction with Supplement
SDPWS AF&PA Supplement Special Design Provisions for Wind and Seismic

American Institute of Steel Construction (AISC)
325-05 Steel Construction Manual, Thirteenth Edition
341 Seismic Provisions for Structural Steel Buildings, including Supplements
360 Specification for Structural Steel Buildings

American Iron and Steel Institute (AISI)
NAS North American Specification for the Design of Cold- Formed Steel Structural Members, including Supplement
General Standard for Cold-formed Steel Framing-General Provisions
Standard for Cold-formed Steel Framing-Header Design Lateral
Standard for Cold-formed Steel Framing-Lateral Design Truss
Standard for Cold-formed Steel Framing-Truss Design
WSD Standard for Cold-formed Steel Framing-Wall Stud Design

American Society of Civil Engineers (ASCE)
7 Minimum Design Loads for Buildings and Other Structures
24 Flood Resistant Design and Construction

International Code Council (ICC)
IBC International Building Code 2015
NFPA 101 Life Safety Code 2015
ADA and ABA Accessibility Guidelines, DOJ 2010

SBCCI SSTD Standard for Hurricane Resistant Residential Construction

Metal Building Manufacturers Association (MBMA)
MBSM Metal Building Systems Manual
MBMA Metal Roofing System Design Manual

Precast/Prestressed Concrete Institute (PCI)
MNL 117 Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products
MNL 120 PCI Design Handbook – Precast and Prestressed Concrete
Mnl-122 Architectural Precast Concrete

Steel Deck Institute (SDI)
DDM03 Diaphragm Design Manual
No. 30 Design Manual for Composite Decks, Forms Decks and Roof Decks
UFC 1-200-01 Design: General Building Requirements
UFC 3-310-01 Design: Structural Load Data
UFC 3-310-01 Seismic design for Buildings

Truss Plate Institute
TPI 1-2002 National Design Standards for Metal Plate
Connected Wood Truss Construction

Steel Joist Institute

Design standards and codes:
AISC current ASD method, ACI, IBC2016

9.2 Reservation

The Diversion Structure will require support personnel and physical plant facilities to operate and maintain the structure and gates. Maintenance and daily operation of the project throughout its useful life will require necessary support buildings with an administration office, operation shops with all necessary mechanical/electrical apparatus, standby emergency power equipment, a pole shed vehicle/tractor/boat storage area, access roadways, levee access (roadways) and a boat launch/ramps. This will be accommodated by a separate security contained area with all above including parking for and access to all areas of the project which is hereby referred to as the “reservation” area and is to be located on the south side of the Gated Diversion Structure. The reservation area is approximately 3,000 feet north Hwy 23 between Ironton and Myrtle Grove in Plaquemines Parish. The design criteria for buildings structures will be per ASCE 7-Minimum Design Loads for Buildings and Other Structures and road and drainage structures per LADOTD standards.

The site layout for the diversion reservation area and support facilities will be designed as a 12-inch thick limestone aggregate surface with 12 inches (minimum) sand subbase with geogrid and geotextile fabric and will allow for ease of construction during levee, structure and channel maintenance activities. Reservation area total dimensions will be approximately 500 feet by 150 feet and will require approximately 6.5 feet of fill embankment to bring the final parking/drive grade from existing (EL 3.5 +/-) to approximate final EL 10.5 around the buildings and to Base Flood Elevation (BFE) of EL 10 at perimeter (low point) with final slab for buildings at EL 11.0. The entire reservation fill area will be pre-surcharged to EL 12.0, and wick drains will be installed to accelerate consolidation as was determined by the geotechnical analysis to final settlement at EL.10.0. Location of the reservation area in relation to the construction cofferdam and excavation is taken into consideration as the large amount of fill required to back fill the excavation will have an effect on the design of the reservation buildings foundations if the footprint extends under the building foundations as it was determined to shift the entire site further to the south. The CMAR haul road will be reestablished along the south R/W as possible for the permanent access roadway at 24 feet width with a parallel drainage ditch. The river end (east) will have a 20 feet width roadway section then connecting to a turnout for the (south bound) access roadway at BFE (EL 10) to cross RR spur at approximately MRL Sta. 1122+75 and over the MRL to access the CMAR and permanent reservation site boat launch (see **Section 9.6**) in the river. The northbound access road from the turnout will provide access to the gated structure via a 24 feet wide, 2-inch thick asphalt wearing surface, 3-inch asphalt binder course on 12-inch stone aggregate and 12-inch compacted sand subbase.

Also, included will be subsurface drainage structures (catch basins, drop inlets, RCP culverts approximately 15 inches to 36 inches diameter) through the parking/roadway areas to a drainage ditch outfall then connecting to LA 23 ditch drain system, utility service such as sewer (treatment plant and lift station of 7,000 GPD as per the building and occupant requirements on pile supported 6" thick slab at EL 0.83), with 8-inch gravity, 4-inch force main and 15-inch A2000 outfall to ditch. Also, an adjacent 6" thick 16'x32' pile supported slab for the buildings generator, fuel tank and platform with 8 chain link fence and 12' gates. Water service line to tie in with parish water distribution system via min. 8-inch diameter lines (4,000 feet +/- of PVC-900) with a minimum of 4 fire hydrants located around the roadway perimeter, power distribution throughout (via local power company and building requirements per section 16, telephone/cable etc., security fencing (10' and 8'- chain link fencing and 2 - 12 foot long electronic gates at each entrance area), parking lot (light pole standards) lighting through limits of the parking and access roads and separate building lighting, 18 parking spots with 2 ADA spots, 4-foot sidewalks and appropriate signage. The radii and turning movements and curb design assumption are using WB 40 tractor trailer and a 40 turning radius. Reservation access roads design assumption to be with 2-inch asphalt wearing on 3" binder and 12-inch stone aggregate and 12-inch compacted sand subbase with swale drainage from Hwy 23.

A garbage/dumpster area will be provided at the north end of pole shed with an 8 feet high wooden fence with 12 feet and 3 feet gates.

9.3 Administration/Maintenance Building

The Administration/Maintenance building is a combination of an administration office building and a maintenance building.

Administration Office Building: The administration building is a single-story office building with metal wall and roof panels comprising 4,160 square feet. The administration building structure is designed as a moment frame using STAAD in accordance with AISC and MBMA outlined requirements. A pre-engineered metal building is not feasible because it is connected to the maintenance building which will house a 5-ton overhead crane, and there is potential for vibrations from crane operation cracking finishes. The building's steel frames are designed for a maximum horizontal deflection of ½ inch. Crane girders to support crane operation are designed for a maximum deflection of ½ inch as outlined by the crane manufacturer's specifications.

Structural frames have a 1 inch diameter rod bracing installed below the roof in three (3) bays to control horizontal movements. The roof system consists of GA 8 inch Z purlins, with 24 GA prefinished standing seam metal roof panels rated for a 171 mph maximum wind speed.

The wall system will consist of 8” GA girts with 24 GA prefinished metal wall panels rated for a 171 mph maximum wind speed.

Structural steel columns shall be attached to the concrete foundation with 1 inch thick base late over 1½ inch grout pad, with minimum four (4) F1554 anchor bolts.

The concrete foundation shall consist of both 8-inch thick and 10-inch thick structural concrete slabs, supported by reinforced concrete grade beams founded on concrete pile caps with 12 inch square precast concrete pilings. All structural concrete will have 4,000 psi compressive strength at 28 day break.

ADMINISTRATION / MAINTENANCE BUILDING

Design Loading:
Live Roof 20 PSF
Dead Roof 10 PSF
Live Slab 250 PSF with a 35% impact load

Wind Loading was determined utilizing, Digital Canal Wind Analysis 10.1
Design Wind Speed 178 MPH
Structure Risk Category IV, Exposure D
MWFRS (Directional Procedure)

Steel design was performed utilizing Bentley Systems software, STADD. Connect structural analysis and design; as a three dimensional Moment Frame Structure, with uniform member loads and nodal loads applied to the structure to represent design loading.
Max Deflection of Columns and Beams were limited to L/360
Max Deflection of Crane Beam was limited to L/480

Concrete design was performed utilizing Digital Canal software Concrete Beam Design 4.1. Slabs were designed as one-way load with the strip method. Concrete beams were designed for the actual induced moments and shears resulting from the design loading; for, the heaviest loaded member (worst-case).

9.4 Pole Shed Building

Pole shed building is a boat storage shed made up of rigid frames. It is a rectangular pre-engineered metal building. The building area is 3,220 square feet.

Pole shed is 37’ 6” wide with an interior column in the middle to reduce the construction cost. The eave height is 20’ 0” and has standing seam metal panels attached to 8-inch Z girts. The roof has 12:1 slope on both sides. The building will have vinyl backed insulation in roof and walls. The front of the building has all five bays open for boat storage and other three sides have side walls with 8-inch Z girts and 24 gauge pre-finished wall panels. Lateral load resisting system consists of 1” diameter tie rods in three bays. Roof bracings will be provided as required by Metal Building Manufacturers. The building slab is set at an elevation of 11.00. Gutters and downspouts on both sides of the roof are provided. The foundation system consists of an 8” thick cast-in-place concrete slab over 10 mil vapor barrier and supported on grade beams and columns supported on concrete footing. The grade beams and footings are supported by 12-inch square pre-cast concrete piles.

POLE SHED BUILDING

Design Loading:
Live Roof 20 PSF – For Modeling Only
Dead Roof 10 PSF – For Modeling Only
Live Slab 100 PSF

Wind Loading, for Modeling Only, was determined utilizing, Digital Canal Wind Analysis 10.1
Design Wind Speed 178 MPH
Structure Risk Category IV, Exposure D
MWFRS (Directional Procedure)

Modeling to determine Steel Column Reactions was performed utilizing Bentley Systems software, STADD. Connect structural analysis and design; as a two dimensional Moment Frame Structure, with uniform member loads and nodal loads applied to the structure to determine column reactions for foundation design.

Concrete design was performed utilizing Digital Canal software Concrete Beam Design 4.1. Slabs were designed as one-way load with the strip method. Concrete beams were designed for the actual induced moments and shears resulting from the design loading; for, the heaviest loaded member (worst-case).

9.5 Bridge Crane

The maintenance building has a 5 ton crane to pick up the stored material/equipment. The crane has a rail to rail span of 22’ 6”. The rail height is set at 18’ 0” from the top of slab (operating level). The pendant station is at 5’ 0” off of the top of slab. The crane is Class D top running double girder crane with rotating axle wheels manufactured by American Crane and Equipment Corporation (ACE Co.). Crane rail is 40# rail which is installed on the crane girder. Crane stops are made up of wide flange beam which are welded to top flange of the crane girder. Rail stops are installed at both ends of the crane girders. Rail is stopped at the inside face of the crane stops. Maximum wheel load is 11,700 lbs. Power supply is 460/3/60. Bridge wheel loads are based on a maximum trolley weight of 2,900 lbs.

