APPENDIX F:
MBSD DESIGN INFORMATION

F1: Design Documentation Report (30% Design)

F2: Preliminary Operations Plan

F1: Design Documentation Report (30% Design)

STATE OF LOUISIANA

COASTAL PROTECTION AND RESTORATION AUTHORITY

MID-BARATARIA SEDIMENT DIVERSION (MBSD) PROJECT

STATE PROJECT No. BA-0153

LaGOV NO. 4400010386

Preparation of Engineering and Design

DESIGN DOCUMENTATION REPORT (DDR)

30% DESIGN

for

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0	11/15/2019	30% Draft Submittal

Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACB	Articulated Concrete Blocks
ACI	American Concrete Institute
AE	Architect 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
BUM	Beneficial Use of Excess Material
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
DDR	Design Documentation Report
E&D	Engineering and Design
EL	Elevation
EIS	Environmental Impact Statement
EM	Engineering Manual
ESA	Environmental Site Assessment
FN	Froude Number
FWCA	Fish & Wildlife Coordination Act
GEBF	Gulf Environmental Benefit Fund
GPS	Guide to Minimum Standards
Gr	Grade

HSDRRSDG	Hurricane and Storm Damage Risk Reduction System Design Guidelines
HVAC	Heating, Ventilation and Air Conditioning
Hwy	Highway
I&C	Instrumentation & Controls
I.C.E.	Independent Cost Estimator
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
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

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
QAQC	Quality Assurance Quality Control
RFI	Request for Information
RM	River Mile
ROE	Right of Entry
ROW	Right-of-Way
SAR	Safety Assurance Review
SCADA	Supervisory Control & Data Acquisition
SIBM	Settlement Induced Bending Moments
SLR	Sea Level Rise
SOV	Schedule of Values
sTons	Short Tons
SUE	Subsurface Utility Engineering
SWL	Safe Water Level
SWR	Sediment to Water Ration
TBD	To be determined
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
WBS	Work Breakdown Structure
WEAP	Wave Equation Analysis for Pile
WRDA	Water Resources Development Act

WSE	Water Surface Elevation

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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 60.7 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-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.

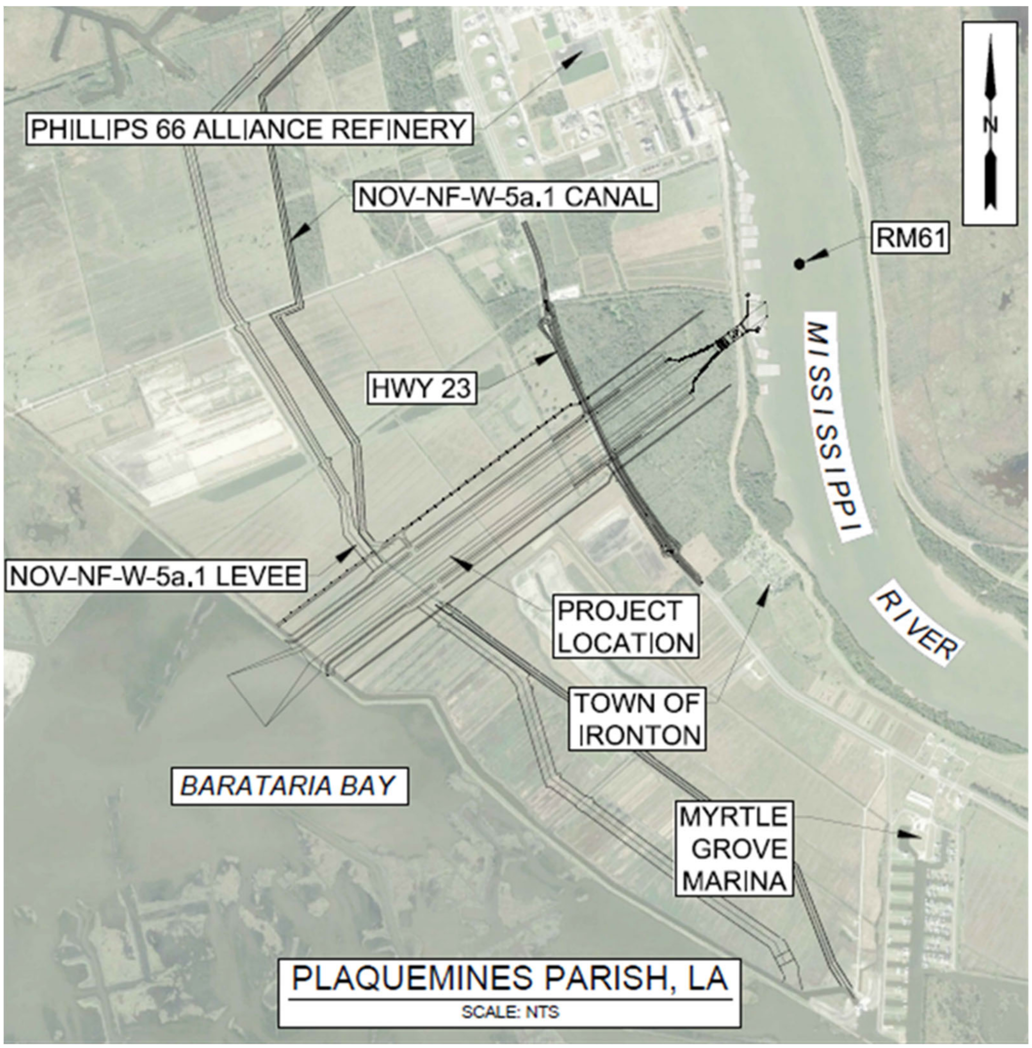


Figure 1-1: Location Map

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 conveyance feature includes an approximate 2-mile Conveyance Channel and guide levees that parallel the channel. The discharge component includes an Outfall Transition Feature 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, Utility Relocations, and Secondary Project Features such as support buildings and a boat ramp.

CPRA has structured 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 30% 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 an interim levee. 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 will request Section 408 Permission at the completion of 60% Design, which is anticipated for Fall 2020. This 30% Submittal is provided to USACE as a pre-cursor to the actual 408 submittal, intending to inform USACE regarding proposed design and providing an opportunity for USACE input early in the design process.

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 12A.

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

2.4 Datum Shift for Design

The USACE specified required design grades in the project vicinity relative to NAVD 88 2004.65, Geoid 09. When converted to the 2009.55 epoch, the cumulative difference in epochs and geoids at the project site required increasing specified elevations by a net 0.1 foot. The State of Louisiana, however, mandates survey data and specified elevations to be referenced to NAVD 88 Geoid 12A, which requires an additional conversion. To convert NAVD 88 Geoid 09 elevations to NAVD88 Geoid 12A at the project site, an additional 0.25 feet must be added to elevations referenced to Geoid 09. Therefore, all Design Grades established by USACE flood protection levels were increased an additional 0.25. For example, the MRL 50-Year Hurricane Protection Design Grade was specified at EL 20.1 NAVD 88 2009.55 Geoid 09, and this equates to EL 20.35 NAVD 88 2009.55 Geoid 12A at the project site. Water surface elevations computed by Design Team hydraulics modeling also reference Geoid 12A. Design Grades specified herein that are established based on DT-modeled water surface elevations plus a freeboard allowance already reference Geoid 12A and therefore do not require additional conversion.

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. The H&H analyses also guided the design of the siphon and pump that facilitate the drainage of the polders separated by the diversion channel.

The modeling performed to-date is at 30% E&D level. Additional numerical modeling is proposed for the subsequent 60% E&D phase to further evaluate design of the structural components.

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 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 transport 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.

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 and summarized in **Table 3.2-1**.

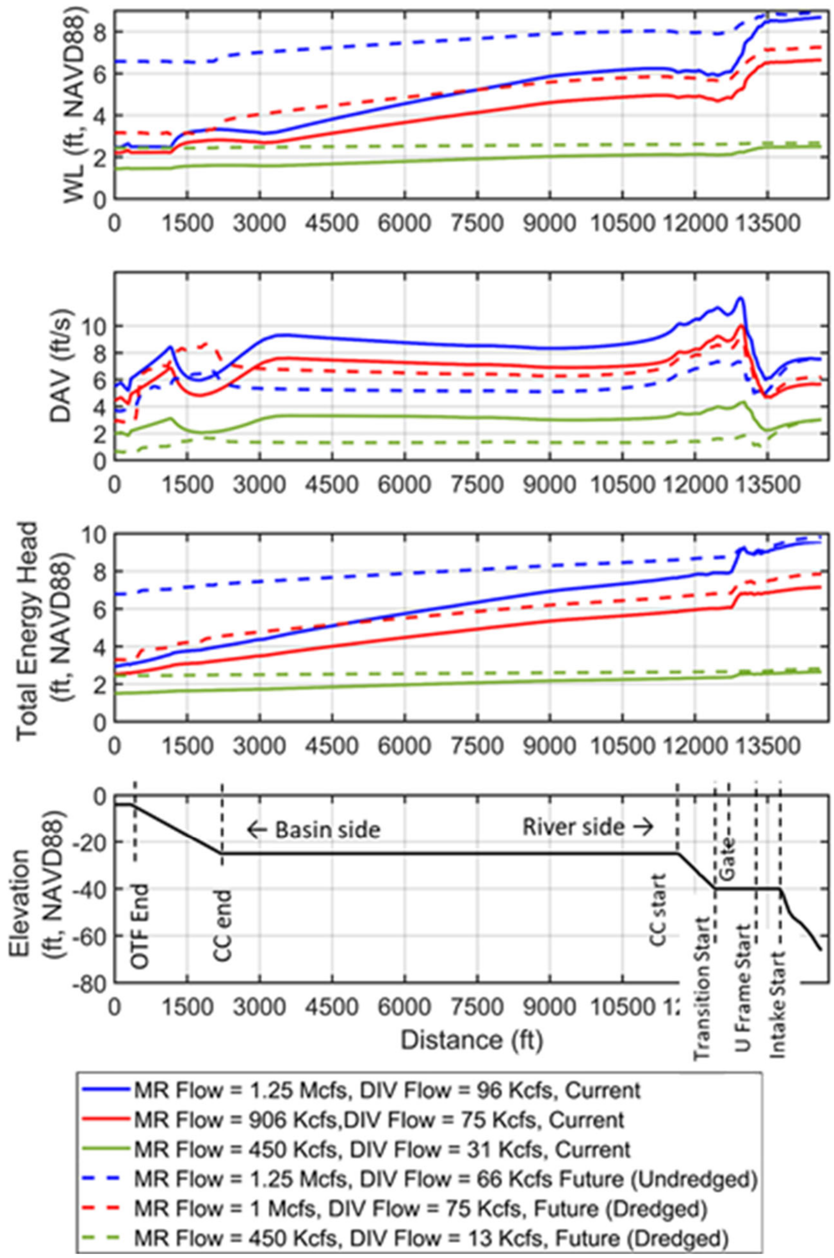


Figure 3.2-1: The predicted profiles of the water surface elevation, the depth-averaged velocity (DAV) and the total energy head along the length of the diversion system. Flow is from right (river end) to left (outfall end) in the above panels.

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

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 and river-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 rip-rap. 2. DAV within the river portion of the intake (river-side of U-Frame start) for rip-rap 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 rip-rap 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 rip-rap 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 at the moment 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 at the moment 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 bathymetry/ topography is from the CPRA land-building Basin-wide model.
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.

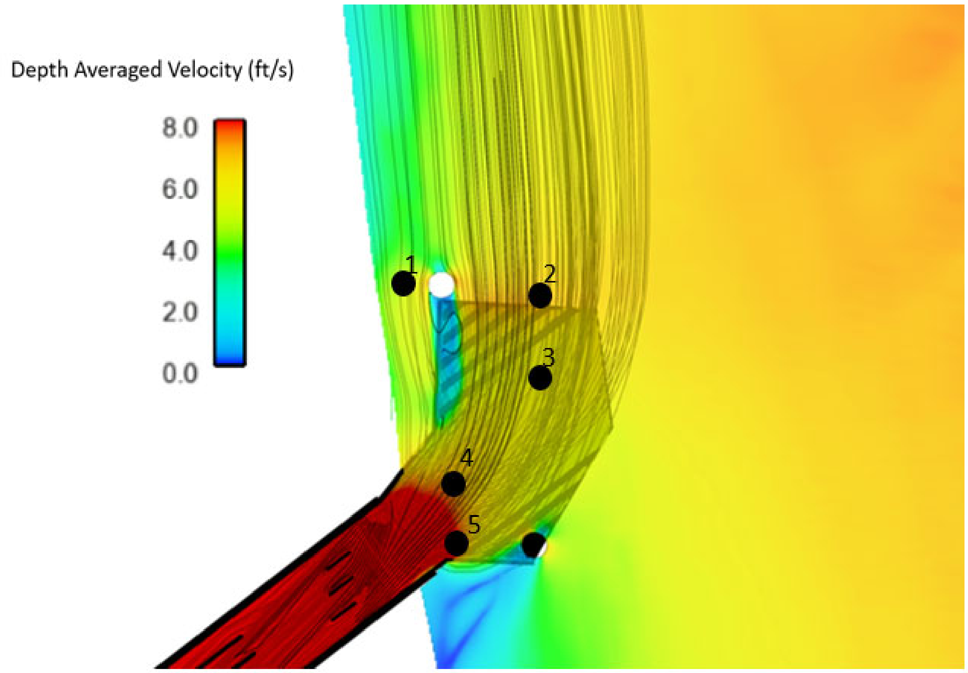


Figure 3.2-2: Depth Averaged Velocity (ft/s)

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	4.8	4.8	2.7
2	794070, 3285379	38	6.7	7.5	4.5
3	794065, 3285330	49	5.7	6.9	2.7
4	794006, 3285241	49	7.0	7.5	4.8
5	794036, 3285160	49	7.9	8.1	6.5

The 10-yr simulation of the local river morphology near the intake showed degradation in the upstream vicinity (**Figure 3.2-3**). This area will be investigated further in the 60% E&D phase through additional numerical and physical modeling, and experience of the subject matter experts. The sediment aggradation downstream of the intake will also be examined in more details.

Similar analysis will be performed to assess effects of cofferdams placement expected during construction phase.

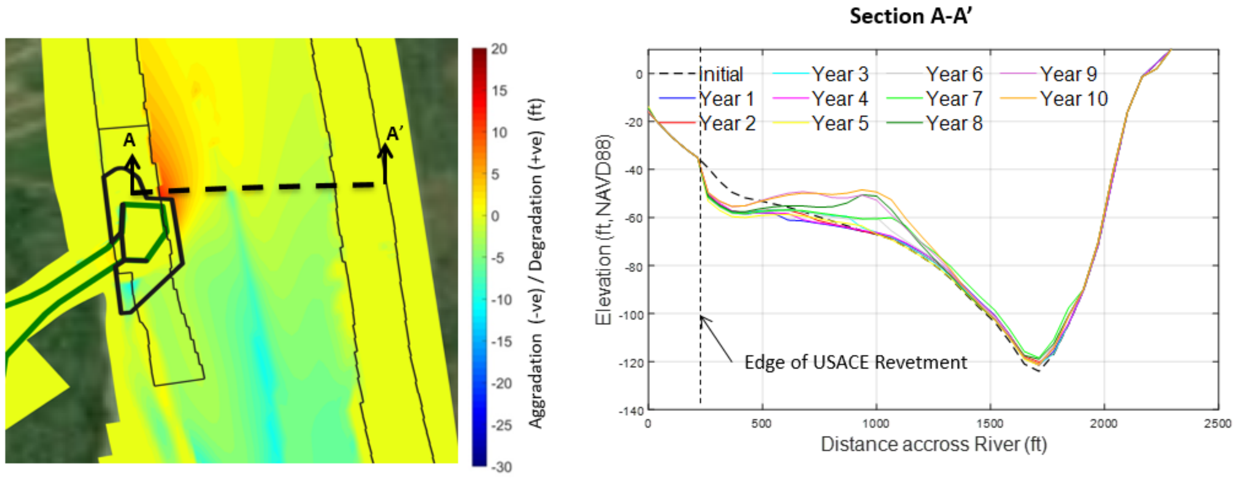


Figure 3.2-3: Morphological evolution of the intake vicinity

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-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 the diversion flows are based on the Belle Chasse Sand Load and Belle Chasse Hysteresis Sediment Rating Curves rating curves developed from measured data in the MR. The modeling process is fully documented in the Conveyance Channel Modeling Report which was previously submitted to CPRA.

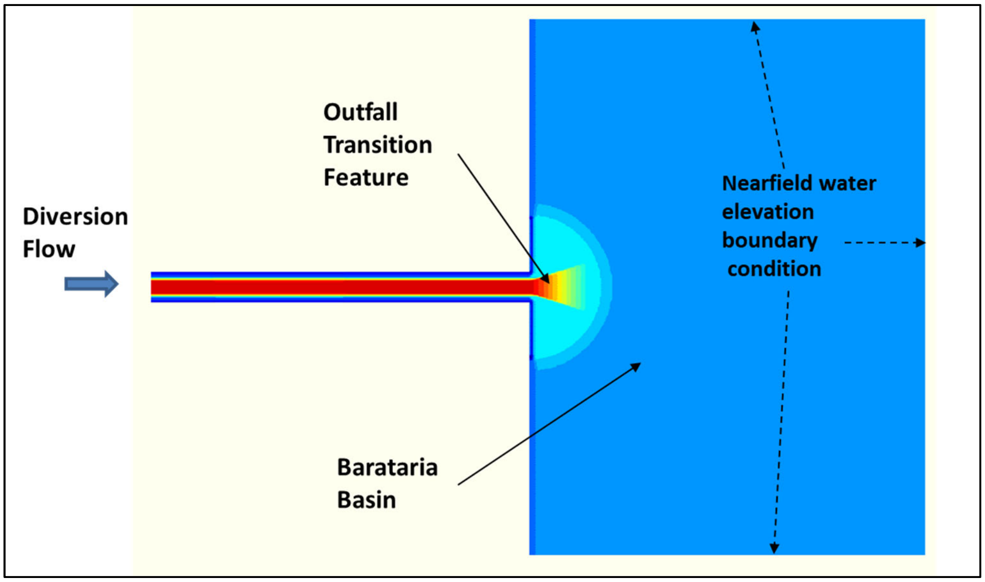


Figure 3.2-4: Delft3D Model Domain for Conveyance Channel Analysis

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

Morphology modeling of the Outfall Transition Feature (OTF) indicated that the scour to an elevation of -14 ft 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 3.4). 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 small over the two few 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 rip-rap 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 3.1 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.