9.6 Launch Ramps and Boat Docks

The launch ramps are designed so that the greatest amount of excavation occurs above the water line, with the underwater portion of the launch ramp closely matching the mudline topography as possible. This reduces the required cut or fill in the submerged/submersible zone and minimizes potential impacts and issues.

The launch ramp at the east end of the project in the Mississippi River is designed to access/inspect the structure during high river stage. The top of launch ramp is set at an EL 10.50 to match the roadway from the levee toe on flood side. The ramp has a turnaround at an EL 10.50 for the boat trailer to back up and launch on the ramp. The bottom of ramp is set at EL 5.50 to match the batture. This ramp is 25’ wide and made up of 12” thick cast-in-place concrete over 12” thick compacted limestone over geotextile fabric. This ramp is constructed with cast-in-place concrete up to EL 5.0 and pre-cast concrete panels to EL -5.50.

Since the Mississippi River low stage is at EL 2.00, a cast-in place concrete ramp can be built during the low river stage. The ramp will be built at 10:1 slope. Riprap will be provided at the end and sides of the 25’ wide ramp to protect it from erosion.

9.7 West Launch Ramp and Boarding Pier at Barataria Back Bay

The west launch ramp and boat dock are designed to access/inspect the structures in the Barataria Bay side of the project. A turnaround at the top of existing levee is provided to launch the boat on trailer. Turnaround consists of an asphalt surface over compacted limestone base. The top of turnaround is set at an elevation of 8.14. The mudline elevation is at EL -2.00. the boat launch ramp is set at 10:1 slope between mudline and top of turnaround. The ramp consists of 12” cast-in-place concrete slab above normal water and pre-cast concrete panels below the normal waterline. Compacted limestone is provided under concrete slab. Riprap is provided at the end of sides of pre-cast concrete panels to control erosion.

A timber dock/boarding pier is provided alongside of launch ramp for access to boat. The 5’-6” wide boarding pier (dock) is designed as a means to help safely and efficiently launch and retrieve boats, and load and unload boaters at the launch facility. Timber walkway consists of timber deck boards, timber stringers and timber piles with cross bracings. The launch lane is 25 feet wide.

9.8 Miscellaneous Design, Analysis, and Construction Items

Performance Assumptions All Buildings

- Building use and operations are for ordinary code occupancy in Risk Category II
- Buildings are to be of normal construction with standard code prescribed live loads and structural deflection limitations.
- The facilities are not designated for use as an emergency operations/communications center and does not store significant quantities of toxic or hazardous materials.
- Maintenance and washing facility structures are subject to normal maintenance activities such as; minor equipment loads, tool cart impact and wash downs.
- Administration building is designed for standard live load.
- Floor slab in the Pole Shed building is to be designed for HS-20 traffic loads, and light or heavy storage loads.
- Floor slab in the Maintenance building is designed for heavy storage loads.
- Slabs and raised floor flatness and levelness is a normal classification with a specified overall FF=35 and FL=25 which is suitable for office use, low speed vehicular traffic and conventional lifts.
- Exposed concrete floors are to have control joints at regular intervals, approximately 20 to 30 feet on center with block outs for steel columns.
- Traffic bollards, curbs or guards are to be implemented where vehicle impact is of concern, such as adjacent to vehicle doors, corners of buildings, and columns immediately adjacent to major driveways.
- Foundations are to be designed to maintain global structural stability with differential settlements per the geotechnical engineer.
- Reinforcement is to be standard deformed type, or a finish coating as required where exposed to view or the elements. Structural steel is to be uncoated unless exposed to view or the elements, in which case a galvanized or protective finish will be applied.
- Concrete shall use Type II Cement; local aggregates are assumed to be acceptable for use.

- Overhead crane and support are required in the Maintenance Building, required allowances for load and deflection are included in accordance with AISC and CMAA recommendations.
- HVAC equipment anchorage and fire sprinkler piping supports are deferred approval items that are not part of the structural drawings.

System Modeling and Calculations

Structural analyses for gravity, wind and seismic forces were performed primarily by manual calculation, Staad Pro Structural System. For seismic design a static equivalent lateral force analysis was used and compared to the wind loading for governance.

Administration/Maintenance building is designed using STAAD.

Final design of Pole Shed building will be provided by the metal building manufacturer and reviewed by the design team for design intent.

CONSTRUCTION MATERIALS

Concrete
Concrete design strengths for structural elements will based on the minimum 28-day compressive strengths (f'c) as indicated below:

Miscellaneous concrete structures: f'c = 4000 psi

Reinforcing Steel

Reinforcing steel will conform to ASTM A615, Grade 60, yield strength, fy = 60,000 psi.

Structural Steel

Plates and shapes shall conform to ASTM A572, Grade 50, Fy = 50 ksi

Minimum Reinforcement Cover

Design criteria for concrete protection of reinforcement should conform to the minimum conditions contained below:

Minimum Concrete Clear Cover
Condition

Unformed surfaces in contact with foundation. 4 in.
Formed or screeded surfaces, subject to erosion. 3 in.
Formed or screeded surfaces slabs on grade. 3 in.
Equal to or greater than 24 inches in thickness. 4 in

RECOMMENDED SOIL DESIGN VALUES

The recommended design soil parameters are based on the test results presented in the Geotechnical Investigation Appendix of this DDR.

For miscellaneous structures and culverts:

- Moist soil unit weight, γ moist = 115 pcf
- Allowable Friction Coefficient = 0.30
- Allowable Passive Pressure Coefficient = 1.7
- Active Earth Pressure Coefficient = 0.39
- At-rest Lateral Pressure Coefficient = 1.00
- Earth Pressure Coefficient for Culvert = 0.33 min., 1.0 max.

Damage limit state

A minimum damage control limit states was utilized to minimize damage to facilities for economic and practical reasons in consideration of minimizing repair costs required after storm events, and to make repairs financially feasible without the need for facility demolition.

Drawings

Drawings are included in the project plan set.

Metal Building System for Pole Shed:

The choice of a Metal Building Systems for this project will provide economy in the design, the extensive use of computers for design and fabrication will result in a low-cost system that will be material efficient, quickly fabricated, and easily erected.

The Metal Buildings for this project are flexible structures and will move under the application of wind, seismic, and crane loading. The IBC2016, MBMA, and AISC codes and design standards were reviewed for guidance for the allowable drift in the building system. The review insured that the maximum allowable frame drift is suitable for the proposed structure considering all details of construction. The Maintenance building support columns are used to support a top running crane system, the crane must be supported so that differential movement between the building columns is minimized for operating loads and does not overstress the columns and result in local column buckling. IBC and AISC Drift limitations were included in the analysis.

The makeup of the buildings system includes purlins, girts, and X or K bracing. Cable X bracing will not be allowed due to deflection limits.

Large sliding doors will require long spanning headers over the openings in the building walls.

The structure will provide adequate coverage to allow maintenance during rain events.

Technical Review

Design Analysis. A building design analysis and check was performed for the Administration/Maintenance Building to determine column reactions. A foundation analysis review was performed for the foundation system as noted. Computer analysis of the building steel frames using Staad.Pro was performed.

Loadings applied were calculated for dead, live, wind and equipment loads and applied at pertinent locations.

Design Requirements All Buildings

Design Loads:
Roof LL = 20 psf
Floor LL= 60 psf

Crane Loads (New 5 Ton Bridge Crane 20 – 22 ft. span Maintenance Building)

2,900.00 lbs.	Trolley Weight
4000.00 lbs.	Crane Beams Weight
10,300.00 lbs.	Max Wheel Load Static
10,000.00 lbs.	Hook Load
11,700.00 lbs.	Vertical + .25 Impact
2,200.00 lbs.	Transverse + .2 Breaking / Running
33,640.00 lbs.	Long Direction+ .1 Breaking / Running

Equipment Loads

250 PSF

Load Combinations ASD

(1) Combined loads DL, DL + LL, DL+ WL, DL+LL+WL

(2) Wind loads were based on current IBC and ASCE7 recommendations.

(3) Seismic loads rarely control the design of lightweight flexible steel structures in the Plaquemines Area. These loads become more of a concern if rigid masonry or precast elements are attached to the structure, such as cladding. Typically, these loads do not govern the design in Southeast Louisiana; the lateral load acceleration factors and loads are normally less than wind lateral loading.

Foundation

Pile Loads:

Pile load allowable assumptions are as follows:
Ultimate Pile Capacity: 40-120 Tons - 40 to 90 foot penetration
Allowable Pile Capacity: 20-60 Tons - 40 to 90 foot penetration

Pile Capacity:

The pile capacity was provided in the geotechnical report based on the presence of a soft clay soil; the pile capacities are assumed at the loads listed above. Assumed Pile Loads are reasonable loads used on similar projects for the Greater New Orleans Area.

Building Element Systems Review

Metal Building Frame:

The metal building frames were analyzed using Staad.Pro with the applied loads listed above.

The frame results show that a metal building system can be utilized and meet the deflection limits required by adjusting the haunch depth.

Roofing Dead Load Assumptions:

Structural Standing Seam Metal Roofing is to be used on the structures, the purlin spacing of 18 inches maximum at corner, edge, and ridge zones and at 30 inches maximum for the remainder of the roof are required and must be clearly shown on the drawings, or in the specification. In any case the manufacturers maximum recommended spacing shall not be exceeded. Slope should be 1 on 12" minimum.

Lateral Force Restraint:

The method for resisting lateral loads will include cross-bracing (X or K-bracing), diagonal bracing, and rigid frames. Bracing conflicts with doorways and openings will be mitigated with rigid frames to provide lateral restraint. Bays and roof/wall openings that must remain free of bracing will be shown on the design drawings. The main frame system for this building is rigid in one or both directions of the framing system. X bracing will be utilized in the roofing system to provide unilateral transfer of horizontal overhead crane loads and reduce the overall system deflections. X or K bracing will be utilized in bays where geometry permits to provide economy of design.

Base Plate Bearing:

Standard practice for the Metal Building System industry does not require base plates to be shimmed with non-shrink grout to fill the void to assure good bearing. Base plates are required at each column support point and at rigid frame header openings for discharge tubes and truck access door openings.

Erection Plan:

An acceptable site-specific erection plan must be provided by the contractor for this project.

The building manufacturer frequently requires the erector, or another third party provide the plan. The contractor will be required to provide an erection plan due to the long span rigid frames and overhead crane system which will require unique erection solutions. Additional roof x-bracing will be required during construction for overhead crane erection. Several metal building structures have collapsed during construction due to inadequate bracing/erection plans. The requirement for a site-specific erection plan will be strictly enforced by the design engineer. A review by the building manufacturer will be required when plans are prepared by a separate erector for this structure.

Job Assumptions:

The building foundations and crane loads calculated in this report were estimates for the purpose of installing the new maintenance building. The contractor will verify the metal building foundations loads for the actual building and crane loads provided by the Metal Building Manufacturer. Actual assumed loads for crane system and building may vary due to the actual systems procured, but the assessment will work for different configurations which utilize similar systems.