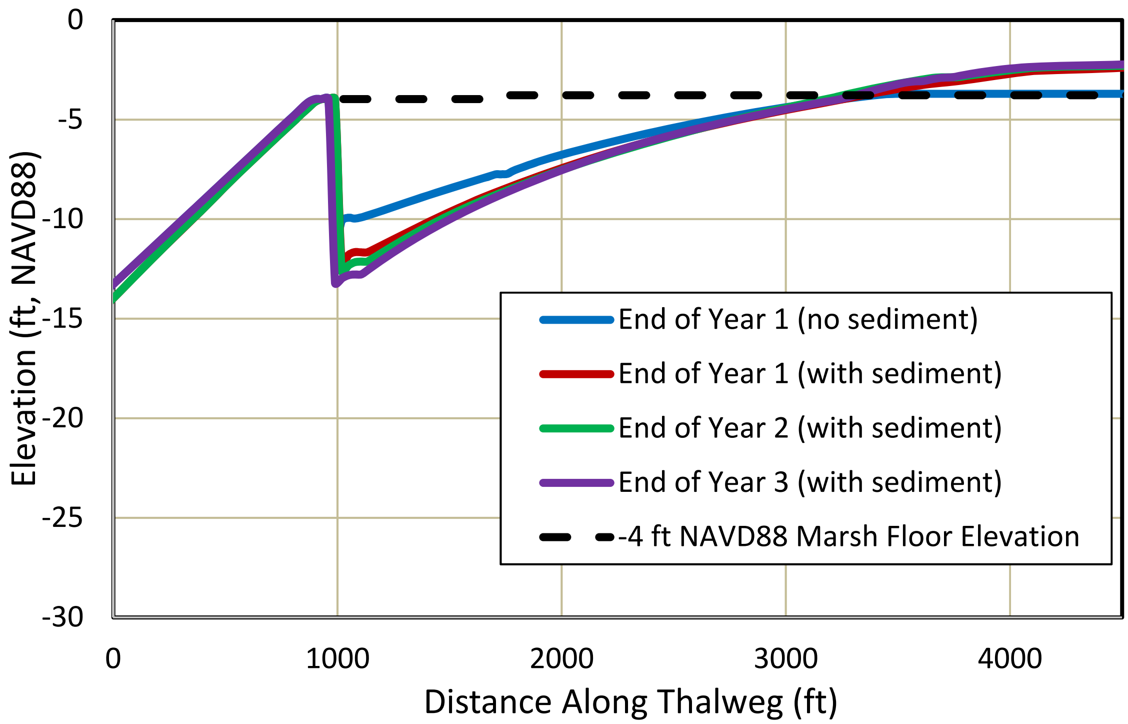


Figure 3.2-5: Predicted scour bed elevations

Figure 3.2-5 shows predicted scour bed elevations from with- and without-river-sediment runs for the 2,000-ft pulled back OTF. The maximum scour bed elevation at the end of 3 years is -14 ft NAVD88.

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

3.3 Physical Modeling

The construction and testing of a physical scale model are required for design of the MBSD. As of the 30% Design Phase, physical model construction has been completed at Alden Labs facility near Boston, MA, and testing is in progress. The MBSD is being designed with an extensive numeric and physical modeling program. The two modeling programs complement each other; each modeling approach (numeric and physical) contributes to developing a comprehensive understanding of how the system will perform. Numeric 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 numeric 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 numeric model. When combined, the two modeling approaches provided the highest degree of confidence in the development of the design.

As shown in **Figure 3.3-1**, the total desired physical model domain includes 12,500 feet of the Mississippi River, the diversion, the conveyance channel and a portion of the outfall transition. The conveyance channel is approximately 9,000 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 developed where the domain is split into two models. The need to maximize model size superseded the desire to have a single model. The 'Diversion Model' includes about 12,500 feet of the Mississippi River, the diversion and a short (~1000 feet) reach of the conveyance channel. The river is about 2,700 feet wide in the area of interest meaning the model is 4.6 channel widths long. The lowest elevation in the model is about -124 ft (NAVD 88). Based on USGS Belle Chasse gauge (7374525) the water level at 1,000,000 cfs is between 10 and 11 feet (NAVD 88). The maximum prototype depth is about 135 feet and the maximum model depth is about 2.1 feet. A more typical river bed elevation is between -60 and -90 feet, giving a model depth between 1 and 1.5 feet. The 1:65 scale Diversion Model is being used to determine the following performance characteristics of the diversion:

- Sediment water ratio
- Headloss through the gate structure
- Riprap stability in the intake approach in the Mississippi River
- Riprap stability in the transition between the gate structure and conveyance channel
- Cofferdam construction sequencing during construction
- Erosion and sedimentation around cofferdam
- Local river sedimentation downstream of diversion

Figure 3.3-1: Physical Model Domains (diversion intake & Mississippi River model yellow, conveyance model blue)

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 diversion model is a recirculating model without a sediment feed system; water and sediment discharged at the downstream end of the model are both pumped to the upstream end of the model.

A 1:65 scale model was constructed to test the conveyance channel and discharge into Barataria Bay. The purpose of the model is to:

- Confirm channel roughness
- Evaluate sediment transport in the flat channel
- Test riprap stability in the conveyance channel
- Test local scour in the outfall transition
- Qualitatively evaluate if bedforms will increase channel roughness
- Test riprap stability in the outfall transition.

The Conveyance Channel model uses a sediment feed system such that the inflowing sediment concentration can be defined. Water is recirculated after being filtered to remove any residual sediment. New sediment is added to the water to achieve the desired concentration.

Because both models have been built and are in testing, preliminary results from ongoing testing may be included to illustrate some concepts. The model scale is based on analysis of the grain size, approximate river depth and water surface slope. The model Froude number matches that in the prototype. The model has a live bed using lightweight sediment (plastic).

The physical models are designed to investigate the transport and diversion of sand size particles. Finer silt and clay size particles have been shown by others to have approximately a uniform concentration throughout the river cross section and the diverted amount of silt and clay is proportional to the diverted water for all diversion designs. Sand sized particles can move as bedload and suspended load. During periods of low flow, negligible sand transport occurs and the sand that is transported moves as bedload. During periods of high flow, sand is primarily transported as suspended load.

Model sediment was scaled to match the Rouse number (ratio of fall velocity to shear velocity) and Shields parameter (critical shear stress) in the model and prototype as closely as practical. Plastic sediment with a mean particle diameter of about 0.30 to 0.35 mm and a specific gravity of 1.08 was selected for the model. The proposed sediment is a commercially available product.

Scaling the model sediment is an imperfect science because of the presence of bedforms, river bends, etc. Therefore, a flume test was used to help confirm appropriateness of the selected sediment. A 2 ft wide x 20 ft long flume was used to determine if sediment on the flume bed will move up into suspension within the concentrations required to match prototype concentrations. Preliminary results from Conveyance Channel testing and Diversion Model testing show that the material will move in suspension.

Model instrumentation includes 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.

The sediment diversion being tested in the physical model was designed and optimized using numeric modeling conducted by the Design Team. Testing in the Diversion Model and the Conveyance Channel Model includes at least 12 tests with each model over a range of flows and sediment concentrations. The testing program also includes repeat tests to determine the repeatability (uncertainty) in the physical modeling program.

Both models have a live bed and used plastic sediment with a specific gravity of 1.08. Flume tests and subsequent model tests show that the sediment is moving in suspension as required. In accordance with Physical Model Plan previously prepared for CPRA, plans for measurements have been developed including the number of measurements and the type of instrumentation. The plan includes flexibility that is necessary to accommodate design changes and modifications that are being developed as part of the numeric modeling program.

The small flume test, first two river model tests, and first seven conveyance channel tests were completed, and they showed that the model sediment satisfies project expectations. The flume testing showed vertical concentration profiles similar to those observed in the Mississippi River. The river testing showed sediment concentrations that tend to be higher than observed sand concentrations in the river. Preliminary analysis of bedforms shows they are consistent with expectations, however, further analysis and comparison with prototype bedforms is required.

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

3.4.1 Inverted Siphon

3.4.1.1 General Description

The inverted 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 siphon has been established to be the conveyance of a 10-Year, 24-hour storm event with no significant upstream impact on water surface elevations within the drainage system.

3.4.1.2 Hydraulic Sizing Inverted Siphon Sizing

The inverted 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 no significant upstream water surface elevation impacts as compared to the pre-diversion conditions.

Total flow through the inverted siphon under this condition will be approximately 783 cfs with a velocity within the individual tubes of 2.71 fps each. The total headloss across the inverted siphon is 0.31'. Details regarding the inverted siphon structural and mechanical design can be found in Section 5.7 of this DDR.

3.4.2 Sluicegate to Drain 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 sluice gate structure was proposed to be located NOV-NF-05a.1 Levee that will 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 onto 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**.

At the end of 30% Design, CPRA advised that they intend to purchase this area between the existing back levee and the proposed NOV-NF-05a.1 levee, and therefore the area will not have to be drained. However, preliminary hydraulic design for a sluice gate drainage structure was already completed, and the drainage structure is shown on the 30% drawings and included in the Interior Drainage Report. In the 60% Design Phase, the drainage structure will be removed from the plans and the drainage design.

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 and redirects 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 from where the flow will proceed to the Wilkinson Pump Station 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.

North Parallel Ditch

Flow Capacity	266.8 cfs
Required Section	53.35 sf
Velocity	5fps
Bottom Width	10 ft
Side Slopes	3V:1H
Flow Depth	3 ft
Freeboard Req'd	1 ft
Total Channel Depth	4ft
Total Channel Width	34ft

South Parallel Ditch

Flow Capacity	261.5 cfs
Required Section	52.3 sf
Velocity	5fps
Bottom Width	10 ft
Side Slopes	3V:1H
Flow Depth	3 ft
Freeboard Req'd	1 ft
Total Channel Depth	4ft
Total Channel Width	34ft

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 as part of the review process for the bridge. The drainage report associated with the LA 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 will be designed by the Construction Manager at Risk (CMAR) utilizing the exploration data developed for the project and the DT's preliminary analyses from the 15% design effort. The DT will provide technical review of the CMAR's design efforts where appropriate.

The DT published a Geotechnical Data Report in April 2019 that included results of the soil design parameters that were approved by CPRA and the US Army Corps of Engineers (USACE) based on geotechnical data obtained in 2018. This data report was included in the Basis of Design Report (BODR) (15% phase) and forms the basis of our 30% design.

Detailed figures and calculation packages for the project features designed at this stage are included in our 30% Geotechnical Engineering Report dated November 15, 2019. This 30% Geotechnical Engineering Report still has geotechnical data gaps. Due to sustained, high Mississippi River stages for approximately one year, borings planned in the river and borings/CPTs planned on land within 1,500 feet of the MRL could not be performed until August 2019 (land work) and September 2019 (River work). The geotechnical exploration program performed in 2019 is not incorporated into this 30% DDR due to the timing of this Design Documentation Report (DDR) submittal, but will be part of the 60% design phase.

4.2 Intake

The Intake is comprised of the training walls, the U-frame and Gated Structure. The intake will be constructed in the open and dewatered excavation made to about EL -50. The slopes of the open excavation will be about 1V:7H. The DT assumes that backfilling will be done 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 of a few feet and high lateral loads on piles unless mitigation is provided such as Deep Mixing Method (DMM), lightweight fill, or a relieving platform. The DT considered controlled re-watering to maintain backfill in a buoyant condition which would reduce settlement and lateral loads but felt that this was not practical. Also, lightweight fill and a relieving platform were judged to be impractical or too costly.

For the 30% design level, the DT recommends that DMM be used to mitigate lateral forces and reduce UBL's and settlement to tolerable levels. Reducing settlement eliminates or reduces Settlement Induced Bending Moments (SIBMs) and drag loads on piles. For example, construction of the Training Walls on the river side of the MRL, requires excavation of the slope at the river side toe of the MRL. This excavation could destabilize the MRL and the future T-wall at its current alignment. AECOM considered moving the MRL landward to improve stability or to strengthen soils along the existing alignment. The recommended option is to maintain the existing MRL alignment and strengthen the soils below the T-Wall using DMM.

Please refer to the 30% Geotechnical Engineering Report for detailed presentation of soil parameters/stratigraphy and geotechnical analyses of each component of the intake. These analyses include axial pile load capacity, downdrag, lateral pile resistance, axial pile stiffness, lateral earth pressures, unbalanced load analysis (global stability), settlement, seepage cutoff and filter diaphragm design.

Scour protection is needed below the U-frame where it enters the Mississippi River and where the U-frame meets the Transition Channel. The DT anticipates that scour protection will be a sheet pile extending wall from the base of the U-frame to a tip elevation below the expected scour depth that provides adequate embedment. At the time of the 30% design, the scour depth is unknown and therefore the size and tip elevation of the sheet pile cannot be determined.

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 within the excavation limits and tie into the existing levee at each end. The T-Walls will be built above the degraded levee. The T-Walls will be constructed in-the-dry and consist of 10 monoliths; T-1 through T-8 on the north side of the U-frame and T-9 and T-10 to the south. Near the U-frame, the T-Walls will be constructed atop 50± feet of clay backfill. The backfill becomes progressively thinner away from the U-frame and the T-Wall will be constructed above existing clay levee fill at the north and south ends where it ties into the existing levee. Please refer to the 30% Geotechnical Engineering Report for detailed presentation of soil parameters/stratigraphy and geotechnical analyses of the MRL and adjoining T-walls.

4.4 Transition

The Transition is the conveyance section that transitions from the gate structure with an invert at EL -40 to the typical conveyance channel section that has an invert at EL -25. This “transition” has flood protection along both sides of the conveyance being provided by pile supported T-walls. For purposes of this 30% design analysis, the DT considered three representative T-Wall monoliths and developed design parameters for them. These monoliths are T-1, T-10, and T-18. 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 (UBL’s) 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 done assuming an excavation having 1V:7H slopes outside the channel starting 20 feet behind the pile cap for the T-Wall.

Please refer to the 30% 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. We also considered DMM and lightweight fill (expanded shale, also called expanded clay or expanded slate) to eliminate the large UBL’s or SBIM’s. Geofoam or foamed concrete was considered but not recommended due to buoyancy concerns after rewatering. Foamed glass lightweight fill was not considered due to cost and availability. Based on these reasons, the DT recommends DMM for the 30% design. The geotechnical engineering report shows various dimensions of the DMM mixing to satisfy the global stability failure criterion.

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-5a 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. The centerline of the CCL will be offset approximately 150 feet from the edge of the Conveyance Channel. 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. Beyond the tie in of the CCL with the NOV-5a levee system, the CCL is no longer hurricane protection and will be designed as a guide levee. Similar analyses will be performed for the guide levee that will extend from the NOV-5a levee to the outfall transition feature considering a levee crown of EL 12.1. Please refer to the 30% Geotechnical Engineering Report for detailed presentation of soil parameters/stratigraphy and geotechnical analyses of the CCL and the adjacent conveyance channel. 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 30% plan drawings.

4.6 Outfall

The Outfall Transition Feature (OTF) or Outfall Channel is considered the area on the basin side of the existing back levee that transitions the Conveyance Channel to the natural ground within the basin. 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.

The 30% geotechnical engineering report presents analyses of excavated slopes for the OTF. The report also includes a rock jetty design concept that forms the northern and southern edges of the OTF to help guide the discharge further into the basin. The rock would settle rapidly into the foundation due to bearing capacity failures occurring. Therefore, the DT is considering a braced steel sheetpile wall structure with a small rock berm for erosion protection along the mudline that meets the channel-side face of the sheetpile. This design concept will be further investigated in the 60% phase.

4.6.1 Soil Erodibility

During the BOD, the DT identified a few SMEs who have expertise in the field of soil erodibility due to water flows in coastal areas. We ultimately engaged Craig Jones, Ph.D. of Integral Consulting, Inc. because of his expertise in this research area combined with his numerical modeling experience. A soil erodibility sampling and testing program was developed to support the design of the proposed MBSD diversion outfall features. The program will provide both 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. Recent 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 development of a deep scour hole necessitates 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 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 will be prepared by the DT.

4.7 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 with existing Reach 8 parameters resulted in factors of safety below 1.0. Because no project specific borings were taken through the existing back levee alignment, the 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 DT performed preliminary analyses for cost estimate purposes only to raise the back levee level of protection to EL 8.5 and maintain a minimum factor of safety of 1.3. The DT considered several methods to stabilize the back levee system:

- Soil mixing beneath the back levee
- I-wall: degrading the back levee and installing steel sheeting
- Degrading back levee, installing high strength geotextile fabric, and reconstructing the levee

Preliminary analyses indicated a 30-foot wide deep soil mixing block extending to EL -55 would need to be present beneath the crown of the levee to provide the minimum factor of safety considering hurricane loading in Barataria Bay. The DT estimated in lieu of deep soil mixing, a 45-ft sheet pile wall tipped at EL -36.5 feet may be used to stabilize the back levee. For the final alternative, the DT estimated the existing back levee could be degraded to EL 0, a layer of high strength geotextile, approximately 75 feet long, installed, and the levee rebuilt to EL 8.5 could also provide the minimum required factors of safety. After preliminary cost estimates and constructability issues, the DT is proceeding with designing a sheet pile wall with kicker piles for the next phase of the project.