Load Requirements for Member sizes and strengths calculated are shown on the provided drawings.

Concrete assumed strength =4,000 psi, steel yield strength = Grade 60 ksi

10. MECHANICAL DESIGN

For the purposes of this DDR, the HVAC, Plumbing, and Fire Protection Sprinkler systems are addressed together. The reader should refer to the Design Criteria Document for additional criteria applicable to the design intent. The text herein is intended address design decisions specific to this project.

10.1 HVAC Systems

The HVAC systems have been determined to a large extent by selecting cost-effective and energy efficient equipment. The systems consist of direct-expansion split units featuring air-cooled remote condensers and indoor evaporator air handling units. Indoor air quality is maintained via direct outside air connections to the air handling units in accordance with ASHRAE 62.1, Ventilation for Acceptable Indoor Air Quality, and the International Mechanical Code Chapter 403.3.

The Administration and Operations Building is fully conditioned within the occupied space and ventilated within the Vehicle Bay/Garage. The HVAC system features one 3.5-ton and two 5-ton heat pump systems with supplemental electric heat. The vertical air handling units are located in an equipment closet and feature ducted supply, return, and outside air connections. The selected equipment has been designed to optimize the HVAC zoning for the building. All restroom ventilation is provided via ceiling mounted exhaust fans. The Vehicle Bay/Garage ventilation consists of electric unit heaters and wall-mounted propeller exhaust fans. Electric heat was selected to eliminate the need to bring gas service to the reservation, thus saving construction costs. The fans may be operated in hand or auto mode with auto-mode controlled via a space mounted carbon monoxide sensor.

Each gate structure control house will be ventilated with a wall-mounted exhaust fan equipped with an exterior louver. The door to the control house will include an intake louver. Both louvers will be specified as hurricane-wind-resistant (Miami Dade County).

10.2 Plumbing Systems

In order to supply the plumbing systems, water services to each building are provided. Double check backflow preventers and meters are installed in the service near the property line to protect the City's municipal water supply system. The water meters, backflow preventers, and site water piping are designed as part of the site distribution system under Division 2 work. For the Administration and Operations Building the domestic water service is brought in at the mechanical room with the fire protection service brought in at the Sprinkler Riser Room.

To implement the most cost-effective products available at procurement, PEX and copper piping is specified for above ground domestic water piping. Insulation is specified for all above ground domestic water piping to prevent condensation of cold water and heat loss of the hot water piping. Cold and hot water connections are provided for all plumbing fixtures requiring such. Cold water connections are provided for all ice makers and refrigerator connections using wall boxes complete with isolation valves. Freeze proof wall hydrants are provided on the building facades at grade level. Hose bibs are provided for maintenance within the Pole Shed building equipment storage area.

A single electric, high efficiency, storage tank, commercial grade, domestic water heater is specified to serve the building. This has been selected to minimize the load on the electrical system over point-of-use instantaneous water heaters. Local thermostatic mixing valves are provided to maintain 110-120^{o}F water at all areas. Domestic hot water shall be supplied to all lavatories, sinks and showers.

Commercial grade plumbing fixtures are specified throughout the facility. Water closets are specified as floor mounted, flush valve units. Lavatories are specified with low flow, manually operated faucets with mixing valves.

The sanitary sewer system is specified as cast iron piping for above grade and PVC piping for below grade. All sanitary sewer system piping is routed to the exterior of the building and extended 5’ from the building perimeter where the continuation is under Division 2 work. All restrooms and mechanical rooms are provided with floor drains with trap seal protective devices. Cleanouts are specified at a maximum of 50-foot intervals. All piping below the slab is to be hung with stainless steel hangers. In the Administration and Operations Building, sediment traps are specified for all Work Area/Shop and Soils Lab sinks to prevent solids from entering the sanitary sewer system.

The Administration and Operations Building does not require an interior storm drain system. However, the Vehicle Bay/Garage within the building features interior trench drains piped to an oil-water separator. The storm drain system piping is routed to the exterior of the building and extended 5 feet from the building perimeter where the continuation is under Division 2 work.

A compressed air system, piped in loop formation, is provided for the Administration and Operations Building. The system features 3/8-inch compressed air outlets connected to the 1-inch loop piping. All piping will route high to avoid conflicts with the Vehicle Bay crane system.

10.3 Fire Protection Sprinkler Systems

Fire protection sprinkler systems are provided at the Administration and Operations Building. In order to supply the sprinkler systems, a 6-inch fire water service is brought into the Sprinkler Riser Room. A double check backflow preventer and meter will be installed in the site service near the property line to protect the City’s municipal water supply system. The water meter and backflow preventer are designed as part of the site distribution system under Division 2 work.

Schedule 40 black steel pipe is specified for sprinkler piping 2 inches and smaller and schedule 10 steel pipe is specified for the larger sizes. Sprinklers are required throughout all portions of the protected buildings. Upright brass sprinklers are specified for mechanical and storage rooms and upright brass sprinklers with guards are specified for the Administration and Operations Building vehicle garage. Semi-recessed sprinklers are specified for lay-in ceiling areas. As the Operations & Maintenance Building vehicle garage features freeze-protection heaters, a dry-type sprinkler system is not required.

10.4 Gantry Crane

10.4.1 General

A new rail-mounted gantry crane (RMGC) will operate the three (3) bulkhead gates at the intake structure. It is anticipated the gates will be operated numerous times generally within the MR flood season each year, meaning the crane will be used often for approximately half the year and sit stored for half. The gantry crane system will be supplied by others; the drawings and specifications provide all data required for purchase of this item. The diversion will typically operate during riverine flood events up to 1.25 million cfs of river flow passing the diversion, but the gates may be partially closed to regulate diverted flows at approximately 75,000 cfs. All gates will be fully closed for a riverine flood event only in rare circumstances. The diversion will not operate during a hurricane or tropical storm event. The three gates will be closed in advance of a storm.

10.4.2 Design Criteria and Loading Conditions

The required RMGC capacity is determined by evaluating the load cases as shown in **Table 5-7** of the design criteria, **Appendix A**. The crane capacity is rated for lifting the full stack of four (4) bulkheads and traveling with a stack of two (2) bulkheads. All loads are unfactored for calculation of required capacity. The crane vendor will be instructed that rated load capacity is based on unfactored loads.

The loads that will be applied to the crane during gate operations include dead load and drag loads. The dead loads include the weight of the gate and silt accumulation. The dead loads are calculated considering buoyant weight for the portions of the bulkheads that are below the waterline on both sides of the gate. The silt weight loads are calculated considering silt accumulation between the flanges of gate truss members that are below the waterline on the basin side.

Drag loads are calculated assuming one wheel in the stack is jammed. A drag friction factor of 0.05 is used for the 15 bulkhead wheels that are assumed to be operational. A friction factor of 0.78 is used for the one bulkhead wheel that is assumed to be jammed. These friction factors are multiplied by the lateral load on the gate to determine the drag load.

10.4.3 Analysis and Design Summaries

The unfactored required capacity is 428 short tons; a slightly conservative load rating of 450 short tons will be provided to prospective crane manufacturers.

10.4.4 Operational Requirements and Loads

- Gantry Rated Capacity, Stationary...450 short tons
- Gantry Rated Capacity, Traveling...120 short tons
- Travel Distance...300 feet
- Rated Gantry Travel Speed, variable without load..............................0.1 to 4 ft/sec
- Rated Gantry Travel Speed, variable with load.................................0.1 to 2.5 ft/sec
- Hoist Rated Capacity...450 tons
- Rated Hoisting Speed, variable, without load..................................0.5 to 15 ft/min
- Rated Hoisting Speed, variable, with load...0.1 to 7 ft/min
- Hoist Lift Height (Upper Limit of Travel to Rail Elevation) ....See drawings
- Hoist Lift Height (Upper to Lower Limit of Travel)... As required
- Trolley Rated Capacity, Stationary...450 short tons
- Trolley Rated Capacity, Traveling...120 short tons
- Rated Trolley Travel Speed, variable, without load..............................0.1 to 4 ft/sec
- Rated Trolley Travel Speed, variable, loaded.....................................0.1 to 2.5 ft/sec

10.5 Gate Wheel and Axle

The bulkhead wheels will be cantilevered from an axle extending into the gate framing. The wheel diameter will be 26 inches and have a flat thread with a width of 9 inches. The maximum wheel load due to a hydraulic head is 349 kips which may be experienced during dewatering. Maximum operating head, excluding a dewatering is 277 kips. The wheels will have self-lubricated spherical bearing to keep the entire area of the thread engaged against the track, thus compensating for gate deflection and irregularity in the track. The axle, wheel and wheel track will be manufactured from AISI 4140, Condition "T". This material is also resistant to corrosion and in this respect, it is between stainless steel 304 and 316. The axle is designed with a minimum safety factor of 5 based on the ultimate strength of the material and on a maximum of 75% of the material yield strength. The axle centerline and bearing centerline are offset by

¼ -inch to allow alignment of all four wheels to a single plain. The wheel and track hardness are based on Rorak's formula for Hertz stress. The Roark's formula is used by the USACE for gate wheels.

10.6 Gate Dogging Devices

The structure will require 18 dogging devices. Two dogging devices, one on either side of a gate or bulkhead, are designed to support a gate consisting of 4 bulkheads with silt loading latched together. The design loading per dog is 262.5 kips. Operationally, the dogs for holding the gate pivot out from the gate wheel recesses and support the gate from the bulkhead wheels. Two dogging devices are required on the ends of the gate or bulkhead. Because of the pivoting action of the dog, the weight of the gate produces a vertical and horizonal load on the dog which are accounted for in the dog's pivot pin and restraining bar. The restraining bar load is approximately a third of the load resulting from the gate or bulkhead weight. All the dogging devices are identical except for the restraining bars on the center piers. These are different because of the configuration of having adjacent overlapping dog recesses on the center piers. The dogs are rotated from the recesses when needed using a hydraulic cylinder. The cylinder will be powered using a standard compact hydraulic unit driven by a 1/3 Hp 120-volt electric motor. The dogs will be controlled by a handheld pendant which may be wireless. The dogs are designed with a safety factor of 5 based on the ultimate strength of the material and not less than 75% of material yield strength. The dogging devices are also checked for 2G loading at 75% yield. The 2G loading is transient and may result during the initial placing of a gate on the dogs.

11. ELECTRICAL AND CONTROLS

For the purposes of this Design Documentation Report, the Electrical and Controls designs are addressed together, since decisions regarding the Electrical Systems impact the Controls Systems, and vice versa. The reader should refer to the Design Criteria Document for additional criteria applicable to the design intent. The text herein is intended address design decisions specific to this project.

11.1 General

The Electrical and Control Systems are being designed with the understanding that the Gate Structure is not intended to be operated during storm conditions or to allow a backflow of water from the Barataria Basin to the Mississippi River.