4.8 Secondary Site Features – Reservation Area

The reservation area will be located approximately 500 feet south 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. Detailed settlement analyses will be performed for the next phase of the project. Refer to the 30% Geotechnical Engineering Report for presentation of axial pile load capacity estimates.

4.9 Railroad (R/R) Bridge

The railroad bridge will span the conveyance channel at approximate Station 35+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 12’ x 26’ footings bearing on the levee. Allowable soil bearing values and slope stability analyses will be performed once loadings for the vehicle access road transitions are finalized and the exploration program for the railroad bridge is completed. As shown in the 30% Geotechnical Engineering Report, analyses performed for the railroad bridge include axial pile load capacities and allowable soil bearing calculations.

4.10 Hwy 23 Bridge and T-walls

The Hwy 23 Bridge will be located at approximate Station 65+00. The bridge will span the guide levees and Conveyance Channel. Currently 18 bents spaced on 128-foot centers are envisioned 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 Hwy 23 bridge include axial and lateral pile capacities, downdrag analyses, settlement computations, design of the approach ramps, and pavement recommendations. The DT performed analyses to support the Hwy 23 T-walls that continue the line of protection the guide levees provide beneath the Hwy 23 bridge. As presented in the 30% Geotechnical Engineering Report, allowable axial pile capacities, estimates of settlement induced bending, downdrag, stability analyses, and seepage analyses were performed for the Hwy 23 bridge and T-walls. The highway approach ramps were also analyzed for settlement and stability. Pavement recommendations were also made and included in the geotechnical engineering report.

4.11 Siphon

During the 30% design phase, the inverted Siphon adjacent to Timber Canal was planned to be located at Station 109+50. 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-5a.1 levee (adjacent to the siphon's inlet and outfall features) considering temporary (during construction) conditions and permanent (after construction) conditions. The NOV-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, to compute minimum sheet pile tip elevations to mitigate underseepage, and analyze global stability of the adjoining T-walls. The DT also evaluated anchored and cantilevered sheetpile wall systems to serve as a temporary retaining structure for the siphon excavation.

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 selections of Design Grades are dictated in Section 3, Hydraulic Design, 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**.

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 -40, a top-of-wall elevation of EL 20.35 and an approximate interior width of 194 feet. The base slab will be supported on steel cased piles that are spaced 25 horizontally and 8 feet longitudinally. It will extend from the gate structure, outward approximately 450 feet, where it will skew and transition into training T-walls, designed and oriented to optimize the flow into the structure.

A general cross section is shown in **Figure 5.2-1**, below, along with a general plan and profile of the current preferred alternative. 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.

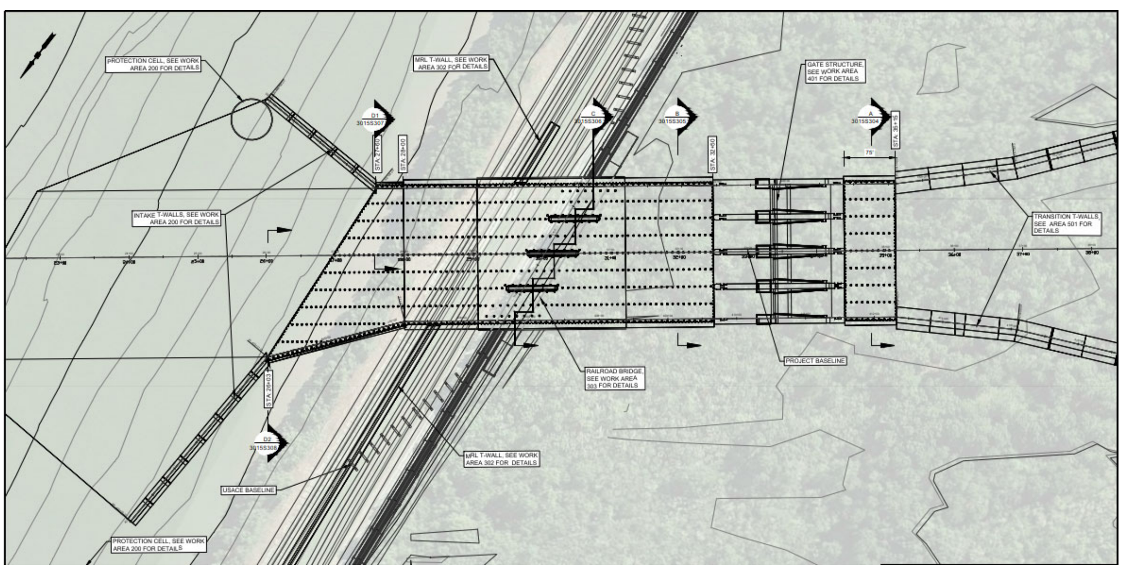


Figure 5.2-1: U-Frame Open Approach

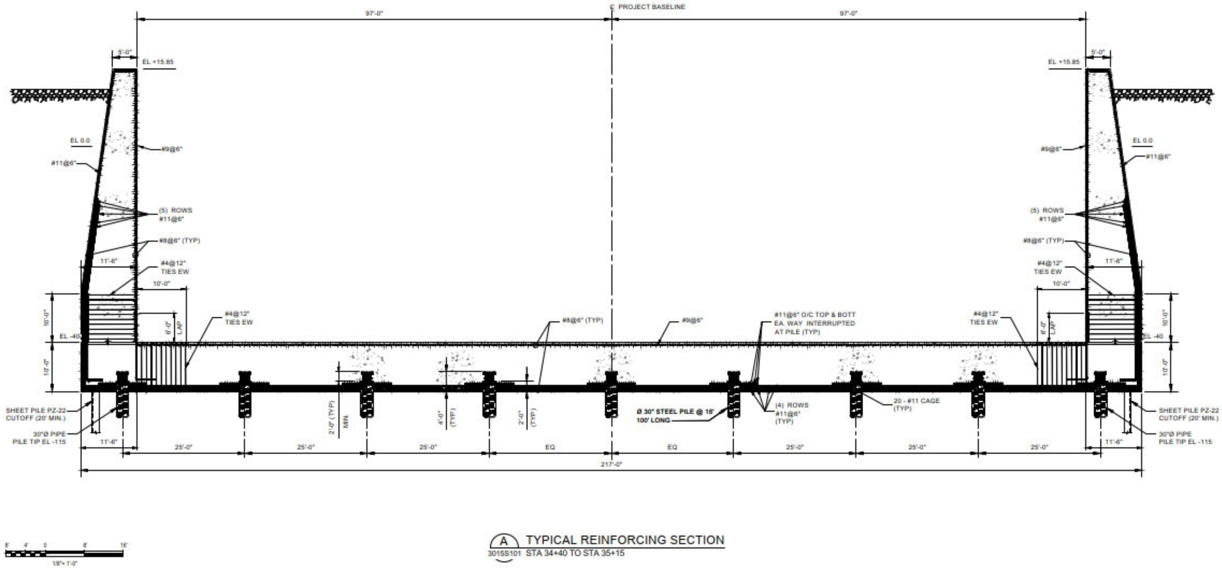


Figure 5.2-2: Typical U-Boat Section

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 ACI 318 with AREMA guidance invoked as applicable for the railroad bridge section. With respect to durability, the intake is designed for a 100-Year Project 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. In general, the thickness and reinforcement of the walls and invert is governed by the maintenance dewatering condition where the water elevation outside of the structure is at EL 5.0 and there is no water inside of the structure. This provides an upward buoyancy force on the bottom of the structure, and an imbalanced soil and water pressure on the walls of the structure. This load case is depicted graphically below. The structure was designed utilizing the strength design method but also checked for serviceability requirements including deflection and crack control.

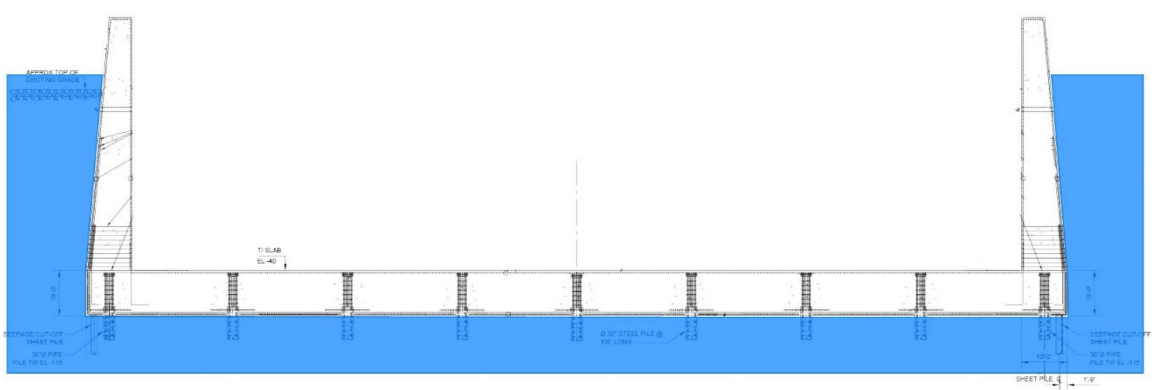


Figure 5.2-3: Maintenance Dewatering Load Case Schematic

The structure was designed utilizing the strength design method but also checked for serviceability requirements including deflection and crack control. It is noted that while this case governs, other load cases provide very similar structural demands – primarily due to the lower load factor utilized for the unusual condition of the maintenance dewatering.

The governing pile loads result from the construction case where the structure is completed inside of a dewatered cofferdam and completely backfilled. In this case, the weight of the entire structure is the non-buoyant weight and the earth pressures are a result of the total unit weight of the soil. It is interesting to note that in the construction dewatering case mentioned above, that the piles do go into tension, 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 dewatering case, but rather only in the service conditions. For the design of the piers, a barge impact load was assumed to be imparted to the piers. For the entire length of the intake, the lateral earth pressures were those from a clay backfill. Near the riverward sections of the intake, unbalanced wave loading was also assumed to act on the walls.

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, for primary models were developed for the intake:

1. Discharge section with top of wall at 15.85
2. Typical intake section with top of wall at 20.35
3. Section at railroad bridge with interior columns/piers.
4. Flared section at Mississippi River

The resulting structural design sections are depicted in the project drawings. It is interesting to note that due to the magnitude of shears and moments at the base of the wall and the sides of the invert, that shear steel is shown in the current design. The alternative is a section with several feet of additional thickness. During future design stages, the design of this can be optimized, investigating options with a haunched slab or varying wall thickness, etc.

The piles were modeled as both pinned-head and fixed-head, with only the outer piles subjected to any significant moment or shear force, primarily due to the moment and shear transfer from the walls. (in general, the resulting base shear on the structure is relatively small due to no significant unbalanced forces on the overall structure). In order to take advantage of the stiffness of the massive concrete structure in distributing the axial loads onto as many piles as possible, axial springs were utilized to represent the piles, thereby distributing the loads to the interior piles as well. While the piles do not all have equivalent loading, some of the vertical loads from the walls are transferred to the interior piles.

The slab thickness and pile spacing in the middle of the typical section were generally governed by the bending moment in between the piles, due to the buoyancy forces in the construction dewatering case. Punching shear and pile embedment were checked.

The design of the railroad bridge section was similar to the typical section, except that the barge impact force and bridge loadings did contribute to the structural demand of the section. The top of the walls has been thickened to provide sufficient space for the bearing seats.

5.2.2.1 Major Design Elements for Future Design Phases

- Storage of removable bridge section
- Sheet piling cutoff
- Location of stop blocks or temporary gates for maintenance dewatering
- Unbalanced slip failure loads from the MRL
- Abrasion-resistant concrete and thermal analysis of pour sequence
- Concrete properties
- Reinforcing Steel properties
- Durability details
- Seismic Analysis
- Site-specific barge impact load development

5.3 Gate Structure

5.3.1 General

The gated structure is part of the Headworks, located between the Intake U-Frame and the Discharge U-Frame. The 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. Also found in the previous BODR was a gate type study, which concluded that a cable-driven tainter gate arrangement is most beneficial for this project.

The Gate Monolith structure consists of four gate bays. It also includes slots for closure bulkheads, a riverside access bridge, and machinery control houses; all component designs are described separately below. The Gate Monolith is pile founded, and DMM is included in the foundation design to resist backfill instability and downdrag effects. Alternative, more economical methods of soil improvements shall be investigated in the next phase of design. 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.

5.3.2 Gate Monolith Design

The Gate Monolith is a 4-bay concrete structure that provides support for tainter gates, control rooms, machinery facilities, maintenance bulkheads, and an access bridge. The structure consists of a 218 feet wide x 190 feet long x 10 feet thick base slab supported by 437 – 30-inch piles, three 8-feet thick interior walls and two exterior walls. The exterior walls taper from 12 feet thick at the base to 8 feet thick at EL 15.85, then maintain 8 feet thickness above this point. Top of the slab (TOS) elevation is EL (-) 40.0 feet, top of the wall (TOW) elevation on the river side is EL 20.35 feet and EL 15.85 feet on the basin side. The middle part of the wall, between the control room and the trunnion of the tainter gate, has a TOW elevation of EL 25.35 feet.

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, 12 load cases are considered and are as follows:

Table 5.3-1: Gate Monolith Design Load Cases

No.	Load Case Name	Description	Factored Load Combinations	Load Category
1- (LC2)	Construction plus Backfill w/, No Uplift	Dead, Lateral Soil One Side, Vertical Soil, No Uplift, Downdrag, Temporary Construction Surcharge on the Slab and the Backfill	1.6(D+EH+EV+EVd+Ls)	Unusual
2- (LC3)	Water at Design SWL or Flowline (impervious)	R/S SWL at EL 14.6, B/S Tailwater at EL 0.0 (2% Hurricane SWL Governs), Gates Closed, Downdrag, Impervious cutoff	2.2(D+EH+EV+EVd+Hs+Hu)	Usual
3- (LC4)	Water at Design SWL or Flowline (pervious)	R/S SWL at EL 14.6, D/S Tailwater at EL 0.0 (2% Hurricane SWL Governs), Gates Closed, Downdrag applied., Pervious cutoff.	2.2(D+EH+EV+EVd+Hs+Hu)	Usual
4- (LC5)	Water at Design SWL plus Wave and Wind, (impervious)	Wave from River Side, Wind, Hydrostatic load R/S at EL 14.6, B/S at EL 0.0, Gates Closed, Impervious sheet pile cut-off	1.6(D+EH+EV+Hs+Hw+Hu+W)	Unusual
5- (LC8)	Water to TOW El 20.35, Riverside Loading	R/S at EL 20.35, B/S at EL 0.0, Gates Closed, Impervious or Pervious	0.9D+0.9EH+0.9EV+1.3Hs+1.3Hu)	Extreme
6- (LC9)	Reverse Head, Basin Side Hurricane SWL, Impervious	B/S SWL at EL 9.1, R/S Tailwater at EL 0.0 (2% Hurricane SWL Governs), Gates Closed, Downdrag applied, Impervious cutoff.	2.2(D+EH+EV+EVd+Hs+Hu)	Usual
7- (LC11)	Reverse Head, Basin Side SWL, plus Wave and Wind, Impervious	Wave from Basin Side, B/S SWL at EL 9.1, R/S Tailwater at EL 0.0 (2% Hurricane SWL Governs), Gates Closed, Impervious cutoff.	1.6(D+EH+EV+Hs+Hu+Hw+W)	Unusual
8- (LC18)	River at 1,000,000 cfs , 75,000 cfs Conveyance Operation (Pervious) All gates Opened	R/S at EL 6.9, D/S at EL 6.9 Pervious sheet pile cut-off, Current Load	2.2(D+EH+EV+Hs+Hd+Hu)	Usual
9- (LC22)	River at 1,250,000 cfs, All gates Opened (Pervious)	Dead load R/S at EL 12.0, B/S at EL 12.0, Pervious sheet pile cut-off Wind Load, Current Load	1.6(D+EH+EV+Hs+Hu+Hd+W)	Unusual

No.	Load Case Name	Description	Factored Load Combinations	Load Category
10-(LC25-a)	Maintenance Dewatering, (impervious or pervious cutoff)	Dead load Impervious or pervious sheet pile cut-off R/S at EL 8.0, B/S at EL 2.0	1.6(D+EH+EV+Hs+Hu)	Unusual
11-(LC25-b)	Maintenance Dewatering, (impervious or pervious cutoff)	Dead load Impervious or pervious sheet pile cut-off R/S at El 8.0, B/S at El 2.0 (Design intermediate piers w/ only one side dewatered)	1.6(D+EH+EV+Hs+Hu)	Unusual
12-(LC25-c)	Maintenance Dewatering, (impervious or pervious cutoff)	Dead load Impervious or pervious sheet pile cut-off R/S at EL -40, B/S at El 2.0	0.9D+1.35EH+1.35EV+1.3Hs+1.3Hu	Extreme

Still Water Level (SWL) condition is considered a usual case and SWL plus wind/wave is an unusual load case. Water to the TOW and dewatering employing the tainter gate with basin side water elevation at EL 2.0 are considered extreme load cases. Backfill is assumed to be clay with dry unit weight of 115 pcf and at rest earth pressure coefficient K_0 of 0.75. The construction case includes a height differential in the backfill of 5 ft and a down drag force is applied for 4 load cases. 50 psf wind pressure is assumed to be applied on walls, gates in open position and machinery rooms. The wave load is assumed to be equal to the intake wave loads, which are calculated based on FEMA55-Vol.II for river at 1,000,000 and 1,250,000 CFS flows with average velocities presented in the hydrodynamic report; velocity is assumed to be distributed uniformly throughout the entire depth of the piers and the drag coefficient is taken to be 1.5. The machinery room loads are calculated based on IBC-2015. Applied tainter gate weights, bulkhead weights and the bridge loads are based on their respective designs.

b. Analysis and Design Summaries

A finite element model is developed using SAP2000 to analyze stresses, displacements and piles reactions. Hydrostatic and hydrodynamic loads from current and waves are applied to the walls, gates and bulkheads. Uplift is applied to the slab. Wind load is applied to the walls, gates and machinery rooms. Soil lateral forces and down drag are applied to the exterior walls. Bridge, machinery room and bulkhead loads are transferred to the walls.

To model the piles in SAP, a pile stiffness matrix for pipe piles (assumed to be pinned at the slab) is derived by the USACE program CPGA based on the geotechnical report and the provided pile capacities. This stiffness matrix generates vertical and lateral springs that represent pile behavior under loading. Walls and slabs are modeled using area shell elements with corresponding thicknesses and properties. The model consists of 27,144 joints and 26,315 area elements with a structured mesh size of 2 x 2 ft.

As part of the analysis the compression, tension and displacement of the piles are evaluated and checked against the allowable pile capacities, overstresses and displacements to confirm that the pile type and layout are satisfactory. Also, wall and slab thicknesses are defined based on the shear and moment capacity of the sufficiently reinforced sections and verified with the model results to confirm that the structure can carry the lateral and vertical loads effectively.

5.3.3 Tainter Gate Design

A steel tainter gate with top elevation of EL 20.35, sill elevation of EL (-) 40.0, and 80-foot radius is designed to a 30% level. Beyond regulating flow into the diversion channel, this gate can act as a dewatering instrument for areas riverside of the gate. The analysis uses the LRFD design procedure described in ETL 1110-2-584, *Design of Hydraulic Steel Structures*, including Appendix D – Spillway Tainter Gates. EM 1110-2-2702, *Design of Spillway Tainter Gates*, is also used as reference where the ETL has limited design information.

a. Design Criteria and Loading Conditions

Five load cases are examined for the 30% design. These are chosen from the larger list of Load Combinations created for the Intake Monolith because engineering judgement suggests they will provide an accurate representation of various extreme conditions the gates will experience. The design cases are as follows:

Table 5.3-2: Tainter Gate Design Load Cases

Load Case Name	Description	Factored Load Combinations (per Table D-1, ETL 1110-2-584)	Limit State
Water to TOW EL 20.35, Riverside Loading	River EL 20.35 Basin EL 0.0 Gate in Closed Position	1.2D + 1.4Hs	Extreme I Case 4.a
Reverse Head, Water to TOW EL 15.85	River EL 2.0 Basin EL 15.85 Gate in Closed Position	1.2D + 1.4Hs	Extreme I Case 4.a
Emergency Dewatering	River side empty (EL -40.0) Basin EL 2.0 Gate in Closed Position	1.2D + 1.4Hs	Extreme I Case 4.a
River at 1,250,000 cfs, All Gates Open	Gate Fully Opened (bottom at EL 14.0) +/- 20psf wind on full gate	1.4D* 1.2D + 1.3W*	Extreme I Case 6
Operation with Extreme Head Differential	River EL 16.4 Basin EL 0.0 Gate Beginning to Open on Two Hoists	1.2D + 1.4Hs + 1.4Fs + 1.0Ft	Usual Operation Case 2.a

* ASCE 7 used as a guide for load factors in the Gate Open Cases. ETL table is not specific regarding Dead Load factors for this condition.

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.

b. Analysis and Design Summaries

A 2-D analytical procedure is followed for the skin plate and rib sizing. The skin plate is conservatively assumed to act as a simple beam spanning between two ribs (ETL 584 allows use of fixed-end moments). The ribs are analyzed as simple beams spanning between horizontal girders assuming the skin and ST rib section act compositely; guidance from the ETL suggesting a minimum rib depth of 8 inches is adhered to.

Horizontal girders, vertical and diagonal girder bracing, end frames, and end frame bracing are sized using a SAP2000 3-D model (see **Figure 5.3-1**). The skin plate assembly is included as a shell element with modifiers applied to represent the added stiffness and weight provided by ribs. All frames are assigned rolled shapes. Water and wind loads are applied to the front and/or back faces of the skin plate. Dummy frame elements with no stiffness or weight are used to impart linear forces such as side seal friction and lifting cable pressure on the skin plate. The load cases described in **Table 5.3-2** are input in SAP2000 with applicable load factors and SAP's steel design process checked all members using the AISC360-05/IBC 2006 code and the USACE's ϕ*α*nominal resistance limit.

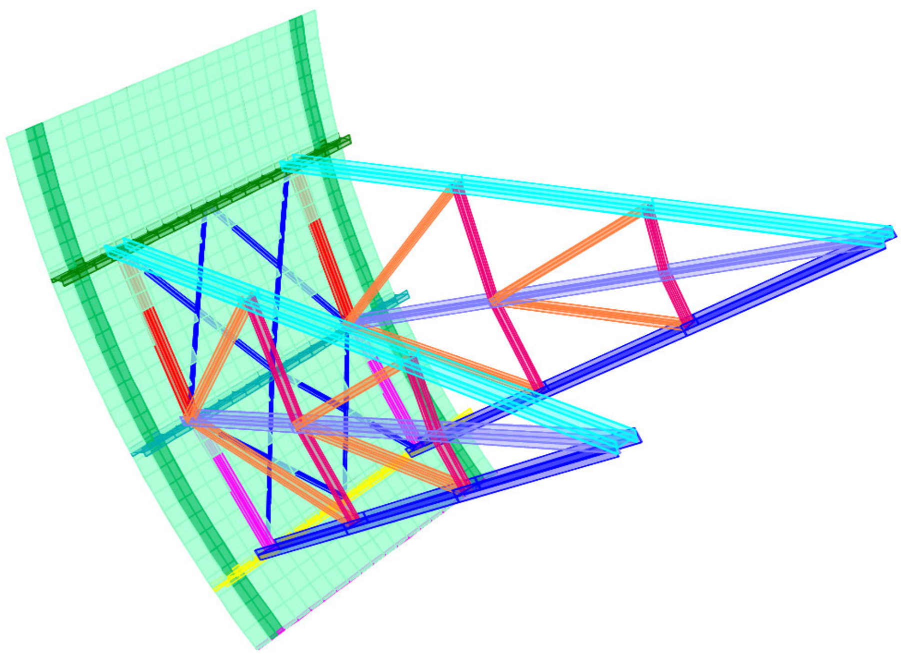


Figure 5.3-3: 3-D SAP2000 Model of Tainter Gate

Boundary conditions for the 3-D model are based on the recommendations of EM 1110-2-2702, Section 3-5.a.(3)(a): the trunnion is modeled as a pin free to rotate with no translation; for gate-closed cases, the nodes along the bottom sill are assigned supports that only restrict movement in the vertical direction; for gate-operating cases, a spring that allows only vertical compression reactions is used at the wire rope attachments, which allows the gate to lift upward when hoist forces are large enough but does not allow downward movement.

Fatigue of gate components due to cyclic loading and operation is not being directly analyzed in this phase of the design. To account for the overall strength reductions related to fatigue, the design team has decided to limit all steel member strength ratios for factored load cases (combinations of axial, major and minor bending calculated by the SAP program) to 0.7. Typically, any ratio less than 1.0 is acceptable for design.

5.3.4 Bulkhead Design

Bulkhead gates are steel truss structures with a skin plate on the river side that are designed to temporarily close an intake bay (See **Figure 5.3-2)**. Bulkheads will be used to dewater gatebays when maintenance is required for the tainter gates or in an emergency situation where the tainter gates are damaged and no longer capable of holding water. Six 10 ft tall bulkheads stacked atop one another are needed to completely close one bay; the complete 60 ft tall stack is called an “End Dam”. The Design Team is currently recommending that two sets of bulkheads (or 4 complete End Dams) be fabricated for the project, allowing 2 bays to be refurbished per 5-month non-operational low water season. The two sets are interchangeable and may be used at any of the three required locations: upstream of the Tainter Gates, downstream of the Tainter Gates, or at the RR bridge piers.

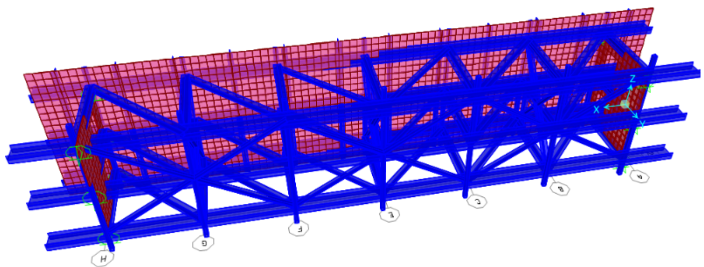


Figure 5.3-2: 3D View of Bulkheads

Two types of bulkheads are designed: the Lower Bulkhead is designed to withstand up to 60 ft of hydrostatic water pressure; the Upper Bulkhead is designed to carry up to 30 ft water pressure. **Figure 5.3-3** shows the configuration of bulkhead placement in an emergency closure. The difference in weight is approximately 5 tons. A third type of bulkhead fabricated from W-sections may be designed in future submissions for the top of the stack, where the hydrostatic load is minimal.

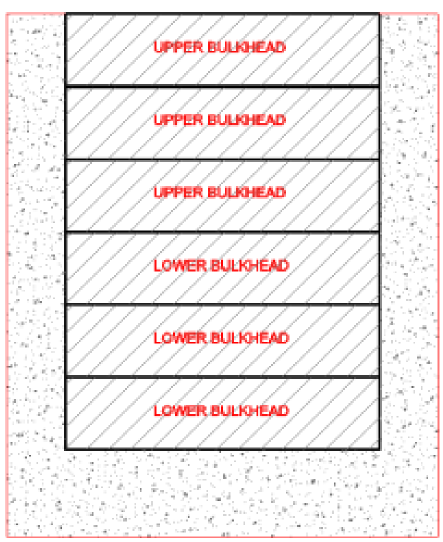


Figure 5.3-3: Bulkhead Configuration

Bulkheads will be stored on top of the gated structure and on a storage pad on grade near the structure; the configuration of storage will be described in future submissions.

a. Design Criteria and Load Conditions

Design of the bulkhead and wheel assemblies shall be done in accordance with ETL 1110-2-584, *Design of Hydraulic Steel Structures* (30 Jun 2014). All steel will be Grade 50.

The bulkheads are designed for both maintenance dewatering and emergency closure. The differential head of the full maintenance dewatering in 50 ft of water far exceeds the emergency closure differential of 12 feet and therefore controls the bulkhead design. Beyond static head differential, the emergency bulkheads are also designed for installation in flowing water. Emergency conditions that would warrant the placement of bulkheads in flowing water would be a break in the basin side levee or a tainter gate jamming in the open position. Bulkheads are also designed for lifting and installation cases. Each bulkhead will be lowered into place using lifting bars attached at either the end truss panel or the end diaphragm; the use of a spreader bar shall be determined after the gate weight is established.

Both the Upper and Lower type Bulkheads use trusses as horizontal beams behind a steel skin plate. Trusses are comprised of W-shape chords with double L-shape webs and braces. Vertical WT-shape intercostals provide additional stiffness to the skin plate between the horizontal trusses. The truss bar eliminates the horizontal thrust applied to the upper horizontal frame. To be conservative in design, the composite action of skin plate with vertical intercostals members is not considered. The skin plate thickness for the lower and upper bulkheads are assumed 1/2" and 3/8", respectively.

Table 5.3-3: Bulkheads Design Load Cases

Load Case Name	Description	Factored Load Combinations (per Table D-1, ETL 1110-2-584)	Limit State
Maintenance Dewatering	Inside dry at EL -40, wet side at EL 12.0 (50 yr. SWL plus 2 'Freeboard + 2ft. SLR.) + Impact	1.2D + 1.4Hs + I	Extreme I
Emergency Closure	U/S at 50 Yr. Hurricane (future) plus 2ft. Freeboard, EL 20.0; D/S at EL 0.0 + Impact	1.2D + 1.4Hs + I	Extreme I

Main horizontal chords are designed as continuous members. Horizontal and vertical struts are modeled as pinned connections. Because of the nature of their use, no fatigue analysis/detailing is required. When in use the bulkheads will not be exposed to significant cyclical loading, and installation loads are infrequent, short term, and provide no stress reversals. Girders above EL 3.0 will be designed for an impact load of 125kips at the girder center, which simulate possible impact from a work barge moored at the bulkhead. When this impact condition is applied, a lower load factor is permitted. The impact load is assumed applied to the horizontal chord.

b. Analysis and Design Summaries

The axial load for all bracing members is calculated for the most critical load combination. For the main girders the flexural strength and shear capacity are checked. Modeling, analysis and design are done using SAP2000 software following the USACE manual for Hydraulic Steel Structures (HSS) and the AISC Steel Construction Manual, 14th edition. Since bulkheads will not be used extensively, the stress ratio for the element design is allowed to reach 0.9.

5.3.5 Access Bridge Design

A maintenance bridge on top of Gated Structure with 30 ft clear width is essential for access to the tainter gates and bulkheads slots. A mobile crane will be used to lift stored bulkheads on the gated structure and place them into slots when either maintenance is required for the tainter gates or for emergency closure. According to dimensions and specifications of existing mobile cranes, at least 30 ft is needed to fully extend outriggers. The bridge's width reduces to 24 feet for where the mobile crane will not be needed. The 24 ft bridge section is a two-lane road with concrete crash barriers on either edge; no side walk-way is considered for this bridge. The bridge will be comprised of a cast-in-place reinforced concrete slab and precast pre-stressed concrete I-beams supported on the Gated Structure and a series of pile bents.

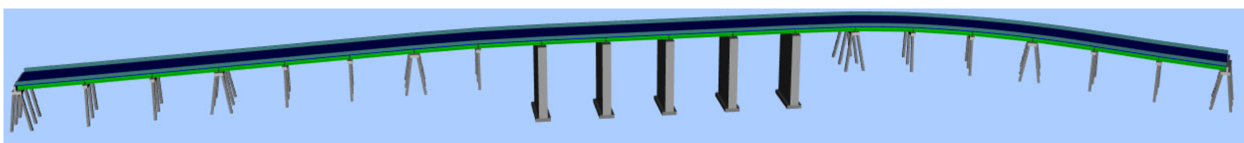


Figure 3.5-4: 3D view of Maintenance Bridge

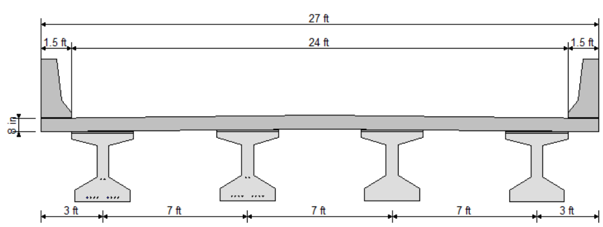


Figure 3.5-5: Section with 27 ft. width

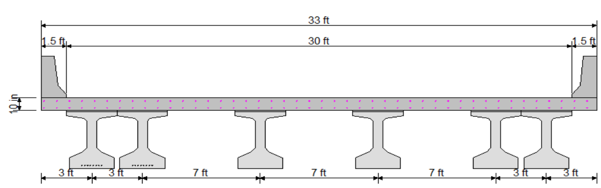


Figure 3.5-6: Section with 33 ft. width (on the Gated Structure)

a. Design Criteria and Load Conditions

AASHTO's LRFD vehicle loads and load combinations are used for design of the bridge. The standard design vehicles provided by the AASHTO LRFD specification such as design truck, design tandem, lane load and moving load are used to design of superstructure. In addition to standard moving loads, a mobile crane model LTM1350-6.1 is considered as a moving load on this bridge. ***Table 5.3-4*** describes the various AASHTO load combinations used in the bridge design.

Table 5.3-4: Maintenance Bridge Design Load Combinations and Load Factors

										Use One of These at a Time				
Load Combination Limit State	**DC DD DW EH EV ES EL PS CR SH**	**LL IM CE BR PL LS**	**WA**	**WS**	**WL**	**FR**	**TU**	**TG**	**SE**	EQ	IC	CT	CV	SC[1]
Strength-I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}		-	-	-	-
Strength-IV	γ_p	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-	-
Strength-V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Extreme Event-I	1.00	**0.25**[2]	1.00	-	-	1.00		-	-	1.00	-	-	-	
Extreme Event-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00	-
Extreme Event-III[1]	$\boldsymbol{\gamma_p}$	**1.75**	**1.00**	**0.30**	-	**1.00**	-	$\boldsymbol{\gamma_{TG}}$	$\boldsymbol{\gamma_{SE}}$	-	-	-	-	**1.00**
Extreme Event-IV[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	**1.40**	-	**1.00**	-	$\boldsymbol{\gamma_{TG}}$	$\boldsymbol{\gamma_{SE}}$	-	-	-	-	**0.70**
Extreme Event-V[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	-	-	**1.00**	-	-	-	-	**1.00**	**1.00**	**1.00**	**0.60**
Extreme Event-VI[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	-	-	**1.00**	-	-	-	**1.00**	-	-	-	**0.25**
Service-I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-		-
Service-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-		-
Service-III	1.00	**1.00**[3]	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-		-
Service-IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.00	-	-	-		-
Fatigue- I LL, IM & CE only	-	1.50	-	-	-	-	-	-	-	-	-	-	-	-
Fatigue- II LL, IM & CE only	-	0.75	-	-	-	-	-	-	-	-	-	-	-	-

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 5.3-5***.