Information within this section is intended to supersede the information presented in the 30% Design Documentation Report.

11.2 Electrical Site Distribution

Electrical service (utility power) to the site will be provided by an overhead, utility-provided distribution system (13.8 kV or 25 kV, nominal). Utility distribution lines will serve utility-provided transformers located at the Reservation, at the Gate Structure, at the Siphon, and at the Drainage Structure. This approach provides significant cost savings as compared to a primary metered service or serving the entire site (or portions thereof) entirely from the Reservation. Additionally, long underground conduit runs present a settlement risk which is mitigated by a several-location electrical service approach.

11.3 Lighting

LED lighting will be specified throughout. LED fixtures have come down considerably in cost over the past several years. They typically have a greater efficacy than other sources, they provide instant-on lighting, and they typically provide a service life of 20 years or more, all resulting in a lower total cost. Other sources have not been considered, but they would be considered if an application-appropriate LED fixture was not available.

Standard LED cobra heads were selected for Reservation Site Lighting. Such fixtures are cost-effective, readily available, and require limited maintenance. Type II LED cobra heads will also be used for the Gate Structure Access Bridge/Roadway.

Exterior Entry / Exit egress lighting for the Reservation Building will be powered by inverters. This approach is typically more economical that individual battery packs for each exterior egress light. Wall packs serving the Vehicle Bay are purpose-driven and not required for egress; therefore, those fixtures will not have integral battery pack, nor will they be backed up by a UL924 inverter.

Obstruction lighting will be via self-contained, battery- and solar-powered ATON-approved light fixtures

11.4 Power

Power to the Reservation Building will be via a pole-mounted utility transformer serving a service-entrance rated main disconnect switch. Power for the Pole Shed and Sewer Lift Station / Treatment Plant will also be provided from the Main Reservation Building.

A generator will provide standby power when utility service is unavailable. Current panel schedules reflect known and/or anticipated electrical loads. Several loads are still unknown, particularly in the Soils Lab, Shop Area, and Vehicle Bay.

Surge protection will be installed adjacent to panelboards (in lieu of integral TVSS) per the requirements of UFC 3-520-01.

Power for the Gate structure, and auxiliary power for the crane, will be provided via a pole-mounted utility transformer serving a service-entrance rated main disconnect switch. A generator will provide standby power when utility service is unavailable.

Power for the Siphon will be provided via a pole-mounted utility transformer serving a service-entrance rated main disconnect switch. No permanent standby power source will be provided; emergency gate operation will be via handwheel/portable drill.

Power for the Drainage Structure will be provided via a pole-mounted utility transformer serving a service-entrance rated main disconnect switch. No permanent standby power source will be provided; emergency gate operation will be via handwheel/portable drill.

11.5 Standby Generators

Standby power is currently illustrated for the Gate Structure and the Reservation Building. The standby generator dedicated for the Reservation Building will also serve the Sewer Lift Station/Treatment Plant and site lighting at the Reservation. The standby generator dedicated at the Gate Structure will also serve the access roadway lighting.

Several factors contributed to the decision to include two generator sets in the design. The major factors are listed below.

> Distance: The Reservation and the Gate Structure are roughly 1000 feet away from each other. Underground feeders would need to be grossly over-sized to limit voltage drop at the Gate Structure, plus there is a risk of differential conduit settlement that would require steel-reinforced concrete duct banks to mitigate.
>
> Cost: It is anticipated that the cost for a 1000-foot, steel-reinforced, concrete encased ductbank would exceed the cost for a locally installed permanent generator.

Gate Structure Standby Generator:

> The generator dedicated to the Gate Structure will be a packaged unit with sub-base fuel tank and weather-resistant housing rated to withstand a minimum of 150 mph winds.
>
> Sub-base fuel tank capacity will be sized to provide minimum run time at full load of 72 hours. Per NFPA 110, that capacity will be multiplied by 1.33 to determine the minimum tank size required.
>
> Generator will start whenever utility power is lost. This sequence will ensure that power for bulkhead dogging pins and auxiliary power to the crane will be maintained.

To prevent fuel from spoiling, a fuel polishing system will be required; without the polishing system, it is not anticipated that the fuel consumed during normal weekly testing will be enough to regularly deplete the tank.

Reservation Standby Generator:

The generator dedicated to the Reservation will be a packaged unit with sub-base fuel tank and weather-resistant housing rated to withstand a minimum of 150 mph winds.

Sub-base fuel tank capacity will be sized to provide minimum run time at full load of 72 hours. Per NFPA 110, that capacity will be multiplied by 1.33 to determine the minimum tank size required.

The generator will start whenever utility power is lost. This sequence will ensure that power for the Sewer Lift Station/Treatment Plant, site security systems, and site lighting will be maintained.

To prevent fuel from spoiling, a fuel polishing system will be required; without the polishing system, it is not anticipated that the fuel consumed during normal weekly testing will be enough to regularly deplete the tank

11.6 Grounding and Lightning Protection

Details of the grounding system will be provided after the 60% submittal. Future designs will illustrate ground loops around the Reservation Building, a ground loop around the Gate Structure Powerhouse, and ground rods for each electrical service location. Ground rods will be installed in test wells for future inspection. Copper-bonded steel ground rods will be specified in lieu of the traditional copper-clad steel rods; the copper coating on copper-bonded rods is made with an electrolytic process preventing the copper coating from cracking when bent or driven. Stainless steel rods were also considered, but there is no evidence yet provided that the soil conditions at this site will be corrosive to copper. Furthermore, copper-bonded steel rods are galvanically compatible with the copper grounding electrode conductors / ground ring; no additional treatment at connections is necessary to prevent galvanic corrosion.

11.7 Fire Alarm and Mass Notification Systems

The Reservation Building will be equipped with a code-compliant fire alarm system. Layouts illustrated on drawings are based on NFPA requirements. Only a tone/visual system is required, unless a Mass Notification System is provided, in which case speakers will be required.

A Mass Notification System is not mandated by NFPA 101 or the IBC. However, if compliance with UFC 4-010-01 is mandated, A Mass Notification System (MNS) will be required. The inclusion of an MNS does result in a significant project cost. An MNS is not currently included in the design, and input from the End User is necessary regarding whether or not compliance with UFC-010-01 will be required.

11.8 Access Control Systems

Access Controls will be provided at the vehicle entry point to the Reservation and at each exterior door of the Reservation Building. Control equipment will be located in the Reservation Building.

11.9 Gate Structure Controls

Any controls required for the actuated bulkhead dogging pins will be manual and will be located in the Gate Structure Powerhouse.

11.10 Lighting Controls

Site lighting at the Reservation will be controlled by photocells connected to a lighting control panel.

Site lighting at the Gate Structure will be controlled by a photocell connected to a lighting contactor via a hand-off-automatic switch.

Site lighting at the Siphon and Drainage Structure will be controlled directly by photocell.

Lighting within the Reservation Building will be controlled by a combination of manual controls, occupancy sensors and schedule-based controls from a lighting control panel.

12. REAL ESTATE

CPRA will acquire both temporary and permanent rights-of-way for the construction and operation of the MBSD Project. Right-of-way plans are currently being developed to show property lines, ownership information, and acreage. Existing landowners include Plaquemines Parish Government, Plaquemines Port, Harbor and Terminal District, Phillips 66, Midway Cattle, River Rest, and other private landowners. The total estimate for required permanent right-of-way is approximately 2,700 acres, and the total estimate for temporary right-of-way is approximately 500 acres.

13. CONSTRUCTION SEQUENCING DESCRIPTION AND GEOTECHNICAL ANALYSIS SUMMARY

13.1 General Description

Construction of the MBSD complex will be accomplished through the safe and well-planned execution of construction activities to complete installation of both temporary and permanent features of work described in this section. Each feature of work will have activities that are performed concurrently throughout the project. This section is accompanied by supporting design criteria, design drawings and engineering analyses to support the temporary works structures that will be utilized to construct the Headworks structure, the conveyance channel, the inverted siphon, and other project features. These supporting documents are included in **Appendix J**.

Temporary Works include:

- Appendix J 0.01, Temporary Works Plans
- Appendix J 0.02, Interim Levee System
- Appendix J 0.03, Temporary Circular Cell Cofferdam
- Appendix J 0.04, River Trestle Dock Facility and Fleeting Area
- Appendix J 0.05, Excavation for Inverted Siphon and DMM
- Appendix J 0.06, Braced Excavations for the Highway 23 Bridge Piers
- Appendix J 0.07, Pump Test Plan Report
- Appendix J 0.08, Dredge Access Channel Survey
- Appendix J 0.09, Emergency Hurricane Evacuation Plan
- Appendix J 0.10, Dewatering System Plan
- Appendix J 0.11, Linear Schedule

Construction of the Headworks intake structure requires excavation to an EL -35, to permit the structure invert to be built to EL -25. Temporary structures to protect against river flooding from a high-water event on the Mississippi River consists of two elements: a steel sheet pile cellular cofferdam system and an Interim Levee (IL) to ring the landside of the excavation tying into the Mississippi River Levee (MRL). These temporary structures will provide continuous protection during construction activities for the headworks structure. The IL will be a line of MRL Flood Protection for estimated 3-4 years and as such has been designed to comply with HSDRRSDG, and USACE EMs, and USACE New Orleans District standards for the MRL. The specific criteria for these temporary elements are listed in the Project Design Criteria, **Appendix J.2**, MB Interim Levee Report R3 and **Appendix J.3**, 60% Cofferdam Design Analysis. The cofferdam is currently designed with a top elevation of 16.4. The top elevation is presented in the 60% design documents and the design criteria for the cellular cofferdam is presented in **Appendix J.3** (See Drawing C-103).

Temporary river armoring will be placed against the outboard sheet of the coffer cells to prevent potential erosion and scour of the riverbed soils outboard of the coffer cells. The size of the stone will be determined using the velocities as determined from calculations, numerical, and physical modeling for the Headworks structure. Stone will be continuous from cell 1 to the downstream cell number 17. Testing of the need for the deflector has been accomplished in a physical model which determined the velocities of the water moving past the cells. The velocities measured in the model confirmed that an upstream deflector was not necessary as a feature to protect the cofferdam against excessive erosion. As such, the deflector system originally planned will be eliminated in the final design of the cofferdam. The results of the model testing are contained in the Hydraulic Model Testing Report.

13.1.1 Summary of Cofferdam Design Analysis

Failure Modes Analyzed

The coffer cells were analyzed for the following failure mechanisms (as required by EM 1110-2-2503 Design of Sheet Pile Cellular Structures, Cofferdams, and Retaining Structures): Sliding, Overturning, Rotation, Bearing Capacity Failure, Interlock Tension, Vertical Shear, Horizontal Shear, Pull out of Outboard sheet pile, and Penetration of Inboard sheeting. The location of the cofferdam and dewatering details were in a state of flux at the time this analysis was performed. Deep seated Sliding, and Seepage control will be evaluated later as these details are finalized. Results of these analysis are summarized on Calculation Summary pages 3 through 7 included in the Shannon & Wilson 60% Design Cofferdam Analysis dated February 1, 2021.

Settlement Analysis

A two-dimensional settlement analysis of the cellular cofferdam was performed along a section cut perpendicular to the cofferdam and roughly paralleling the main excavation centerline. The section was chosen because it corresponded with the maximum cuts, maximum cellular fill to be placed, and within an area where groundwater drawdown was high. Large changes in effective stress will occur in this area due to construction activities.

The settlement analysis used consolidation parameters from results of consolidation testing in the vicinity of the proposed cofferdam location and considered correlations to index testing where consolidation testing was not available. Several dewatering and excavation scenarios were analyzed to provide an estimate of settlement at different phases of construction, and at several locations along the section. The analysis indicates the greatest settlement is anticipated under the center of the cell and decreases moving toward the excavation. Due to complexity, we will inspect and monitor regularly.

Settlement of the cofferdam is currently being evaluated in greater detail considering the proposed construction means and methods.

13.2 Construction of Cofferdam and Tie-In Structure

13.2.1 Clearing and Site Preparation for Cofferdam

The area where the cofferdam will be constructed will be cleared of the existing sediment, foreign debris, and the revetment mat. Track hoe excavator and drag line equipment will be used on a floating plant to clear the area. Divers will be used to sever the cables that are part of the revetment mat system, making removal in smaller segments compatible with lifting equipment that will remove the mat material. Once the area is cleared of debris, then placement of the coffer cells and connecting arcs will proceed. It should be noted that only the footprint of the coffer cells will be cleared of revetment and debris.

13.2.2 Construction Sequencing of Cofferdam

The cofferdam will be constructed in the sequence presented in the following paragraphs. For the benefit of material handling, installation and safety, the sequence has been evaluated for river velocities at the maximum flow of 1,000,000 cfs utilizing the physical hydraulic model that was used for modeling of the inlet structure. Results of the model testing are included in the Mid-Barataria Interim Physical Modeling Report (draft submittal)dated July 09, 2021.

Two cell placement crews will be used to place templates, set sheets, and drive sheets for the cells and connecting arcs at the upstream arm of the cofferdam. Filling of the cells and arcs will be accomplished by a separate operation consisting of clamming cranes, loaders, push boats and material barges. Placement of cell fill will be by conveyor or clamming, not with hydraulic methods. Placement of the templates, setting of the sheets, driving of the sheets, and filling the cell to the prevailing river elevation will be a continuous operation. This will ensure safety during installation and minimize risk of damage to a cell from environmental loading such as wind and river flow that may cause the cell to lean or rack in position.

13.2.3 Sequence of Cell Placement

Prior to placement of the coffer cells the riverbed will be excavated to clear any foreign material within the footprint of the cofferdam that may obstruct driving of the sheet pile. Revetment, metal objects and anchor systems will be located and removed. Installation of the sheet piles will utilize both vibratory and impact hammers to reach the required tip elevations for each cell and arc. Construction activities for installation of the cofferdam and trestle will be conducted for river stages up to elevation 12.0 at the Alliance gage. Looking at the forces on the cofferdam due to the water levels, it is the Alliance gage that will provide the actual level of water that the coffer cells are dealing with relative to the overturning and sliding analysis for the cells. There is no linear correlation between the Carrolton gage and the Alliance gage. Thus, for actual levels for water loads, using the Alliance gage provides the actual water levels that will introduce the water loading on the cells. The cells are currently designed for water loads up to 16.4 at the Alliance gage. The 16.4 maximum design water elevation for the coffer cells may be reached during a surge event from a hurricane and gaging as close to the cofferdam as possible gives one the actual levels to correlate with the design analysis. When the water load reaches 14.4 at the cofferdam the emergency evacuation operation protocol takes over for a hurricane event and rewatering of the cofferdam is initiated thus reducing the differential water load on the coffer cells. The Alliance gage provides the accurate water level to relate to the design assumptions when dealing with forces from various loading (water in this case) at the point of application.

Sequence of cell placement begins with cell 1 and progresses sequentially to cell 7 (See Drawing C-101). At the onset of placing cell 8 a second cell setting crew, crew two, will begin placement of cell 23 and proceed upstream to cell 16. The placement of the cells 9 to 15 will proceed toward cell 16 making closure of the cofferdam. Once the cells are in place and filled to within 2 feet of the top, the cell fill will be capped with 2 feet of 130 lb. stone to prevent scour should the coffer cells be overtopped. The stone armoring along the external base of the cells will also be placed as cells and arcs are completed. The inside stability berm will be completed during excavation of the interior headworks excavation.

13.2.4 Cell Fill Material

Sand for cell fill material will be obtained by hydraulically dredging river sand from the USACE permitted Alliance South borrow site location (See Drawing C-103). Alliance south is west of the ship channel, directly adjacent to the MBSD project and the planned cofferdam. Approximately 150,000 CY of material will be hydraulically dredged, placed in hopper barges and clamed into the cofferdam cells.

Prior to removal of the cofferdam cells, the cell fill within the cofferdam cells will be removed and redistributed into the river. The fill material is classified as an SP sand with less than 10% passing the number 200 sieve. The permeability is estimated at 0.17 cm/sec using D_{10} correlation. A gradation report for the cell fill material is included in **Appendix J.3**.

13.2.5 Upstream Deflector

Installation of an upstream deflector was originally planned for the cofferdam to mitigate any erosion adjacent to the upstream arm cells. Based on preliminary velocity readings from the fixed bed hydraulic model the deflector will not be required and as such will be removed from the previous submitted layout of the cofferdam. A report is being assembled and will be provided later.

The cell diameter is currently set at 61.68 feet for design. The current layout is based on a sheet pile width of 19.69 inches as currently being manufactured for a PS31 sheet pile. PS31 sheet pile will be new and unused and is readily available from manufactures such as Skyline, L.B. Foster, and other suppliers.

13.2.6 Surcharge Loading of Coffer Cell

Surcharge loading on the cells will consist of vehicle traffic along with a 4100 Manitowoc crane (or crane of similar capacity) that will be used to lift and place materials into the cofferdam that is delivered by barge. The operating weight of the crane is 350 kips with a maximum lifting load not to exceed 200 kips, resulting in an applied load at the top of the cells of 550 kips. The crane loading will be distributed through the track assembly area and the crane mats below the tracks. The increase to the hoop stress and interlock tension due to the crane/vehicle loading was analyzed by using the entire weight of the crane and lifting load and spread across one mat located at the edge of the cell. The increase in vertical stress was analyzed using a stress distribution below the corner of a rectangular area. The increase in lateral pressure from the increase in vertical stress was added to the horizontal pressures calculated for determining the maximum hoop stress. Bearing capacity of the cell with the added surcharge loading was determined as well. A factor of safety of 2.0 has been met for both interlock tension as well as bearing capacity per EM 1110-2-2503, Design of Sheet Pile Circular Structures. Criteria for separation of cranes on top of the cofferdam will be set by safety policy related to boom swing, but in any case, cranes will not be located on adjacent cells or arcs that have the potential to increase interlock stresses (See **Appendix J.3** for calculations)

13.2.7 Water Loading in the Cell

The water level inside the cell has been set at 16.4-foot for the outboard side of the cell, with a slope of 2H:lV to the inboard side of the cell. This water profile is the maximum water loading used for design and determination of the interlock stresses. Interlock stresses were also checked assuming full cell saturation as a worst-case scenario. The dewatered elevation of the interior excavation of the cofferdam (Headworks Excavation) is set at 5 feet to 10 feet below excavation grade line inside the cofferdam as the excavation progresses to the required depth of -35 feet. Cell fill, and subsurface soils have been evaluated for drainability when responding to the draw down from the dewatering system.

13.2.8 Interior Stability Berm

The interior cofferdam stability berm is designed using procedures and guidance contained in EM 1110-2-1902, Slope Stability. Detailed analysis was performed using SLOPE/W. The interior cofferdam stability berm fill will consist of CL/CH existing soil. The crest of the berm will be at -20 with a berm width of 25 feet at the top and with slopes set at 1V:8H to elevation -35. Erosion of the slopes during rewatering with pumps will be prevented by use of pipes extending to the base of the excavation. Overtopping of the cells will be prevented by use of the flood gates and flood way that will be utilized after the interior of the excavation has been filled in a controlled manner to an elevation of 0.0.

13.2.9 Subsurface Materials

Subsurface information has been obtained along the alignment of the cellular cofferdam to define the soil properties for the cellular cofferdam design. Material type, soil classification, strength parameters and permeability characteristics have been determined and the results are included in the document, Geotech Report, **Appendix C**.

13.2.10 Impact Loading to the Cofferdam Cells

The upstream river arm cells and arcs will be protected from vessel impact by energy dissipating devices (See Drawings C-121 and S-230). Protection of the river cells is being addressed using berthing features that are commonly used for commercial dock facilities. The energy absorbing features consist of rubber pneumatic energy absorbers that absorb the impact energy from an inland rivers standard hopper barge fully loaded with a gross tonnage of 2272 tons. The energy absorber system will absorb the impact energy of the vessel without damage to the vessel or the coffer cell structure. Impact loading was analyzed using standard methods presented in Military Handbook, Piers and Wharfs, MIL-HDBK-1025/1,30 October 1987 superseding NAVFAC DM 25.1, November 1980. Loading has been analyzed as a berthing load such that the structure and vessel do not sustain damage from the impact.

An impact load perpendicular to the cofferdam was determined and then reduced to the angle component based on the geometry of the cofferdam in relation to the flow of the river and probable direction of approach from a vessel. The berthing energy was determined using the kinetic energy method, which is the recommended method for naval piers and wharves. The berthing energy was then used to size the appropriate energy absorbing system. One example is the Pneumatic Rope Type Marine Dock Rubber Bumper Fenders. Several sizes, models and capacities are available, and selection of the appropriate size will be made and included in the design following velocity measurements obtained from the hydraulic model studies.

The selected fenders will be mounted from the coffer cells in a continuous or evenly spaced arrangement along the river arm cells and allowed to move up and down as the river changes elevation from the low water to high water periods. In general, the fenders have been provided to protect the upstream and river arm cells from runaway barges. No fenders will be provided on the downstream arm cells since its unlikely they will be impacted.

13.2.11 Cofferdam Removal

The cofferdam will be removed upon completion of the headworks gated structure with the lift gates fully functional. Removal of the cofferdam cells, arcs and MRL tie-in walls will begin with the removal of cell 12 and proceed upstream and downstream utilizing two cell removal crews and two connecting arc crews. One crew will proceed upstream to cell 7 and the second crew will proceed downstream to cell 15. The crews will then continue removing cells working toward the tie in cells at the MRL. Sheet pile will be cut off at 2 feet below the finish grade line utilizing divers to cut the sheets for removal. Final design grading and placement of stone protection per the contract drawings will follow the removal of the cells and connecting arcs.