Table 5.3-5: Access Bridge Design Load Cases

AASHTO Designation	Limit State Objective	Loads
Service I	Limit compressive stress in girder and deck to maintain adequate factor of safety against concrete crushing	Full value of service (un-factored) dead and live loads
Service III	Limit tensile stress in girder to maintain factor of safety against concrete tension cracking	Full service dead load, but reduced service live load
Strength I	Provide adequate resistance to girder "breaking" failure	Factored live and dead loads
Fatigue	Limit stresses caused by repetitive vehicle live load	Loads produced by "fatigue truck"

a. Analysis and Design Summaries

Precast pre-stressed concrete girders acting as simply supported beams will carry loads from the superstructure to the substructure. Since an operating crane on the deck applies concentrated loads through outriggers, a 10-inch thick reinforced concrete slab is assumed for the bridge on the gated structure and an 8-inch thickness is assumed for the remainder. Although the reactions of outriggers will be distributed on the bridge deck, conservatively four concentrated loads distributed by a 4ft x 4ft pad are considered for the design and 75% of the total load (lifting load, crane weight, counterweight, accessories weight, etc.) is assumed to be carried by one of the outriggers in a critical scenario.

Driven 24in square precast piles transfer superstructure load to the soil. The piles are connected to the pile bent directly and extended into soil to approximately EL. -110.0. To strengthen the bridge against lateral loads, every third pier has piles battered in the longitudinal direction of the bridge. The two approaches have 5% slopes to the top of the gated structure; the bridge is horizontal across the structure. Center to center spacing of piers is 50 ft, which makes the clear span of the girders 46 ft. TX46 type girders are used for the superstructure on the gated structure and TX40 are used elsewhere. All bent caps are 2 ft - 6 in deep.

A 3D model of entire bridge is developed using the LEAP CONCRETE Software. The size and number of prestressed girders as well as reinforcing strands are determined by the software. A hand calculation is performed to design exterior girders on the portion of the bridge located on top of the gated structure where the mobile crane will operate. The sections are checked for shear capacity and results in the use of 16 strands with 0.6 in diameter for the girders. The concrete slab is check for shear and flexural stress to ensure distribution of the loads from the deck to the girders; #5 rebar with spacing of 9 inches is obtained.

5.3.6 Storage Rack and Crane Pad Design

The Design Team has previously recommended that two storage racks be constructed at the site. There are a total of twenty-four (24) bulkheads for the gated structure; eight (8) will be kept on the Gated Structure and sixteen (16) will be evenly divided between the two storage racks. The structural system of each storage rack is a one-way slab supported by 6 ribs which are held up by 18 piles. The storage rack slab is designed to be 80 feet wide x 50 feet long with a thickness of 8 inch. The ribs are 3 feet wide x 50 feet long with a thickness of 2 feet – 8 inches. The rack is founded on 24-inch square prestressed concrete piles spaced at 14 ft on center in the short direction and 20 ft in the long direction.

The crane pad is used for bearing the weight of the crane while in use. There are two 60 feet wide x 40 feet long x 10-inch thick crane pad slabs proposed for the site. The TOS elevation for the storage racks and crane pads is EL 6.0 feet. The dimensions for both slab designs are subject to change for the next submittal. In addition, when the crane type is selected for the crane pad, a detailed foundation and slab design we be provided.

a. Design Criteria and Load Conditions

The design of the storage rack and the crane pad follow ACI 318-14 and EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures. Two load cases are applied in this analysis and are as follows:

Table 5.3-6: Load Conditions

LC No.	Description (with factor)	For the design of the slab, we considered the controlling case of LC2 (Dead and Live Load combo).
1	Dead (1.4D)	
2	Dead & Live (1.2D + 1.6L)	

b. Analysis and Design Summaries

The weight of the bulkheads for one slab is approximately 800 kips (8 bulkheads weighing 100 kips each) which is applied as a live load. The weight of timber mats, which allow for more distribution of the bulkhead's load on the slab, are applied as a live load. In addition to the timber mats, a personnel load is applied uniformly over the entire slab as a live load.

The one-way slab is designed in SAP 2000 finite element software to extract ultimate moment and shear outputs. **Figure 3.5-5** shows how the live loads are distributed in the SAP 2000 model. The pink area represents the distributed load of personnel, the red area represents the distributed load for the timber, and two of the blue lines represent the distributed load for a single bulkhead. The bulkheads are 6 ft wide but will rest on two supports, so the design is distributed on two strips of 1 ft. The dead load is automatically applied in SAP 2000 as it considers the dimensions of the slab and type of concrete (f’c = 4000 psi). **Figure 3.5-6** shows the deformed shape of the factored live load case (the controlling load case for this design).

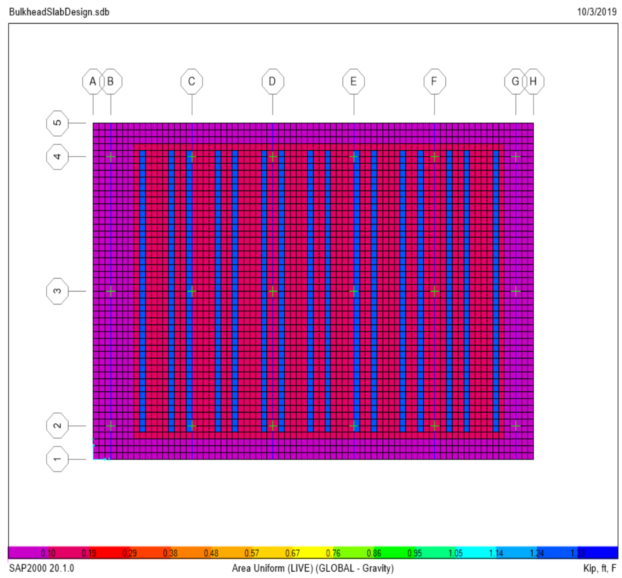


Figure 3.5-5: Distribution of Applied Live Loads in SAP 2000

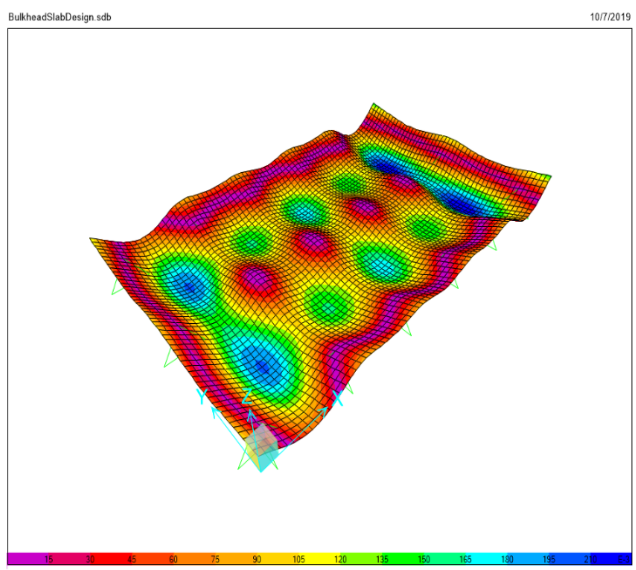


Figure 3.5-6: Deformed Shape of Factored Live Load in SAP 2000

The positive and negative ultimate moments (M11) and ultimate shear force are generated by the SAP model for the factored load cases to design the 8" slab. With the ultimate moments and shear, the area of reinforcement can be selected. The maximum shear force and ultimate moment (factored load case 2) values from the applied frame in the SAP model are used to determine the area of reinforcement for the design of the beam. The unfactored joint reactions from SAP are used to design the pile foundation. The joint reactions are multiplied by a safety factor of 3 because the soil has not been tested; that load is then used to determine the length of the pile. For this design, the highest joint reaction with the safety factor is equal to 323 tons which requires a pile length of 121.5 ft.

5.3.7 Control House and Ancillary Features

The control house is situated on the second story above the piers. The operating floor elevation of EL 40.85 is set based on the height needed for the machinery cable drum to raise the gate to the fully open position (bottom of gate aligns with trunnion pin at El 14.0). There are five Control Houses, all sized to accommodate the machinery and electrical control panels. Exterior stair cases permit more work space. Gate controls shall be housed on Piers 2 and 4.

The Control Houses shall be either reinforced masonry or precast concrete buildings except for the bottom stories, which shall be reinforced concrete. The Control Houses and first floor storage room are designed to withstand a 140 mph wind force in accordance with ASCE 7-10. A roof panel shall be removable to allow for the removal and repair of the gate machinery. Five foot wide walkways that double as supports for the gate hoist torque tubes connect the Control Houses; current walkway design is a precast concrete slab supported on corbels.

5.4 Mississippi River Levee (MRL) Tie-Ins

5.4.1 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 ten (10) MRL T-walls, eight (8) (M-1 thru M-8) T-Walls located on the north and two (2) (M-9 and M-10) located on the south side of the U-Frame. The T-Walls on the North side of the U-Frame extend approximately 448 feet and the South side T-Walls extend approximately 96 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. DMM is proposed beneath the base slab to eliminate any unbalanced loads and settle induced bending on both North and South side T-Walls. The DMM panels are 3 feet in diameter by 70 feet wide at 7'-6" O.C and to a depth at EL -90 feet.

5.4.2 Design Features

5.4.2.1 Base Slab and Stem

All MRL monoliths (M-1 thru M-10) have a TOW EL 20.35, TOS EL 12.0, 3-foot thick base slab and 3'-4" thick stem wall. North side T-Walls (M-1 thru M-8) have a 19'-2" wide base slab to support a proposed emergency maintenance roadway to railway track on the land side. Top of roadway elevation is at 16.85 ft. Since the roadway alignment to rail track is TBD, for this 30% design phase the base slab is assumed to be the same throughout all north side T-Walls with HS-20 truck loading. The South Side T-Walls have a base slab width of 15 feet and do not support a maintenance road. A continuous cut-off sheet pile curtain wall is embedded 9 inches into the base slabs. All monoliths (M-1 thru M-10) 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 30% design phase. See **Appendix D** for pile layout, tip elevations, sizes and other design features. North side T-walls, all piles are plumb piles. Batter piles are battered at 1:12 slope on south side.

5.4.2.2 Cut-off Wall Sheet Pile

The cut-off wall of sheet piling is provided to limit seepage to a tip elevation at -65.0 feet, 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.

5.4.3 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 Rev. 9, dated April 22, 2019.

Table 5.4-1: MRL T-Wall Design Load Case Summary

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
2	Water at Design SWL or Flowline (impervious)	14.85	0.0	2.2(D+EH+EV+Hs+Hu)
4	Water at Design SWL plus Wave, (impervious)	14.85	0.0	1.6(D+EH+EV+Hs+Hw+Hu)
6	Water at Design SWL plus Wind (Impervious or pervious) plus 100 yr Barge Impact (200 kips)	14.85	0.0	1.6(D+EH+EV+Hs+Hu+W+Im)
8	Water to TOW, River side Loading	20.35	0.0	1.6 (D+EH+EV+Hs+Hu)
9	Water at Design SWL (impervious), Truck Load (HS-20) on P/S Base	14.85	0.0	1.6(D+EH+EV+Hs+Hu+V)

Notes: 1) No Unbalanced loads.
3) D= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, V= Truck Load and W= Wind

5.4.4 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, 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-2-1100-2104 (Design of Concrete Hydraulic Structures). Moment calculations for the steam and base slab will be performed in the next design phase. 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. The North side base slabs have an additional load due to the HS-20 Truck Load. Factored concrete design loads shown in Table 5.4-1 are used to confirm the adequacy of the stem wall and slab thickness.

The MRL T-Wall's (M-1 thru M-10) stem is 3'-4" thick and 8'-4" tall. M-1 thru M-10 base slabs have a top slab elevation of 12.0 and are 3 feet thick. The north side T-Walls (M-1 thru M-8) have 14-inch plumb H-piles spaced at 8 feet o.c. which have a tip EL -55.0 feet. The south side T-Walls (M-9 and M-10) have 14-inch plumb and battered H-piles spaced at 8 feet o.c. with tip elevations of -43.0 feet and -71.0 feet, respectively. North and South side T-Wall piles are embedded into the base slab 14 inches (one pile diameter) to create a fixed connection to reduce the deflection. See **Appendix D** for pile layout, tip elevations, sizes and other design features.

5.5 Transition Structure

5.5.1 General Description

The Transition T-Wall monoliths are located on both sides of the Conveyance Channel starting from the Gated Diversion Structure and spans from the U-Frame to the guide levee tie-ins. There is a total of forty-two (42) Transition T-Wall Monoliths, twenty-one (21) identical T-Wall Monoliths on North and South side of Conveyance Channel. All Transition T-Walls have a TOW EL 15.85. The top of base slab elevation decreases as the T-Walls approach the guide levee tie-ins. The top of base slab at the U-Frame is EL -40 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-9 is 32 feet and 7 feet, respectively. Monoliths W-10 thru W-16 base slab width and thickness is 24 feet and 5 feet, respectively. Monoliths W-17 thru W-21 base slab width and thickness is 15 feet and 3'-6", respectively. The stem walls for monoliths W-1 thru W-16 are 2'-6" thick at the top and thicken at a 1H:12V slope toward the land side. Monoliths W-17 thru T-21 have uniform walls with a thickness of 2'-6". The Transition T-Walls W-1 thru W-9 are support by 24-inch diameter by ½-inch pipe piles. Monoliths W-10 thru W-16 are support by 24 inches diameter by 3/8-inch pipe piles and monoliths W-17 thru W-21 are supported by 18-inch diameter by 3/8-inch pipe piles. Six-inch thick lean concrete (2500 Psi) mud mat is placed below base slab as a level platform for base slab construction.

All Monoliths contain a continuous PZ-22 sheet pile 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 and land side of the T-Walls is backfilled with sand to EL 4.0. An 8-foot clear roadway is also proposed on top of the T-Wall to provide small 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.

5.5.2 Design Features

5.5.2.1 Base Slab and Stem

The base slab width and thickness for monoliths W-1 thru W-9 is 32 feet and 7 feet, respectively. Monoliths W-10 thru W-16 base slab width and thickness is 24 feet and 5 feet, respectively. Monoliths W-17 thru W-21 base slab width and thickness is 15 feet and 3’-6”, respectively. The stem walls for monoliths W-1 thru W-16 are 2’-6” thick at the top and thicken at a 1H:12V slope toward the land side. Monoliths W-17 thru T-21 have uniform walls with a thickness of 2’-6”.

5.5.2.2 Cut-off Wall Sheet Pile

The cut-off wall of 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-21/W-42 monoliths at the guide levee tie-in. The top of the sheet pile at these locations is set to match with the guide levee tie-in crown elevation.

5.5.3 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. Engineering judgment was used in selecting the load cases by comparing the magnitude of the applied loads and the allowable overstress. Only the selected basic load cases are evaluated. All other load cases and combinations will be evaluated in a later phase of the project. As previously discussed, 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, Rev. 9, dated 04/22/2019.

Table 5.5-1: Transition T-Wall Design Load Case Summary

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
2	Construction with Backfill	N/A	N/A	1.6(D+EH+EV+EVd+Ls)
3	Water at Design SWL (impervious)	9.35	0.0	2.2(D+EH+EV+Hs+Hu)
8	Water to TOW Channel side Loading (impervious or pervious)	15.85	0.0	1.3(D+EH+EV+Hs+Hu)
9	Reverse Head Retaining Wall (impervious)	0.0	0.0	2.2(D+EH+EV+Hs+Hu)
14	Reverse Head, NOV5a Overtopped, (impervious or pervious)	0.0	8.4	1.6 (D+ EH+ EV+ Hs+ Hu)
17	Maintenance Dewatering (impervious or pervious)	TOS EL	0.0	1.3(D+EH+EV+Hs+Hu)

Notes: 1) No Unbalanced loads.
3) D= Dead Load, EVd= Down Drag, EH= Lateral Earth, EV= Vertical Earth, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

5.5.4 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**, considering 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 method beneath the base slabs of tall T-Wall monoliths (W-1 to W-16 and W-22 to W-37). See section 9 of geo-tech report for details on DMM used for eliminating unbalanced loads and settlement induced bending moments on piles.

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. Only three GROUP models have been performed for the 30% design phase (W-1 thru W-5/W-22 thru W-26, W-10 & W-11/W-31 & W-32 and W-21/W-42), all T-Walls will be modelled in GROUP in the next design phase. 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. W-1 thru W-5 results are used to prorate the pile tip elevations based on TOS elevations for W-6 thru W-9/W-27 thru W-30. W-10 and W-11 GROUP results are used to prorate pile tip elevations for W-12 thru W-16/W-33 thru W-37. W-21 results are used to determine the pile tip elevations for W-17 thru W-21/W-38 thru W-42. The calculation of the pile capacities is performed by using the pile capacity curves for 24-inch diameter and 18-inch diameter, provided by AECOM. 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-2-1100-2104 (Design of Concrete Hydraulic Structures). Moment calculations for the steam and base slab will be performed in the next design phase. The stem of the T-Walls is designed using the pressure calculations of the Construction w/ Backfill and TOW load case. The base slab is designed creating a STAAD model to analyze pile reactions of the governing load case from GROUP. Maximum shear is obtained from STAAD and compared to the allowable strength of the concrete. Factored concrete design loads shown in Table 5-5 of design criteria are used to confirm the adequacy of the stem wall and slab thickness.

5.6 Outfall Transition Feature (OTF)

Structural components of the OTF will include a sheet-pile toe wall to mitigate potential scour at the end of the outfall. There will also be braced sheet pile walls at the ends of the OTF flares into the basin. The design of these components will be further developed in the 60% Design Phase.