13.2.12 Cofferdam Instrumentation and Monitoring

Instrumentation of individual cells of the cofferdam will consist of survey monitoring points, inclinometers, and piezometers. Instrumentation drawings show the location and details of the instruments planned for the cofferdam. The survey points are to be located on the inboard and outboard of individual cells at the locations shown on the instrumentation drawings. The survey markers will be read daily by our survey crew to determine horizontal and vertical (settlement) movement of the cells. Red-line deflections will be established during the 90% design phase and will be provided to the Quality Control Manager through instructions to the field document. Inclinometers will be installed on cells 5, 9, 11, 13, 15, 17 and 19 to correlate with the top of cell surveys (See Drawings C-104 and C-501). The inclinometers will be installed on the outmost sheet of the cell, and readings taken twice a week will be compared to the baseline readings obtained prior to cofferdam dewatering and after initial dewatering. River surveys along the cofferdam will be performed monthly to detect any developing scour close to the coffer cells. Red-line deflections will be established during the 90% design phase and will be provided to the Quality Control Manager through instructions to the field document.

13.2.13 Cofferdam Flood Gates

Two flood gates approximately 16 feet wide and with a sill elevation of 8.5 feet will be provided for the cofferdam in the event river stages, from a hurricane surge, higher than 16.4 occur during construction of the Headworks structure (See Drawings S-220 through S-223). The flood gates will consist of a concrete paved sluiceway, steel beams, timber needles and a riprap protected spillway. The floodway will also include a perimeter of sheet pile that will extend from the top of the cell stability berm to the base of the stability berm. The sheet pile walls will extend to 5 feet above the top of the stone protection to be placed at the spillway. The interior of the floodway will be lined with geotextile and then filled to the top of the sheet pile, with 400-pound riprap stone. Erosion on the sides of the spillway will be controlled by use of sheet piling. The flood gates will be located at arcs 15a and 16a on the downstream arm of the cofferdam. The gates are designed to enable filling of the cofferdam within 24 hours from elevations 0.0 to 16.65. The design of the flood gates is provided in **Appendix J.3**.

13.2.14 Dewatering System and Under Seepage Control

A pump test program was completed in March of 2020 which consisted of pumping a deep well and a series of educator wells at the project site (See **Appendix J.7**). The data obtained has been evaluated and a report of the findings was completed. The pumping test provided data to determine the permeability values for the coarse point bar material and SM/CL material. Borings were obtained to determine the stratigraphy delineating the coarse materials from the fine materials.

The deep well test program involved the installation of nineteen (19) piezometers at distances varying from 10 feet to 101 feet from the pumping well, which was constructed similarly to the planned temporary dewatering wells. Two (2) primary tests were performed, a specific capacity test and a 72-hour single well pumping test; recovery following the 72-hour pump test was also monitored for a period of 24-hours via transducers.

The eductor test program involved the installation of six (6) piezometers at distances varying from 10 feet to 52 feet from the eductor alignment. Three (3) primary tests were performed, a 24-hour single unsealed eductor test, a 24-hour single sealed eductor test and a 72-hour multi-eductor (eleven (11) eductors) pumping test (unsealed); recovery following each test was also monitored for a period of 24 to 72 hours via transducers. All testing was monitored via transducers with periodic manual readings.

The results of the tests are summarized as follows:

- The single deep well test analysis yielded permeability values between 0.011 – 0.018 ft/min.
    - The radius of influence (ROI) of the deep well test was ±2,400ft; ROI was determined via Distance-Drawdown graph.
    - The test also provided information regarding the two aquifers (fine point bar and coarse point bar sands). It is clear the aquifers are hydraulically connected, and the overlying fine point bar sands present more of a leaky or unconfined aquifer condition rather than a confining condition.
    - The average flow during this test was 115 GPM.
- The 24-hour unsealed eductor test yielded a permeability of 0.00039-0.00045 ft/min.
    - The radius of influence of the test was ±240ft; radius of influence (ROI) was determined via Distance-Drawdown graph.
    - The average flow during this test was 5.19 GPM.
- The 24-hour sealed eductor test yielded a permeability of 0.00044 ft/min.
    - The radius of influence was ±200ft; ROI was determined via Distance-Drawdown graph.
    - The average flow during this test was 5.19 GPM.
- The 72-hour unsealed eductor test yielded a permeability range of 0.002-0.00037 ft/min.
    - The radius of influence was ±3,200ft.
    - The average total flow during this test was 38.5 GPM; maximum flow of 53.12 GPM at the start of the test and a minimum flow of 31.86 GPM at the end of the test.
- Flow during 24-hour single eductor tests began at a flow rate of ±8 GPM and declined as each test proceeded.
- Flow during the 72-hour multi-eductor test began at ±53 GPM and declined as the test proceeded.
    - It is unclear if scaling/clogging of the eductor heads or decreasing water levels in the well(s) resulted in the decreasing flows as the tests progressed. However, water quality data indicates fouling (scaling/clogging) of the eductor heads and some incrustation at all wells should be anticipated.

The dewatering system is designed with the deeper and higher capacity wells located around the perimeter of cofferdam transitioning into the low-capacity wells. Two levels of well points are planned to be installed as the excavation is carried to the -35 elevation. The dewatering system is designed to maintain the water level to EL -35. A maximum water elevation of 16.4 in the Mississippi River will be used to design the system for a head of approximately 51 ft. Additional pump capacity will be incorporated into the system as needed to assure drawdown required levels are maintained.

The dewatering system (See Drawings C- 125 thru C-131) will consist of 26 shallow deep wells jetted to 100 feet deep; 33 deep wells, drilled to 130 feet deep; 165 eductors installed to 80 feet deep; and wellpoints spaced at 6 ft. to 7 ft for a total of 2,205 linear feet. The initial drawdown pumping rate will be approximately 8,900 GPM diminishing to a steady state pumping rate of 2,500 GPM. The system flow rate will be impacted by river levels. To minimize initial flow the dewatering system startup will be sequenced to bring equipment online as it is available.

All wells will be plugged and abandoned in accordance with requirements contained in the Guidance Manual for Environmental Boreholes and Monitoring Systems, Prepared by Louisiana Department of Natural Resources and Louisiana Department of Environmental Quality, April 2020.

13.2.15 Construction Activities Inside the Cofferdam

The cofferdam is designed to permit pile driving and other construction operations inside the cofferdam containment area at all river elevations up to elevation 14.65 at the Alliance gage. The cofferdam is also designed for installation of the dewatering system up to a river elevation of 16.65 feet. Construction activities will utilize the river elevations recorded at the Alliance gage to guide construction activities inside the headworks excavation. These activities are, but not limited to, excavation, deep soil mixing (DMM), pile driving, and concrete placement.

13.3 Overview of Sequence of MBSD Project Construction

Generally, construction will commence with survey layout, mobilization of equipment and materials, clearing and development of access roads into the site. For discussion purposes, the Linear Schedule explains the work sequence relative to the Landside Work and Riverside/Marine Work (See **Appendix J.11**)

Landside Work- Interim Levee:

Construction of the earthen interim levee (IL) on the east side of Highway 23 (Hwy 23) will begin immediately after clearing of the land and relocation of the NOGC Railroad track.

The Interim Levee will be built using suitable on-site material that is processed, placed, and compacted in controlled lifts.

Once the levee alignment is cleared, levelled-off, and the IL control-line established, high-strength geotextile material will begin to be placed on the footprint of the levee alignment. As the geotextile is placed, the first course of levee material will be placed and compacted, with settlement plates installed as the operation proceeds. This placement process will start at Station 0+00 of the IL alignment, and progress along the length of the IL until reaching the Station 35+00 (see Drawings C-110 and C 111). Subsequent courses of levee embankment will be placed starting again at Station 0+00, and then progressing linearly to Station 35+00, Four total courses of embankment are anticipated with base course material placed on the final lift. Once the IL has been brought to EL 18 and is accepted, the first stage of the dewatering system will be activated as excavation for the Headworks proceeds to EL-35. Landside work relative to the construction of IL which includes initiating excavation of the Headworks excavation will begin at the same time as the marine work consisting of the cellular cofferdam, trestle, and fleeting area.

Landside Work - Balance:

Slope inclinometers and piezometers will be installed for monitoring the excavation slopes prior to excavation of the Headworks area. Settlement plates will be installed at select location along the interim levee as the construction proceeds (See drawing C-104).

Excavation of the Headworks area will follow the installation of the dewatering system. The Headworks area will be excavated to the final elevation of EL-35, which is the bottom of the stone aggregate that will cover the work area to provide a stable work area for pile driving and other construction activities. The Intake U-Wall structure will then be constructed beginning with the sheet pile cutoff walls, then support foundation piles, followed by foundation footings and intake walls. Gate structure, transition walls, and levee flood walls will proceed in the same sequence. As the Intake structural elements have been completed, the headworks excavation will be backfilled with controlled lifts to final grade. Project Plan Sheets 3013C101-C103 show the general sequence of backfilling. Backfill material will be placed as required by the contract specification.

On the west side of Hwy 23, the wick drain system, beginning with the sand drainage-blanket, will commence as soon as the initial clearing and construction of haul roads are complete. The general progression of work will start from Hwy 23 moving toward the Barataria Bay and will advance concurrently along the north and south sides of the channel. Wick drains will be installed by a specialty ground improvement subcontractor utilizing tracked rig equipped with vertical drilling attachment. The drill shall utilize a slender mandrel sized for the as-designed wick drains. Given the soft soils the drains will be installed under static load as opposed to dynamic loads used for harder soil conditions. The pattern, depth and spacing of the wick drainage system shall be in accordance with the design drawings and specifications. Following the installation of the wick drains the sand blanket will be installed by pushing sand material meeting specification with a dozer to the as-designed thickness and horizontal limits. General sequence of construction shall be as follows:

- Geotextile separator fabric and approved field sand will be placed within the Guide Levee footprint. Dimensional and sequencing details are shown on Sheet 6013C203-6013C214 of the project plans, Wick Drains will then be installed as the sand blanket is placed with operations progressing concurrently on the north and south side of the conveyance channel as follows:
- Install and secure geotextile separator on existing soils
- Install sand work base to design elevation, sloped to drain
- Install wick drains to design toe and spacing, terminate at 6 inches above sand base
- Install balance of sand blanket to design elevation in footprint of levee

Once the wick drains are sufficiently advanced, installation of cut-off sheet piles will follow. As shown on project plans, they will be driven along the centerline of the respective guide levees prior to adding subsequent layers of separator fabric and clay levee material.