5.7 Siphon

5.7.1 General Description

The Inverted Siphon consists of the Intake Structure at the north diversion channel levee, the Inverted Siphon piping beneath the diversion channel, and the Outlet Structure at the south diversion channel levee. The Intake and Outlet Structures are pile supported, reinforced concrete, subdivided rectangular U-frame channels with partition walls subdividing the structures at each Inverted Siphon pipe.

The Intake and Outlet Structures each feature steel bar screens and a 20-foot wide access deck across the width of the structure to facilitate maintenance. Both structures feature pile supported wing walls and sluice gates for each Inverted Siphon pipe.

Steel H-piles will be utilized for the for the Intake and Outlet Structure foundations. The Intake and Outlet Structures have been designed as U-Frame channels in accordance with EM 1110-2-2007, *Structural Design of Concrete Lined Channels,* EM 1110-2-2004, *Strength Design for Reinforced-Concrete Hydraulic Structures* and ACI 318-14, *Building Code Requirements for Structural Concrete.*

The Inverted Siphon piping will consist of six - 96" diameter fiberglass reinforced pipes and will be designed in accordance with EM 1110-2-2902, *Conduits, Culverts, and Pipes.*

The siphon piping will be fully pile supported between the Intake and Outlet Structures and the top of channel bank to prevent issues arising from differential settlement between the Intake/Outlet Structure and the siphon piping.

From the top of bank to the toe of the channel bank, the siphon piping will be supported on piles with lengths tapering linearly down to 10' at the final siphon piping bent providing a smooth transition of siphon piping support from fully pile supported at the top of bank to soil supported below the bottom of the channel.

The siphon piping section along the bottom of the diversion channel will be tied down to a 12-inch tremie slab and backfilled with flowable fill allowing for the pipes to be place 2'-0" apart to minimize excavation and backfill. This will provide a factor of safety against uplift of greater than 1.2 during construction 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 the Inverted Siphon Headworks Structure. Tension connectors will be required on all piling for the Intake and Outlet Structures to counteract buoyancy in the channel-dry maintenance condition.

5.7.2 Design Criteria

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

1. The siphon piping is submerged by 1 foot at drainage canal's low water elevation.
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).
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.
5. Personnel and vehicular access are provided for cleaning the bar screens and operations and maintenance.

6. Portable sluice gate actuators will be used when the gates need to be operated during maintenance or storm conditions.

7. Operator safety and facility security are maintained.

5.7.3 Excavation

The method of excavation for construction of the Inverted Siphon will utilize a combination of sloped and TRS excavation. The upper portion of the excavation from -4 to -25 being a simple sloped excavation with a vertical sided excavation for the lower portion utilizing a temporary retaining structure (TRS) to minimize the overall amount of excavation. 4H:1V side slopes would be utilized from natural grade at -4 to -15. TRS would be utilized from -15 to -39. The width of the excavation is from top of bank to top of bank of the diversion channel is 64.00 feet. The Inverted Siphon excavation from top of bank to the Intake and Outlet structures is 82.83 feet. The excavation will be dewatered and performed in the dry. The design of the TRS for the vertical excavation will be designed by the Contractor.

5.7.4 Siphon Piping Design Criteria

With number and diameter of pipes provided by the completed interior drainage model, the Inverted Siphon piping has been designed according to EM 1110-2-2902. 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 weight and flowable fill 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.

The Inverted Siphon piping is a 96-inch fiberglass reinforced pipe (AWWA C950, Fiberglass Pressure Pipe). 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 4 flexible joints for each Inverted Siphon pipe. They are located at the top of bank and toe of bank.

5.7.5 Inverted Siphon Geometry

The Inverted Siphon profile parallels the Diversion Channel and levee. The invert of the Inverted Siphon at the Intake and Outlet Structures is -13.5. The Inverted Siphon pipe then descends to 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 sheet pile cutoff tie-in to the Intake and Outlet Structure headwalls providing the flood protection as the levee slopes down to provide vehicular and personnel access for structure maintenance.

5.7.6 Intake and Outlet Structure Description

The Intake and Outlet Structures are designed as a U-Frame channel. There are 20 degree wing walls at the Intake 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 wall will also function as a floodwall tying into T-Walls on either side of the Intake Structure. The length of the Intake Structure is 46'-9". The width of the structure is 82'-10". The height of the Intake Structure is 15 feet with top of U-Channel wall EL 4.0 and an invert EL -11.0 that transitions down to EL -13.5 at the Inverted Siphon piping.

The height of the headwall is 29.35 feet with the top of wall at EL 15.85. PZ-22 steel sheet pile will be driven below the headwall to provide seepage cutoff. The Intake Structure feeds six 96-inch Inverted Siphon pipes. The channel is subdivided between each Inverted Siphon pipe location.

10-foot sluice gates are provided for each Inverted Siphon pipe at the headwall. Stoplogs are provided adjacent to the access deck at the front of the structure. The stoplogs adjacent to the access deck are for maintenance dewatering purposes. There is a 20-foot access deck at the front of the Intake structure. This deck will be designed for HS-20 loading. Additionally, there is a steel bar screen at the entrance to the structure to capture debris.

The Intake and Outlet Structures are pile supported on steel H-piles (HP14x73) with tension connectors.

5.7.7 Intake and Outlet Structure/T-Wall Load Combinations

The table below show the load combinations investigated in the design of the Intake and Outlet Structures and T-Walls:

Table 5.7-1: Load Combinations

Load Case	Name	Loads	Factor	Load Category
1	Construction	D + Ls + W	1.6	Unusual
2	Operating Condition	D + Ev + Eh + Hd + W	2.2	Usual
3	Maintenance	D + Ev + Eh + Hu + W	1.6	Unusual
4	Debris Impact	D + Ev + Eh + Hd + W + Id	1.3	Extreme

5.7.8 Gates and Trash Racks

The Intake and Outlet Structures each have six (6) 13-foot wide stoplog closures at the entrance, one for each bay, to facilitate maintenance. At the rear headwall each structure has six (6) 10-foot sluice gates, one for each Inverted Siphon pipe. All cast iron 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.

5.8 Marine Structures

5.8.1 General Description

The current design includes two protection circular sheet pile cells, one at the upstream and one located at the downstream training wall. Dolphin type cells spanning the intake will be investigated in the 60% design, the concern is obstruction of sediment flow. Hydraulic modeling will be required. The river side wharf is included in Section 9, Secondary Features. Riverside, temporary construction structures are addressed in Section 13.

5.8.2 Design Features

The circular, closed cells shall be constructed of straight PS-31 flat sheets and have a minimum diameter of 70ft. The final diameter shall be selected when the soil properties from the recent riverside borings becomes available. The upper 12 ft of cell shall be a concrete hoop designed to resist vessel impact. Trapezoidal energy absorbing devises will be added to the impact zone of both cells. The two cells have been included in both numerical and physical hydraulic models, their presence has negligible impact to sediment capture.

5.8.3 Design Criteria

The ship channel is approximately 900 feet from the nearest diversion structure. The design Vessel is the Handyman with a light draft of 23ft, and a draft of 50ft when fully loaded. The upstream training wall and protection cell are located on bank contours no lower than El -12. The downstream contours are lower, intake armoring will be added to El -14. It is assumed that ship traffic will not impact the protection cells. The cells shall be designed to resist barge traffic. The force from river currents shall be calculated in accordance with AASHTO LRFD Bridge Design Manual. Vessel impact forces shall be determined using guidance in AASHTO and PIANC Harbor Approach Design Guideline (2014) , Report No 121. Protection cells shall also be designed to resist vessel impacts from aberrant barges propelled into the cells by hurricane force winds. Cell stability design shall be done in accordance with EM 1110-2-2503, Design of Sheet Pile Cellular Structures.

5.8.4 Analysis and Design Summaries

The upper concrete ring is designed to distribute impact forces across the entire cell diameter. Energy absorbing devices shall be included in the design. The reduction in lateral force due to energy absorption shall only be considered in extreme load cases. Design load cases are:

Table 5.8-1: Load Combinations

1	High River Current plus Wind	D + WA +W+Id	1.3	Unusual
2	Low River current plus sediment build up	D + WA + W +EH	1.6	Usual
3	High Water , 6 Barge Tow	D + WA+ I	1.3	Unusual
4	High River 12 Barge Tow	D + WA + I	1.0	Extreme
5	Hurricane driven light Barge 50 year storm	D + WA+I	1.3	Unusual

Design calculations shall be submitted in the updated DDR (Jan2020) after receipt of the latest sol properties).

5.9 T-Walls under Hwy 23 Bridge

5.9.1 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.

5.9.2 Design Features

5.9.2.1 Base Slab and Stem

The base slab for the Conveyance Channel T-Walls is at EL 3.0 and the T-Wall monoliths extends 250 feet (50-foot per Monolith) 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 five 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'-10" and is the same for all monoliths. Base slab width and thickness is 15 feet and 3'-6", 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 30% 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.

5.9.2.2 Cut-off Wall Sheet Pile

The cut-off wall of sheet piling is provided to limit seepage to a tip elevation at -37.0 feet, 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.

5.9.3 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 Rev. 9, dated April 22, 2019.

Table 5.9-1: Hwy 23 T-Wall Design Load Case Summary

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
2	Water at Design SWL or Flowline (impervious)	9.35	-3.0	2.2(D+EH+EV+Hs+Hu)
3	Water at Design SWL or Flowline (pervious)	9.35	-3.0	2.2(D+EH+EV+Hs+Hu)
4	Water at Design SWL plus Wave, (impervious)	9.35	-3.0	1.6(D+EH+EV+Hs+Hw+Hu)
7	Water to TOW, Channel side Loading	15.85	-3.0	1.6 (D+EH+EV+Hs+Hu)
9	Reverse Head, NOV5a Overtopped, Pervious	1.0	8.4	1.3(D+EH+EV+Hs+Hu)

Notes: 1) No Unbalanced loads.
3) D= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

5.9.4 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 18-inch diameter open-end steel pipe 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-2-1100-2104 (Design of Concrete Hydraulic Structures). Moment calculations for the steam and base slab will be performed in the next design phase. 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 are used to confirm the adequacy of the stem wall and slab thickness.

The Hwy 23 T-Wall's stem is 2'-6" thick and 12'-10" tall. The base slab has a top slab elevation of 3.0 and is 3'-6" thick. The 18-inch diameter piles spaced at 7.5 feet o.c. have a tip EL -90.0 feet and -68.0 feet for batter and plumb pile simultaneously. The piles are embedded into the base slab 18 inches (one pile diameter) to create a fixed connection to reduce the deflection. See **Appendix D** for pile layout, tip elevations, sizes and other design features.

5.10 Training Walls

5.10.1 Training Walls Design

Training Walls are cantilevered concrete retaining wall structures (T-Walls) that extend upstream and downstream along the banks of the Mississippi River. The purpose of these walls is to guide the sediment-rich water towards the intake structure. There is a total of thirteen (13) Training T-Wall Monoliths. T1 through T8 are on the upstream side and have a TOW EL. -5.0; their base slabs stair-step downward from TOS EL. -17.5 TO EL. -47.0 as they approach the intake structure. T9 through T13 extend downstream and have a sloped TOW ranging from EL. -15.0 to EL. 5.0, respectively, and their base slabs also stair-step downward towards the intake structure from TOS EL. -27.5 to EL. -47.0.

Upstream, land side of the T-Walls will be backfilled with sand 10 ft wide from bottom and 30° from the vertical to EL -10.0 armoring and rip rap will lay on top of the sand backfill to TOW EL -5.0 (matches TOW elevation). Downstream a similar fill methodology will be used but the top elevation of the riprap slopes upward from EL -15.0 to approximately EL -2.0.

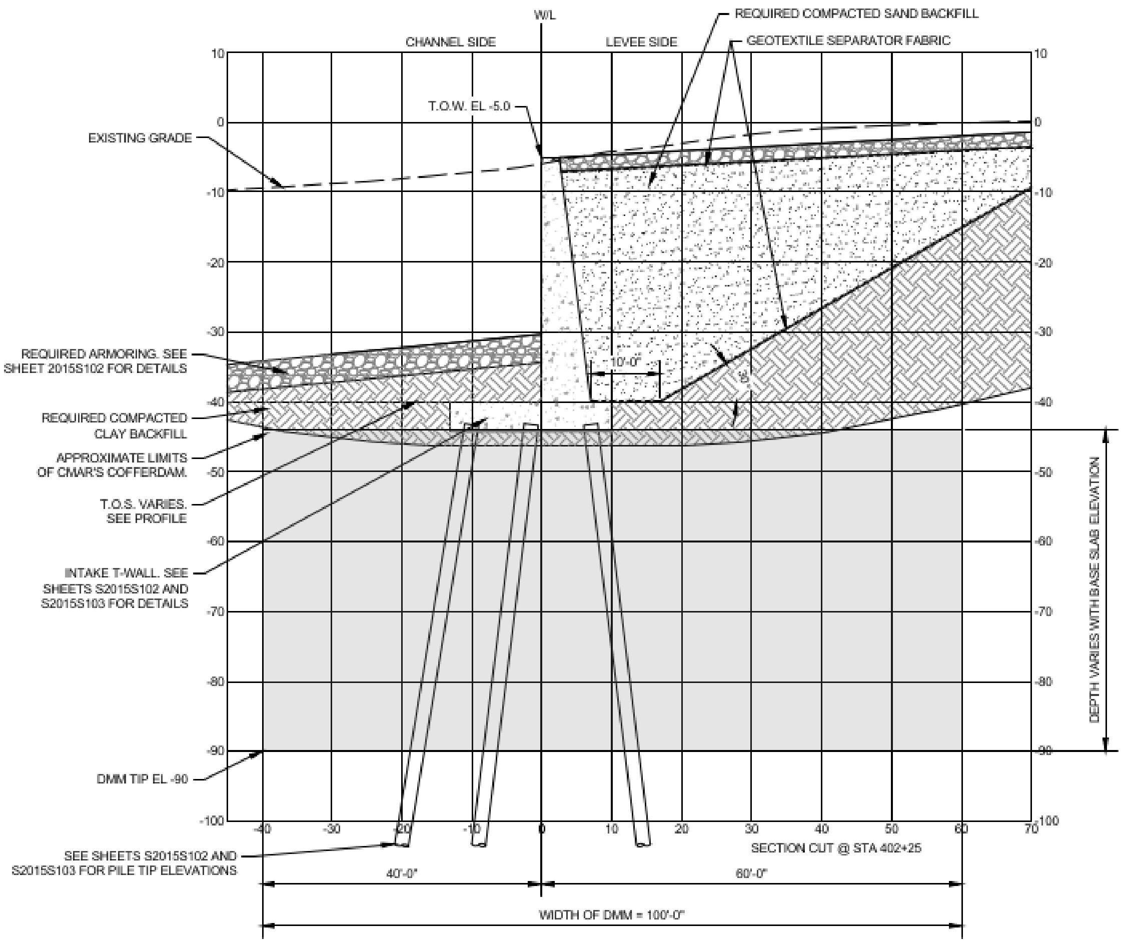


Figure 5.10-1: Typical Section of T-Wall

a. Design Criteria and Loading Conditions

The training walls shall be designed to guide the flow of water and sediment but not act as a flood protection barrier. The load cases are described below in ***Table 5.10-1***. Only two load cases are selected for this submittal, a construction load case and an operating load case.

Table 5.10-1: Training Wall Design Load Cases

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
1	Construction in the dry with Backfill and Wind (Unusual)	N/A	N/A	1.6(D+EH+EV+W)
2	Water at Design SWL (Usual)	5	0.0	2.2(D+EH+EV+Hs)

b. Analysis and Design Summaries

Analysis of the Training T-Walls is performed using a combination of hand calculations, excel spreadsheets and CPGA. Appropriate methods and safety factors are taken from ACI 318-14 and EM 1110-2-2104, *Strength Design for Reinforced Concrete Hydraulic Structures*. There are no unbalanced loads considered during the analysis due to the use of deep soil mixing beneath the wall structures.

There is a total of three wall designs for this submittal. Monoliths are grouped based on similar wall and soil heights. T1 through T3 and T9 are Design #1; T4 and T5 are Design #2; T6 through T8 and T10 through T13 are Design #3. For each load case the dead load, lateral and vertical soil forces, and hydrostatic forces are calculated. Unfactored (service level) load combinations are input in CPGA to design the pile type, length, batter, and spacing. Factored load combinations as described in ***Table 5.10-1*** are used to design concrete walls for shear and moment.

Walls vary from 2'-3" uniform thickness to a sloped section with top width of 2'-6" and base width of 7'-0". Base slabs are 4'-0" or 5'-0" thick. Monoliths T1 through T5 and T9 are founded on battered HP14x89 piles driven to EL. -120.0 to EL. -140.0. Monoliths T6 thru T8 and T13 are founded on battered 24" diameter x 0.5" wall pipe piles driven to EL. -150.0 to EL. -160.0.

6. CIVIL DESIGN

6.1 General

This section summarizes the Civil Designs included in the 30% Submittal for both the three-component diversion system and the secondary features. Civil features will be further developed and detailed in the 60% design phase, and this document will be updated accordingly. 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 a 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 feet 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, on the channel-side berm, 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 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 2500 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. Design of those tie-ins will be developed during the 60% design phase.

6.5 Outfall Transition Feature

The outfall transition feature is 2000 feet long and transitions the Conveyance Channel to the natural ground within the basin. Beginning approximately 1000 feet basin-side of the proposed NOV-NF-05a.1 levee, the outfall transition slopes between the Conveyance Channel at invert EL -25 and the natural ground of the Basin at approximate EL -4. The stability berms and guide levees along the outfall flare at a 23° angle for approximately 1300 feet. At the ends of the guide levees, a sheet-pile toe wall installed at the end of the armoring will minimize scour effects. Armoring of the outfall is further described in Section 6.6. Beyond the toe-wall, a flared sheet pile wall with riprap will tie-in to the guide levees and extend approximately 700 feet to the end of the outfall transition.

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 Conveyance Channel, the Transition section, and the Outfall was further developed during the 30% 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.

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 7.9 acres (in-the-wet) and 8.3 acres (in-the-dry). Additionally, 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 wet, to EL -40; and the portion of the channel contained within the intake U-frame cofferdam, to be constructed in the dry. 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 (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 ACM bank scour protection upstream and downstream of the intake. Riprap overlap from point of ACM disruption is prescribed by USACE District practice at 80 ft 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 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 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. 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 the figure superimposing LADOTD 130 lb Class Riprap gradation on Grade Stone B gradation plot.

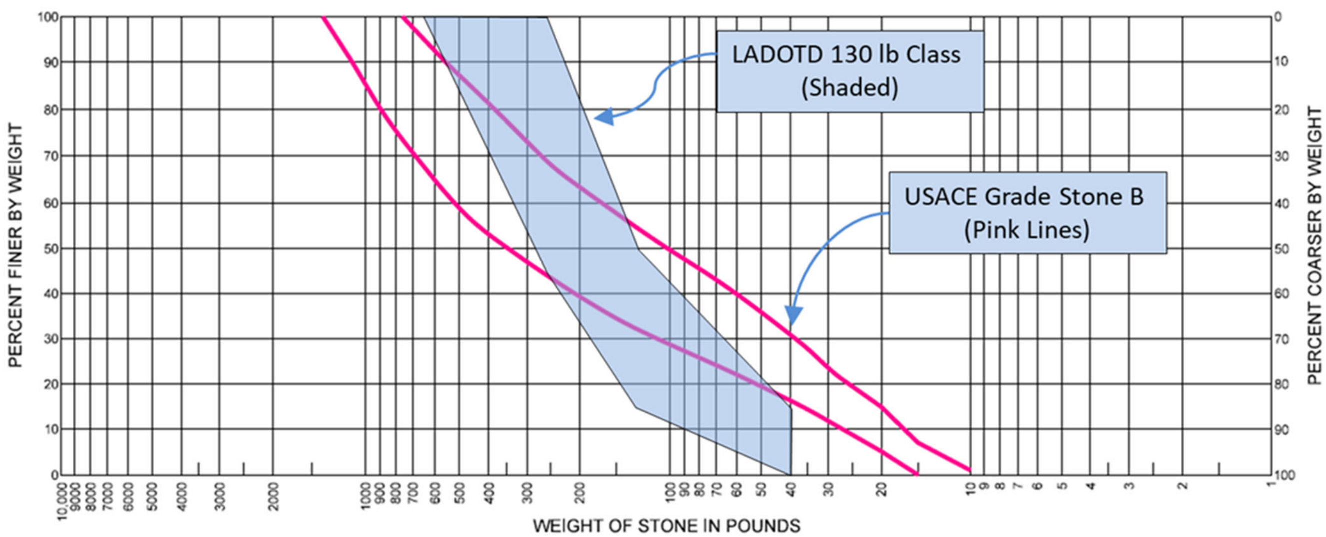


Figure 6.6-1: LADOTD 130 lb Riprap and USACE Grade Stone B Gradation Plots

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.

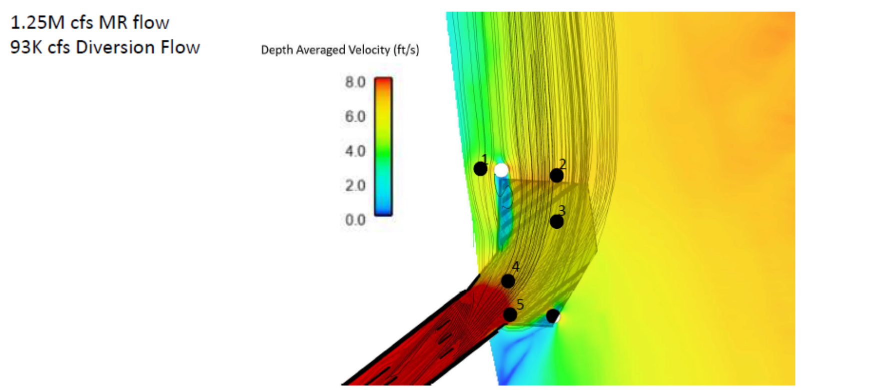
Riprap armoring terminates 50 feet from the toe of the levee to allow vehicular access on levee toe side. DAV was analyzed at this distance from levee to demonstrate resistance to model-predicted velocity by grass turf (6.0 ft/s when established in clayey soil, EM 1110-2-1601).

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 depth average velocity (DAV) for the controlling flow case (1,250,000 cfs MR flow; 93,000 cfs diversion flow). DAVs were provided from the June 2019 hydraulic model by FTN. Both the Isbash formula and EM 1110-2-1601 Eq. 3-3 were employed, and the results compared for DAV at five selected locations. Model output is shown in the following figure.



Point Locations UTM 15N (meters)			Flow Depth (ft)	DAV (ft/s)	Velocity at surface (ft/s)				Velocity from bottom (~ 6 ft) (ft/s)			
#	Easting	Northing			Vx	Vy	Vz	Vmag	Vx	Vy	Vz	Vmag
1	793966	3285379	10	4.8	0.1	-4.8	-0.1	4.8	0.5	-2.6	-0.1	2.7
2	794070	3285379	38	6.7	-0.2	-7.5	0.0	7.5	-0.6	-4.5	0.1	4.5
3	794065	3285330	49	5.7	-1.0	-6.8	0.0	6.9	-1.8	-2.0	0.0	2.7
4	794006	3285241	49	7.0	-4.0	-6.4	0.0	7.5	-4.6	-1.6	0.0	4.8
5	794036	3285160	49	7.9	-7.2	-3.7	-0.2	8.1	-6.5	0.3	0.0	6.5

Figure 6.6-2: Depth-Average Velocities, 93k cfs MBSD / 1.25M cfs MR, from June 2019 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 US Hurricane season, and there is a 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 10-year conditions were also evaluated. The 50- and 100-year storm conditions were provided by the USACE and are summarized in **Table 6.6-1** below.

Table 6.6-1: 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 elevation -40 feet and the sloped sides of the intake which extend from -40 to -10 feet with a 1:6 slope. The areas area designated in **Figure 6.6.3**. 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.

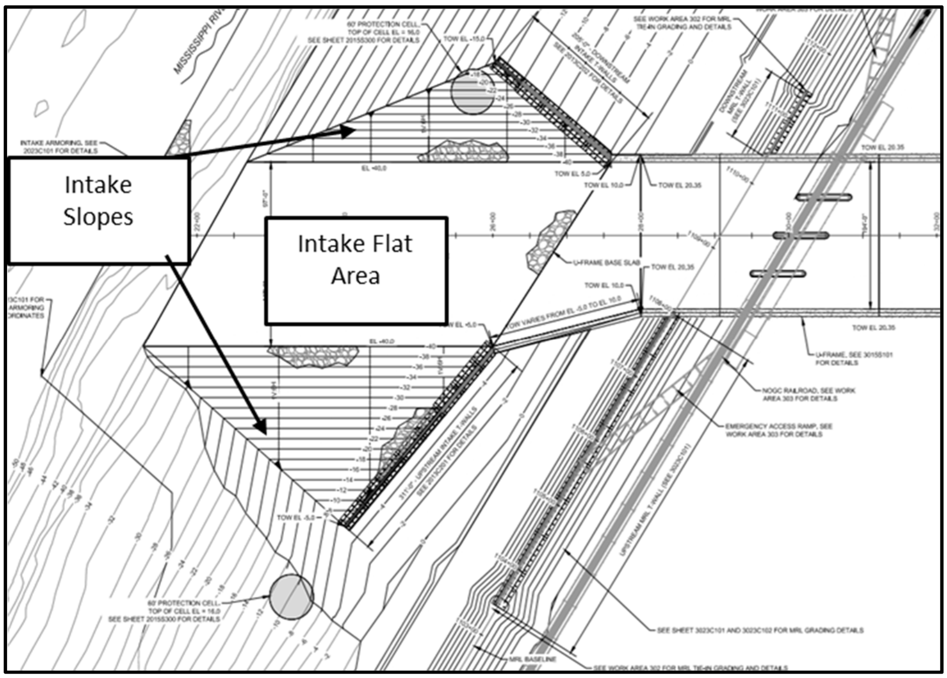


Figure 6.6-3: Armored sections of the Diversion Intake

The results are summarized in **Table 6.6-2** for the flat section of the intake and in **Table 6.6-3** for the sloped sections.

Table 6.6-2: 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	2.5	3.8	<0.25

Table 6.6-3: Summary of result for the sloped sections of the intake

Design Condition (yr)	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)
50	52.7	2.3	3.8	11.94
100	25.8	2.5	3.8	13.1

These armoring rip-rap sizes have been compared to those developed using the design flow conditions and it was determined that the rip-rap sizes required from the design flow conditions control the rip-rap 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 6.6-4: Calculated Minimum Riprap Gradations from DAVs

EM 1110-2-1601 Equation 3-3 and Isbash Inputs					
Point No:	**1**	**2**	**3**	**4**	**5**
Sf	1.1	1.1	1.1	1.1	1.1
Cs	0.3	0.3	0.3	0.3	0.3
Cv	1.25	1.25	1.25	1.25	1.25
Ct	1	1	1	1	1
d (ft)	**10**	**38**	**49**	**49**	**49**
γ_s (lbs/ft^3)	155	155	155	155	155
γ_w (lbs/ft^3)	62.4	62.4	62.4	62.4	62.4
V (ft/s)	**4.8**	**6.7**	**5.7**	**7**	**7.9**
theta (1V:5H)	**14**	**14**	**0**	**0**	**0**
phi	40	40	40	40	40
K_1	0.93	0.93	1.00	1.00	1.00
Low Turb. Isbash C	1.2	1.2	1.2	1.2	1.2
High Turb. Isbash C	0.86	0.86	0.86	0.86	0.86
Calculated Values					
EM Eq 3-3 D_{30} (ft)	**0.10**	**0.17**	**0.10**	**0.16**	**0.22**
Assumed D_{85}/D_{15}	4.0	4.0	4.0	4.0	4.0
EM Eq 3-3 D_{50} (ft)	**0.16**	**0.27**	**0.15**	**0.26**	**0.35**
Isbash D_{50} (ft) (Low Turb.)	**0.17**	**0.33**	**0.24**	**0.36**	**0.45**
Isbash D_{50} (ft) (High Turb.)	**0.33**	**0.64**	**0.46**	**0.69**	**0.88**
Min DOTD Class for Highest D_{50}	**10 lb**	**30 lb**	**10 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 was maintained at the 15% design thickness of 5 ft, since placement accuracy is considered low in flowing river water, and filter layers are omitted. Armoring layer thickness in-the-dry is established at the maximum D_{100}, 2.0 ft, with 1.0 ft underlying stone filter/foundation layer and geotextile.

6.6.3 Transition from Gates to Channel

As illustrated on Sheets 5013C201 and 6013C401, the Transition section is situated between the intake gates and the beginning of the Conveyance Channel. As shown, there is a 75-foot section of U-frame downstream of the gate monolith that remains flat at EL -40.0. Then the Transition climbs at a 2% slope for 750-feet to reach EL -25.0 at the beginning of the Conveyance Channel.

Throughout the ascent, the Transition also widens - beginning at the end of the flat section it changes at an angle of 7° from horizontal on each side, going from a top width of 194-ft to 234-ft over the first 175-feet. Then it widens even more rapidly, at an additional 7° on each side (a total of 14° from horizontal on each side), reaching a top width of 516-feet at the end of the 575-foot long segment. It then flares outward another 31° on each side (a total of 45° from horizontal on each side), reaching a top width of 809-feet over 146.5-feet of length. Finally, it returns to the horizontal maintaining the 809-foot top width over the final 70-feet.

The elevation and widening have opposite effects on the Transition section cross-sectional area. The raising of the bottom reduces the cross-section, while the widening of the channel increases it. As discussed in Section 3, these geometric changes create corresponding variations in the flow velocity, which must be considered when designing the protective revetment. In general, there is a reduction in the flow velocity as water moves through the Transition section, indicating that the widening of the channel has a greater effect in increasing the cross-sectional area than the upward shift of the bottom has in reducing the area. This is illustrated in **Table 6.6-5**, which shows the cross-sectional area at various locations along the alignment of the Transition section, assuming a constant WSE of 6.5-feet.

Table 6.6-5: Cross-Sectional Areas at Various Locations along the Transition

Station	Position Description	Elevation (ft, NAVD88)	Cross-Sectional Area* (ft^2)
35+15.00	End of flat U-Frame Monolith \ Beginning of 2% slope upward	-40.0	9,021
36+90.00	End of first 7° flared section \ On upward slope	-36.5	10,062
42+65.00	End of second 7° flared section \ End of upward slope \ Beginning of side slopes	-25.0	13,343
44+11.00	End of 31° flared section	-25.0	14,510
44+81.54	End of horizontal section \ Beginning of typical section	-25.0	14,042

*Assuming a Constant WSE = 6.5-ft, NAVD88

6.6.3.1 Numerical Hydraulic Modeling

Table 6.6-6 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 6.6-4**; they are at distances of 0-ft, 175-ft, 530-ft, and 750-ft, respectively, downstream of the gate monolith.

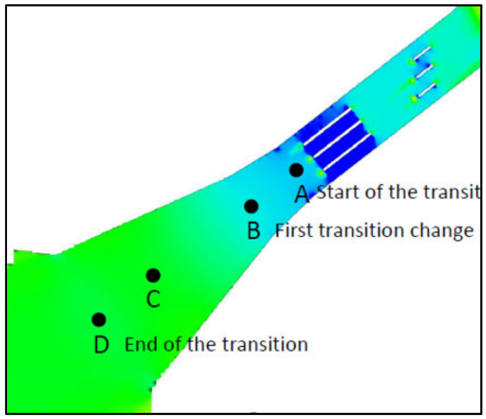


Figure 6.6-4: Locations of Transition Points

Table 6.6-6: 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 6** 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 6.6-6** 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, which varies from an elevation of -40-feet at the end of the U-frame monolith to -25.0-feet 750-feet downstream. Thus, the flow depths range from 47.1-feet to 31.4-feet, corresponding to the combination of bottom and water surface elevations. **Table 6.6-7** was generated by applying Equation 3-3 using the derived numbers.

Table 6.6-7: 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 6.6-7** 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 FOSs 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-ft, 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-ft, 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 6.6-5**.

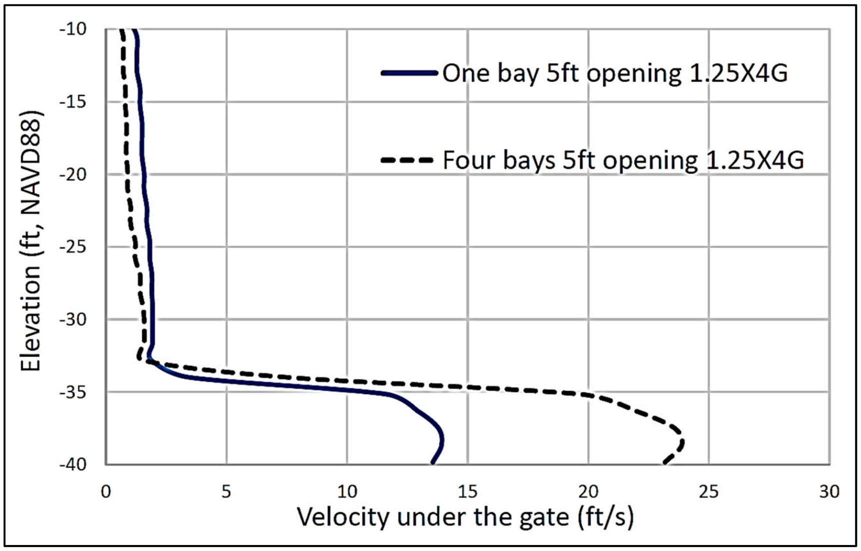


Figure 6.6-5: Velocity Under a Partially Open Gate or Gates

Additional hydraulic analyses show the velocities at 10-ft, 40-ft, 70-ft, and 100-ft past the gate opening into the Transition section, with all gates partially open 5-feet. The data from these analyses along with a depiction of where the various data collection points are located are plotted in **Figure 6.6-6**.