As the wick drain blanket system is installed the excavation of the proposed conveyance channel will begin with in the dry with excavated material processed and placed on the drainage blanket to begin the surcharging/consolidation period for the guide levees. Successive layers of suitable clay levee material will be placed and compacted up to the final design elevation for the guide levees, plus overbuild for future settlement. Monitoring instrumentation will be installed as the levee is built, and a 4-month surcharging/settlement period is currently predicted before additional fill is added on top of previously completed lifts. The final lift will include the roadway section for the levee access road.

As shown on Project Plan Sheet 6013C202, the conveyance channel excavation will be staged, with in-the-dry excavation followed by a dredged phase to complete the channel cross section. The dredged material will not be used as levee material and will be disposed of as excess material and placed in location(s) specified by CPRA.

Armoring of the conveyance channel will follow the final grading of conveyance channel, with In-the-Dry and In-the-Wet stages of placement.

Siphon

Siphon Construction will begin once access roads west of HWY23 are built, allowing construction equipment, and materials into the general siphon area.

Construction of the siphon will be staged, with notes as shown on sheets C106-C107, and is scheduled to take 18-24 months to complete. Once completed, Timber Canal drainage will be diverted just upstream from the siphon, through the siphon, and then reconnected to the canal downstream from the siphon.

Soil improvement using DMM in the areas parallel and outside the area of the pipes to provide ground support for construction equipment as well as stabilize the side slopes for the deep excavation, will take place during the early stages of the project. General sequence of construction shall be as follows:

- Install and allow cure time for proposed DMM at future side slope excavation
- Excavate for pipe installation
- Fly-in pipe sections with suitably sized crane and set and align on dunnage to allow access to weld joints below pipe
- Make-up pipe connections weld/grout ring per manufacturer recommendations
- Backfill in accordance with the design plans and specification, may require flowable fill up to spring line and interior spacing between adjacent pipes

Once the guide levees and flood protection T-walls along the length of the conveyance channel have been brought to grade, the proposed NOV-NF-W-05a.1 Levee will be removed at the location of the crossing with the MBSD channel. This will ensure that protection from back flooding will be always maintained. The channel will then be connected to the bay with water introduced for final dredging and armoring of the conveyance channel. Dredged material will be transported to beneficial use areas designated by the CPRA for placement. Final levee roads, armoring and turf protection will be completed along the length of the MBSD.

Riverside/Marine Work:

Once materials are delivered and marine equipment mobilized, work on the Riverside will begin with the construction of the cellular cofferdam, and temporary trestle. Detailed description of the cofferdam construction sequence is included in previous sections of this report.

Once the cofferdam is completed and tied in to the existing MRL and the IL is tied into the existing MRL, the contained area between the cofferdam and existing MRL will be dewatered as described in Section 13.2.14.

Demolition and removal of revetment, sloped paving and the existing MRL within the temporary tie-in limits will follow with excavation to working grade prior to installation of the permanent piling and concrete features. The IL will remain in-place until the final concrete structural features are in place, including operational gates, affording full permanent-grade protection to Plaquemines Parish and Hwy 23.

13.3.1 Headworks Construction & IL Sequencing Description

The following paragraphs describe the sequence of landside construction in the Headworks area, including the IL.

Phase 1 of the Headworks Area (Land):

The construction of the IL, which will serve as the main line of protection during construction of the MBSD Headworks, is the key element of Phase 1. The IL will be in place and fully functional as the main line Mississippi River flood protection before the MRL will be breached to initiate construction of the intake structure.

Phase 1 starts with clearing and grubbing of the area immediately west of the MRL, construction of access roads, and the footprint layout for the IL. The IL will be built up over a 9-month period in various lifts to achieve the target build elevation at the end of construction. The stages will be built up with successive courses (12 inches loose/8 inches compacted) to soil density specified in the contract specifications. All IL lifts will be built from excavated and tested on-site material.

The first lift of the IL will comprise the stability berm for the earthen levee. The successive lifts will form the authorized MRL design levee grade and section. All material used will be processed and density controlled as per USACE standards. Levee construction will be accomplished using suitable on-site material that is excavated mechanically from the conveyance channel footprint and trucked-in with off road articulates and spread using dozers, compactors, motor graders for placement. Included in the levee construction will be instrumentation to measure foundation settlement and strength gain between stages and of the completed levee.

Phase 1 concludes with the acceptance by CPRA of the IL, for compliance with design and construction requirements. The levee will then serve as full protection during subsequent Phase 2 of the Headworks construction.

Phase 2 of the Headworks Construction:

Phase 2 consists of the breach of the existing MRL at the specific location of the MBSD Headworks and ensuing elements for construction of the proposed Headworks Structure for the MBSD. The IL will remain in place during the construction of the Intake Structure and until the Headworks structural features have been completed.

Construction of Phase 2 will require sloped excavation and dewatering of the Headworks area to begin construction of the Intake Structures. Final excavation elevation will be -35 at the bottom of the granular base. Dewatering of the Headworks Excavation area will be in two stages and occur prior to and throughout the excavation period, estimated to be 36-42 months.

Headworks Features:

<u>Intake and Gate Structures</u>

After reaching final excavated elevation (EL-35), construction of the Headworks will begin with placement of a 12” layer of rock over the footprint of the Intake and Gate foundation slabs, providing support for pile driving equipment. Permanent sheet pile cutoff walls will be driven as shown on Drawing 4015S301 of the project plans. The pipe piles supporting the Intake and Gate Structure base slabs will then be delivered and driven as laid out in the Project Design Drawings (4015S301). As the piles are completed, a concrete work slab will be poured to support rebar mats, formwork and equipment needed for construction of the foundation slabs. Perimeter foundation slabs will be poured first with backfilling of the slabs proceeding along the perimeter as the slabs are completed (See Drawings C 112-C-113). The exterior walls of the Intake and Gate Structures will be formed and poured as the foundation slabs progress. Backfill sequence of the exterior walls will be as shown on Design Plan Sheet 3013C101. Interior walls for the Intake and Gate Structures will be poured once the exterior walls are completed. Pour sequencing and individual pour sizes for the base slab are still being determined and will be finalized following the outcome of the concrete mix design batch testing to evaluate mass concrete characteristics. Based on the design criteria it is understood that the completed "U" section must be established with adequate strength gain before backfilling can commence. On-site batching of concrete is anticipated with truck delivery to either concrete pumps or conveyor system or combination thereof.

Transition Walls

The Transition Wall area will generally follow sequentially behind the construction of the Intake and Gate Structures. DMM ground improvement will be required in the Transition Wall area. As the stair-stepped wall foundation grades are established, DMM operations will begin with mixing-plants providing pumped grout to the multi-axle mixer rigs. Pipe pile driving for the wall foundations will follow behind the DMM operations and concrete work slabs poured as the pile progress. As the Wall foundations and walls are completed, backfilling will follow the sequence shown on plan sheets 3013C101-3013C103.

Flood Walls at the MRL:

The concrete Flood Walls will be built once the Intake structures are complete and the intake area is backfilled to the specified grade. This will be one of the final concrete structures built within the Headworks.

As the Headworks Structure is completed, the structure and excavation will be backfilled, and the dewatering system withdrawn in a controlled manner. Soil processing and compaction equipment used for the back-filling operation will typically consist of dozers, pad-foot compactors, etc. Backfill material will be placed as specified by the contract specifications.

Phase 3 of Headworks Construction:

Phase 3 consists of removal of the IL once the Intake Structure is complete and approved. The final structural elements and gates of the Intake Structure will provide permanent protection for the Plaquemines Parish at that time.

13.3.2 Interim Levee

The IL crown elevation will be maintained above the levee design grade (EL 16.65), NAVD88 to match the MRL project flowline (EL 12.65 feet., NAVD88) plus freeboard (4.0 feet). Due to predicted settlement during the duration of construction, the IL grade will be overbuilt in increments to elevation 18.0, NAVD88 and the landside slope will be constructed to 1:3.5 slope, which will settle down to 1:4 slope. The interim levee elevation will be monitored during construction, and material and equipment will be readily available to add fill back to design grades, as needed if settlement below EL 16.4 FT, NAVD88 occurs.

Settlement of the IL is projected to be 12-20 inches during the three (3) year period when it will serve as the MRL. This estimated settlement will be included when determining the required volume of levee fill required to build the IL. Total long-term settlements (immediate, primary, and secondary) were estimated to be between 33 and 48 inches if the IL were to remain indefinitely. The calculation package for these analyses is included in **Appendix J.2**.

The IL will be instrumented with piezometers, slope inclinometers and settlement plates and will be monitored throughout construction of the headworks (See Drawing C-104). EL 15.5' will be the triggering threshold for providing additional lifts of fill on the IL. Should the assumed project schedule differ significantly from that which was analyzed, additional analyses may be required to refine settlement estimates. The overbuilt IL was the governing case for slope stability analysis since the subsoil would not have gained strength due to consolidation.

13.3.3 IL Levee Crossing & Levee Access for Plaquemines Parish Government (PPG) During Construction

Crossing of the IL will be limited to the area where the IL crosses the project center base line at Station 47+00, where access to the interior of the inlet excavation will require a ramp from the crest of the levee to the prevailing ground surface. Mobile equipment will cross the levee at this location to access the excavation for all construction activities related to, but not limited to, removal of excavation material, installation of the dewatering system, conveyance of construction equipment and delivery of construction materials. Slope stability analysis of the MRL at the location of the trestle and the IL has been performed and is included with the IL analysis.

Construction equipment anticipated to cross the MRL at this location will include Off-Road End dumps, Crawler and Hydraulic Cranes, Tractor-Trailer Dray Trucks, Rubber Tire loaders, etc. More equipment information and definition will be known as the pre-construction process evolves.

Access along the levee crown will be continuously provided for levee maintenance, security patrols and accessing the cofferdam for reading the instrumentation located on the cells. Since construction of the channel is going to disrupt current access along the current MRL crown, the IL crown will connect to the MRL and include continuation of the crown road allowing access around construction.

13.3.4 Armoring

Armoring of the cofferdam will be as shown on the drawings with a rock berm on the outside of the cells. Armoring of the IL will not be required because of the levee being set back where river currents are less than 2 ft./sec. As such the IL will be vegetated to control the erosion of the slopes. Raising the IL as it settles will be best accomplished with slopes that are only vegetated and not covered with armoring stone. Stone protection placed along the cells will be carried up the slope to the crest to protect the existing MRL at the intersection of the seepage cut off wall and end cells 1 and 23.

13.4 Excavation and Installation of the Deep Mixed Retaining Structure for the Siphon Under the Channel

Siphon Construction will begin once access roads west of HWY23 are built, allowing construction equipment, and materials into the general siphon area. Construction of the siphon will be staged (See Drawings C-106 and C-107) and is scheduled to take 18-24 months to complete. Once completed, the Timber Canal drainage will be diverted just upstream from the siphon, through the siphon, and then reconnected to the canal downstream from the siphon. The approach channel (up-stream of siphon) and discharge channel (down-stream of siphon) will be excavated and graded per design with a temporary plug (earthen or temporary sheet pile) left in place at the connections to the Timber Canal. Once the siphon structure is completed both plugs will be removed allowing the diversion of the Timber Canal through the siphon structure. The abandoned portion of the canal will be excavated/filled in accordance with the design documents.

The excavation for installation of the siphon pipe(s) under the channel will be accomplished by open cut excavation from the existing ground surface to the channel invert (EL -25). The excavation to elevation -39 will be a vertical cut through the DMM portion of the excavation. The DMM will be installed 35 ft wide to a depth of -45. Side slopes of the excavation from natural ground to EL -25 are projected to be 1V on 9H, resulting in a construction case factor of safety of 1.3. The width of the excavation will accommodate the required width for the siphon pipes plus 40 feet on each side to accommodate equipment access. Positive dewatering of the groundwater is not anticipated because of the homogeneity of the clay foundation and cutoff wall because of the deep mixed soil.

13.5 Cofferdam Structures for Highway 23 Bridge Pier Installation

The excavation for installation of the bridge piers will be accomplished with internally braced sheet pile cofferdam structures that will be installed to facilitate driving of the foundation piling to a tip elevation of -130. The interior of the cofferdams will be excavated to an elevation of -28 to -38 for the different cofferdams prior to driving the foundation piles. Sheet piling will either be removed or left in place and cut off 3 feet below grade line. An unwatering system will be installed to control surface ground water in the excavation as needed but a dewatering system is not anticipated. Calculations and general layout are provided in **Appendix J.6**. The structural design of the braced cofferdams for the bridge piers will be included in the 90% design submittal.

13.6 Temporary MR Trestle Dock

Transfer of barged materials delivered on the River will be by means of the temporary dock and fleeting area constructed immediately downstream from the cofferdam. Preliminary details of the pile supported trestle area and levee crossings is in **Appendix J.4**. Off-loading of aggregates from the barges will be done by excavator(s) working from the dock, and top loading directly onto dump trucks. Dump trucks will then transport the materials to the appropriate locations for final placement.

The trestle will be located downstream of the cofferdam and will provide access for offloading of materials. The temporary trestle foundation is designed using standard procedures for design of pile foundations as presented in USACE, EM 1110-2-2906, Design of Pile Foundations. Soil-pile properties were determined during the design process of the trestle. Loads to be supported were determined based on size and capacity of cranes, lateral loading of barges containing construction materials as well as other equipment that will access the trestle during normal construction activities. Steel pipe pile will be the preferred foundation support and a factor of safety of 2.5 for compression and 3.0 for tension has been applied to determine individual pile capacity for the structure. PDA testing would confirm the design pile capacities for the trestle piles. Calculations and details for the trestle are in **Appendix J.4** (See Drawings S-201-S-212).

General construction sequence for the Trestle shall be as follows:

- Pre-excavate pile drive lines to identify and remove any obstructions
- Installation of pipe pile, either by marine based piling equipment or land based (top-down method) or some combination thereof depending on factors including schedule requirements, cost, sourcing and delivery of pipe pile
- Installation shall be by impact hammer, with PDA. Pile to be installed to design tip elevation as determined by the Engineer of Record and based on approved pile test program criteria
- Install structural steel stringers and cross channel members
- Set precast deck panels
- Install battered timber fender pile
- Install deck curb timbers and safety railing

13.7 Maintenance of Back Hurricane Protection

The planned NOV-NF-W-05a.1 Levee will be constructed prior to scheduled construction of the Guide Levee and T-walls for the MBSD Project. It will be maintained and provide back flood protection until full protection has been provided by the MBSD Project elements (Guide Levees and T-Walls). The MBSD Guide Levees will tie into the NOV-NF-W-05a.1 Levee at the points where the two levees intersect along both sides of the Conveyance Channel. The NOV-NF-05a Levee will then be removed in between the two Guide Levees once they are brought to design grade and the T-walls are complete. Storm water drainage conveyance along the Timber canal will be maintained until the siphon is operational (See Drawings C-108 and C-109).

13.8 Channel Excavation & Guide Levees

Channel excavation and Guide Levee operations will run concurrently, with material excavated from the channel processed and then used as levee material. All excavated material proposed for use as levee material will be tested for conformance with the requirements of contract specification for backfill.

<u>General Sequence:</u>

Once the areas of the proposed Channel and Guide Levees are cleared and haul roads are in-place, Geotextile separator fabric and approved field sand will be placed within the Guide Levee footprint. Dimensional and sequencing details are shown on Project Plan Sheets 6013C203 thru Sheet 6013C213, Wick Drains will then be installed as the sand blanket is placed with operations progressing concurrently on the north and south side of the conveyance channel. Once the wick drains are sufficiently advanced, cut-off sheet piles will follow on. As shown on project plans, they will be driven along the centerline of the respective guide levees prior to adding subsequent layers of separator fabric and clay levee material. Monitoring instrumentation will be installed as the levee is built, and a 4-month surcharging/settlement period is currently predicted before additional fill is added on top of previously completed lifts.

As shown on Project Plan Sheet 6013C202, the conveyance channel excavation will be staged, including both In-the-Dry and Dredged phases, to complete the channel cross section. Armoring of the conveyance channel will be staged similarly, with In-the-Dry and In-the-Wet stages of placement.

The final lift will include the roadway section for the levee access road. Excess material excavated from the conveyance channel will be later permanently placed in location(s) specified by CPRA.

13.9 NOGC RR

The existing NOGC RR track will be removed at the beginning of construction, and then later replaced with a new approach and bridge over the MBSD Intake Structure. Temporary storage of rail cars during construction will be provided through construction of the temporary rail spur 4200 linear feet along the north side of the conveyance channel. Construction of the proposed replacement track will begin once the Headworks structures are complete and the intake bowl, including the MRL tie-ins, backfilled.

13.10 Reservation Area

The Reservation Area features will be built upon embankment placed during the early stages of construction. The embanked area will be allowed to settle for two years prior to installing piling, utilities and beginning foundation work.

13.11 Outfall Transition Feature including temporary storm protection

Note: The proposed USACE NOV-NF-W-05a.1 Levee is presumed to be either in-place or in-progress at the start of construction of the MBSD project. That levee would provide important storm surge protection from water overtopping the existing NFL back-levee. Once Guide levees are in-place, the OTF will be dredged starting from the Barataria Bay, through the existing back-levee, and east toward the NOV-NF-W-05a.1 Levee (See Drawings C-108 and C109).

Access for the dredging operations will provided from the west (See Drawings C-150 thru C-155). Dredging will be done with hydraulic and mechanical dredges. Dredged material will be pumped form the OTF to the final placement areas as shown on the Project Plan Sheets 8003C101-C104.

F2: Preliminary Operations Plan

CPRA's Preliminary Operations (Water Control) Plan

Mid-Barataria Sediment Diversion

This plan serves as the initial operating strategy for the Mid-Barataria Sediment Diversion. Applicable permitting agencies (e.g., CEMVN) should rely on this Water Control Plan (v.1) to define the operating thresholds and controls of the MBSD structure for purposes of the environmental impact analysis. CPRA acknowledges that this Water Control Plan is subject to revision and refinement as the engineering and design plans for the MBSD are further developed. As part of engineering and design, CPRA anticipates preparing a a more extensive Operations, Monitoring, Maintenance, and Adaptive Management document. This plan, however, provides adequate information to enable CEMVN and any cooperating agencies to evaluate the potential environmental consequences of the proposed MBSD project.

Operation

Standard Operational Triggers

When the Mississippi River (MR) discharge at Belle Chasse exceeds a value of 450,000 cubic feet per second (cfs), CPRA will open the diversion for full operations. The structure will be opened to pass the maximum amount of water considering the water elevation difference between the MR intake and the diversion outfall in the receiving Basin. The MR discharge of 450,000 cfs will be the standard operations "trigger" for the MBSD; however, future operations criteria may be modified under the Adaptive Management strategy. Conversely, full operations (with the exception of base flow, discussed below) will cease when the MR discharge falls below 450,000 cfs or when certain other stop triggers or "Emergency Operations" are met (below). An example of triggering events over a range of annual hydrographs is shown in Figure 1.

Figure 1. Mississippi River hydrographs showing several standard operations periods

The discharge of the structure will vary with discharge of the MR, realized as stage or elevation. The elevation of the receiving Basin will also modify diversion discharges. The structure is designed to discharge 75,000 cfs when the MR is at 1,000,000 cfs. The diversion structure will be closed when the relationship between the water levels in the MR and the Basin would create reverse flow.

Base Flow

To protect newly vegetated or recently converted fresh, intermediate, and brackish marshes near the diversion outflow, a background (base) flow would be maintained. The planned discharge of this base flow will be a not-to-exceed volume of 5,000 cfs. This minimum flow through the structure (s) would be maintained throughout the operations year to the maximum extent possible. This flow would be achieved, when possible, allowing for differences in water level between the MR and the receiving Basin.

Emergency Operations

Spills and other Hazardous Discharges

a. Mississippi River

In the event of a spill or unauthorized discharge requiring notification or other reportable release of hazardous materials upstream of the diversion intake with high likelihood to be imminently entrained, CPRA will cease operations immediately (i.e., CPRA will fully close the diversion gates). For spills or other hazardous discharges downstream of the diversion, a decision will be made regarding any changes in standard operations by the Operator in consultation with relevant agencies. Additionally, CPRA will cease operations upon learning that an imminent threat of a spill exists (vessel groundings, collisions, loss of steerage, etc).

b. Barataria Basin

For spills occurring in the receiving Basin, CPRA will assess the event and potential impacts in consultation with LOSCO or LDEQ and other relevant response agencies to determine what, if any, changes in diversion operations are warranted.

Navigation

In the event diversion operations cause an unintended and severe impediment to navigation, as determined by the US Coast Guard in consultation with CPRA, CPRA will coordinate with the US Coast Guard and CEMVN and determine what, if any, changes in diversion operations are warranted to address the impediment.

Climatic Conditions

CPRA will close the diversion gates and suspend all flows through the diversion when tropical activity (Depression or named storm) is forecasted to impact the Barataria and Mississippi River Basins. The structure will be closed in advance of storm impact to avoid affecting water levels in the MR or the Basin. Upon a determination that operations can resume, after passage of event, without unnecessary, unexpected impacts, the gates will be opened.

Structure Emergency

CPRA will suspend diversion operations if it becomes apparent that the diversion structure has suffered damage that presents a risk to operations or is not operating properly. CPRA will modify diversion operations until emergency repairs or corrective actions have been successfully implemented.

Public Safety

CPRA will modify or cease operations of the diversion in the event of a threat to public safety. Threats to public safety may include items such as a breach of the structure safety zones, threats to flood protection structures adjacent to the diversion channel, or other items deemed by CPRA or another governmental entity with jurisdiction to jeopardize public safety. Diversion operations would be modified until corrective actions have been successfully implemented.