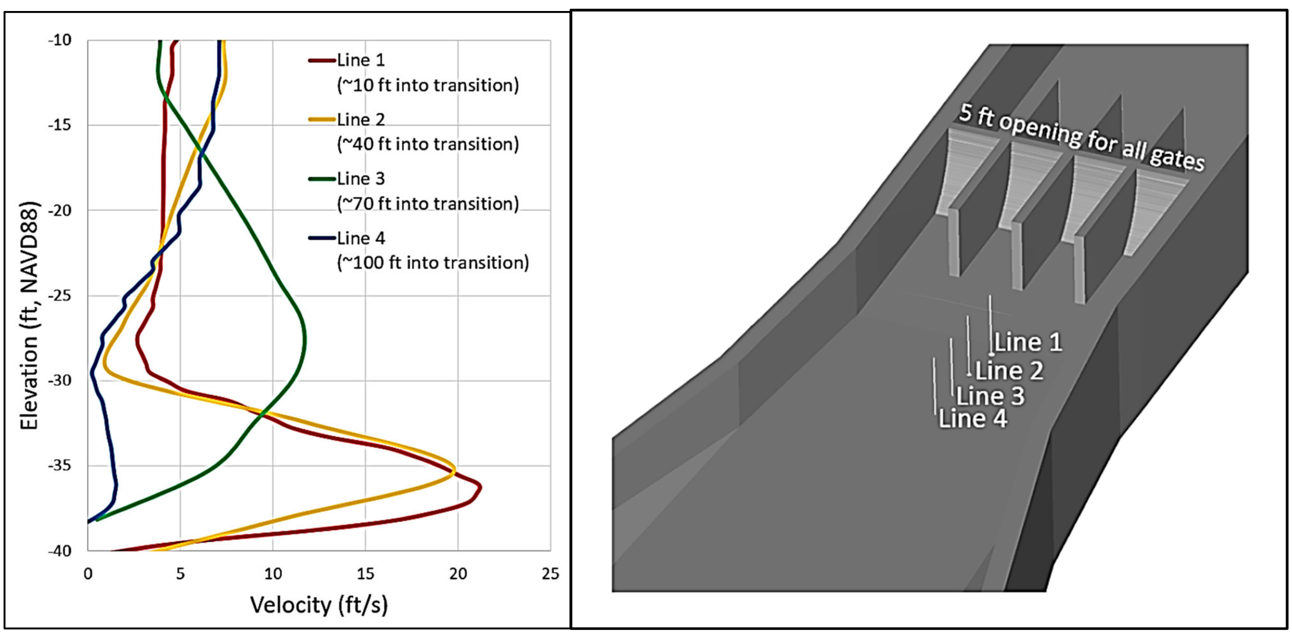


Figure 6.6-6: Velocities at Various Locations for Partially Opened Gates

As **Figure 6.6-6** shows, the maximum flow velocities up to 40-ft away from the gates range from 20 – 22 ft/s; they slow to 12 ft/s 70-ft from the gates; and they are less than 10 ft/s 100-ft into the Transition. The data are presented in **Table 6.6-8** along with the corresponding riprap size and weight that would be required to resist such velocities.

Table 6.6-8: 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 6.6-8** shows, very large, Derrick stone size 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-ft yields a D_{50} size of 1.1-ft 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-ft and a W_{50} weight of 130 lbs. **Figure 6.6-7** presents the gradation curves for the LADOTD 130 lb Class riprap. The required layer thickness calculated from Equation 3-3 is 1.75-ft. Assuming the riprap is installed in-the-wet, then the required thickness becomes 1.5 x 1.75-ft = 2.63-ft, 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-ft mark from the gates can be protected by a 3-feet 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.

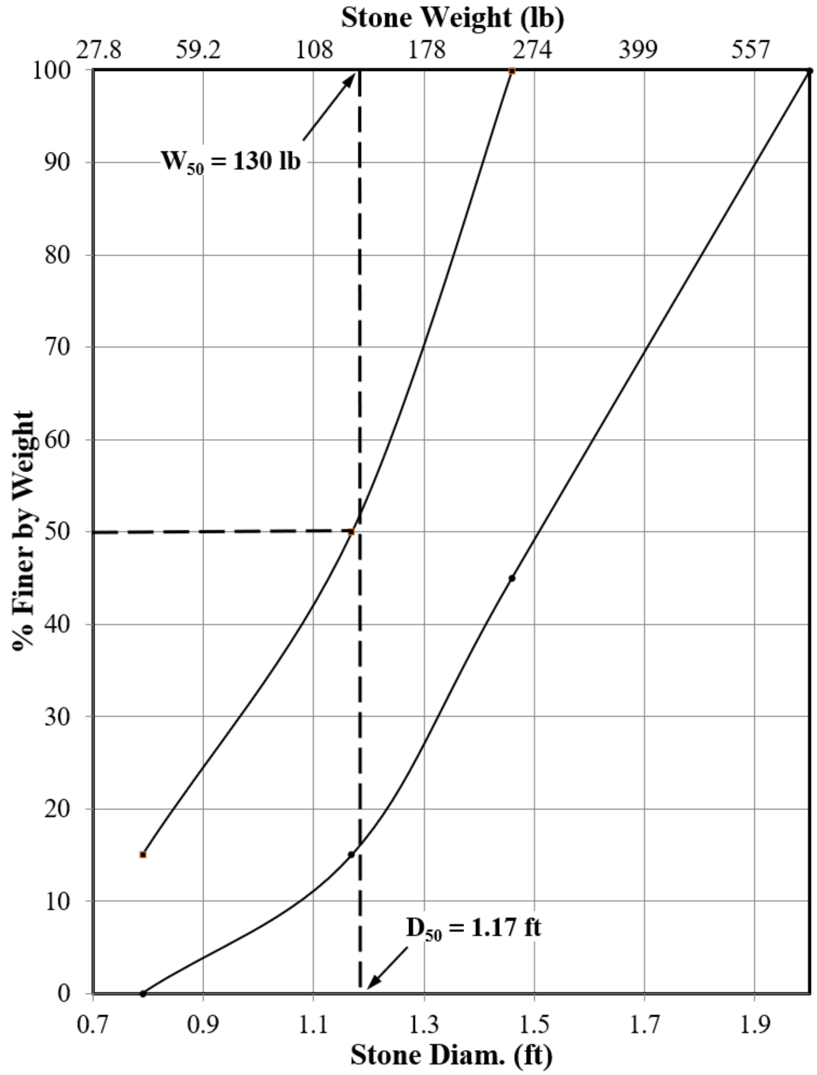


Figure 6.6-7: Gradation Curves for LADOTD 130 LB Class Riprap

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-ft, 2-ft, 4-ft, 6-ft, and 10-ft from the bottom. The water level downstream of the gates was set at 0-ft 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-ft 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-ft, 40-ft, 70-ft and 100-ft 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 6.6-8**.

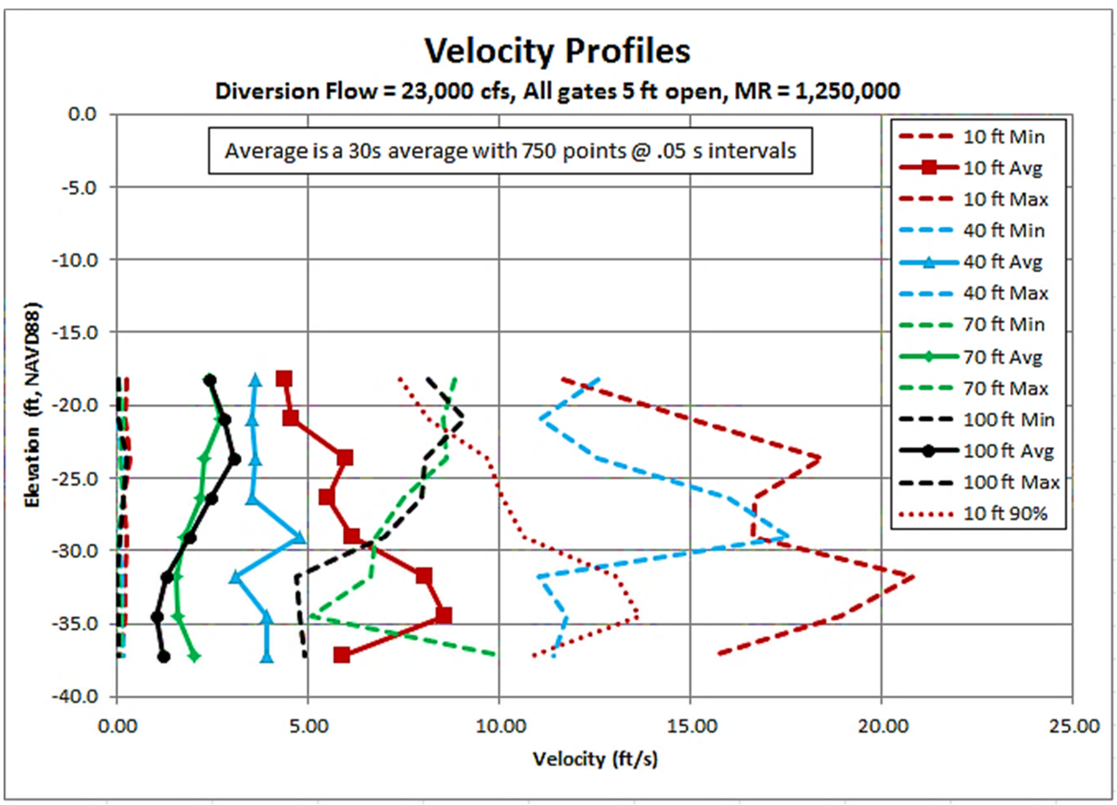


Figure 6.6-8: Velocity Profiles in the Transition Section - All Gates Open 4-ft, River at 1.25 M cfs, and Diversion Flow of 23,000 cfs

As **Figure 6.6-8** 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 10-feet from the gates is less than 10 ft/s. The maximum velocity at 40-ft from the gates exceeds 15 ft/s, while the average velocity at that location is less than 5 ft/s. At 70-ft from the gates, the velocity can reach 10 ft/s, with an average velocity of 2 – 2.5 ft/s. At 100-ft 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-feet 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-ft 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-ft with a W_{50} weight of 30 lbs, as shown on **Figure 6.6-9**. The requisite thickness is 1.22-feet; multiplying by 1.5 for in-the-wet placement results in a layer thickness of 1.83-ft, which is rounded to 2-feet.

For the 30% Design, 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 will be performed during the 60% Design 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.

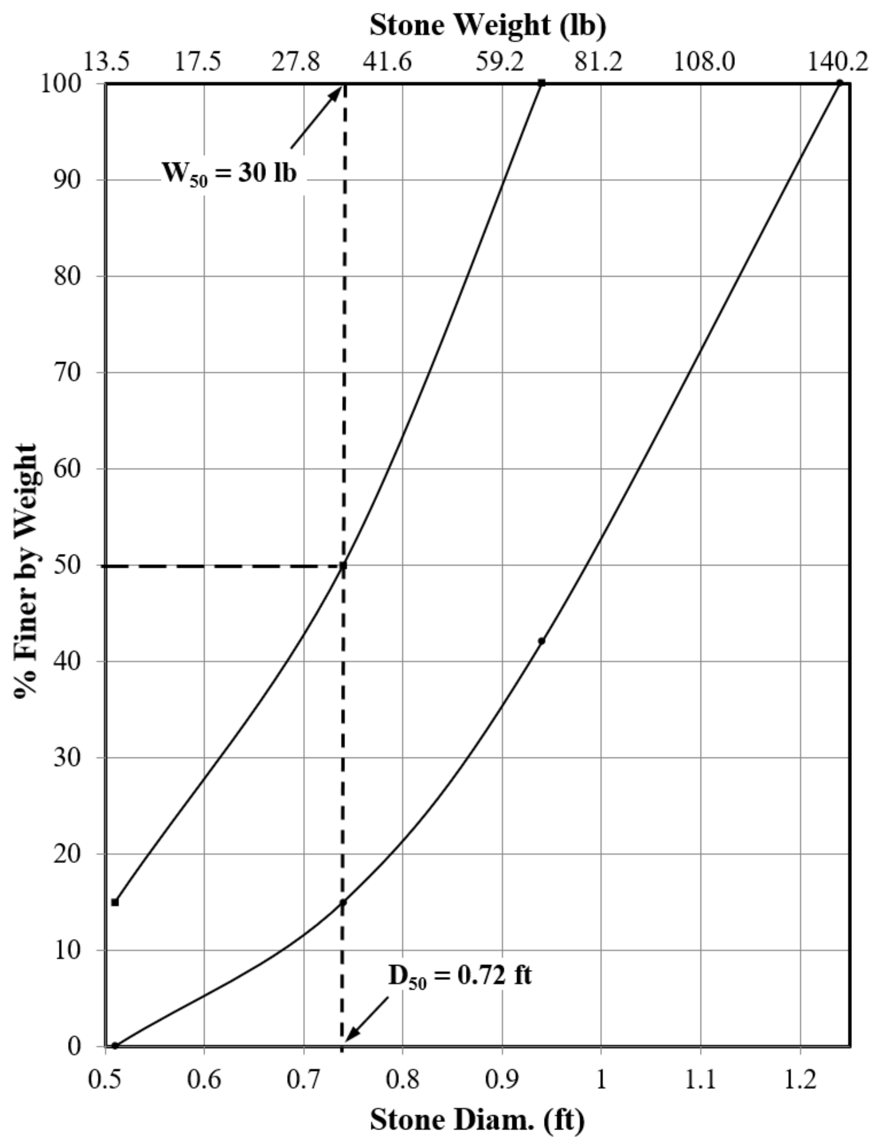


Figure 6.6-9: Gradation Curves for LADOTD 30 LB Class Riprap

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 6.6-10**.

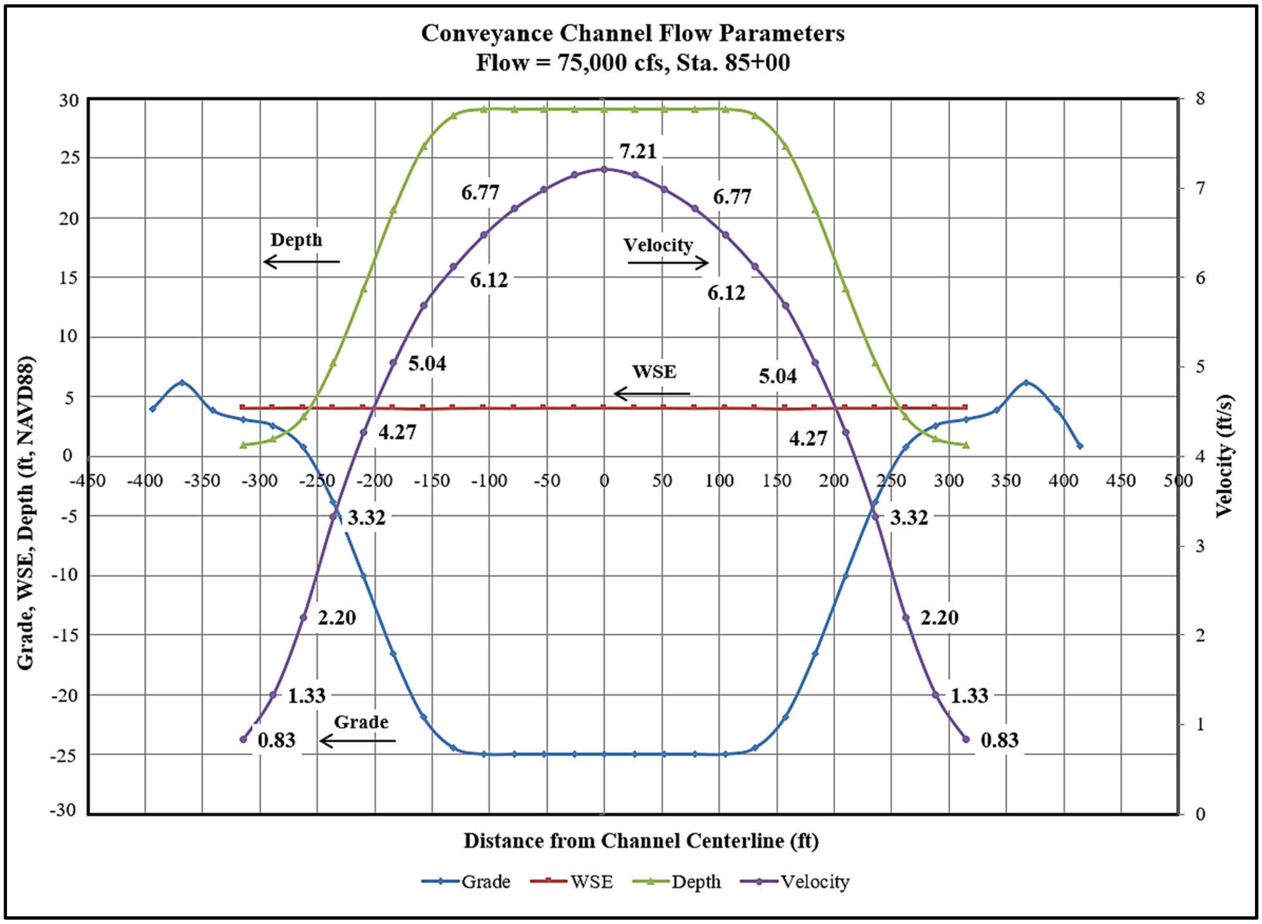


Figure 6.6-10: Water Surface Elevation, Depth & Velocity at Mid-Channel (Sta 85+00) for Normal Flow Conditions (75,000 cfs) [Velocities are Depth-Averaged Velocities]

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-ft. 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 6.6.11**.

Design of Rip-Rap Revetment							
Basic Input Data	Velocity of water v =	7.21	ft/s				
	Unit Weight of Stone γ_s =	155	lb/ft³	← 155 lb/ft.³ is the conservative value assumed by the USACE			
	Unit Weight of Water γ_w =	62.4	lb/ft³				
	Specific Gravity of Stone sg =	2.48	-----				
Method	**Equation**	**Additional Parameters**			**D_{50}**	**Weight**	**Layer Thick***
EM-1601 (USACE) 1994	$D_{30} = S_f C_s C_v C_t d\left(\left(\gamma_w/(\gamma_s-\gamma_w)\right)^{0.5}\left(v/(K_1 g d)^{0.5}\right)\right)^{2.5}$						
	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					
	γ_w = Unit weight of water, lb/ft³	62.4					
	γ_s = 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 6.6-11: Design of Rip-Rap 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-ft, 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-ft, which is approximately 60% larger than the required 0.318-ft, 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-ft, or 2) 1.5 times the upper limit of the D_{50} which is 1.5 x 0.65-ft = 0.98-ft. Thus, the required layer thickness for in-the-dry construction is approximately 1-ft. 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-ft thick layer.

Figure 6.6-12 presents the gradation curves for the LADOTD 10 lb Class riprap.

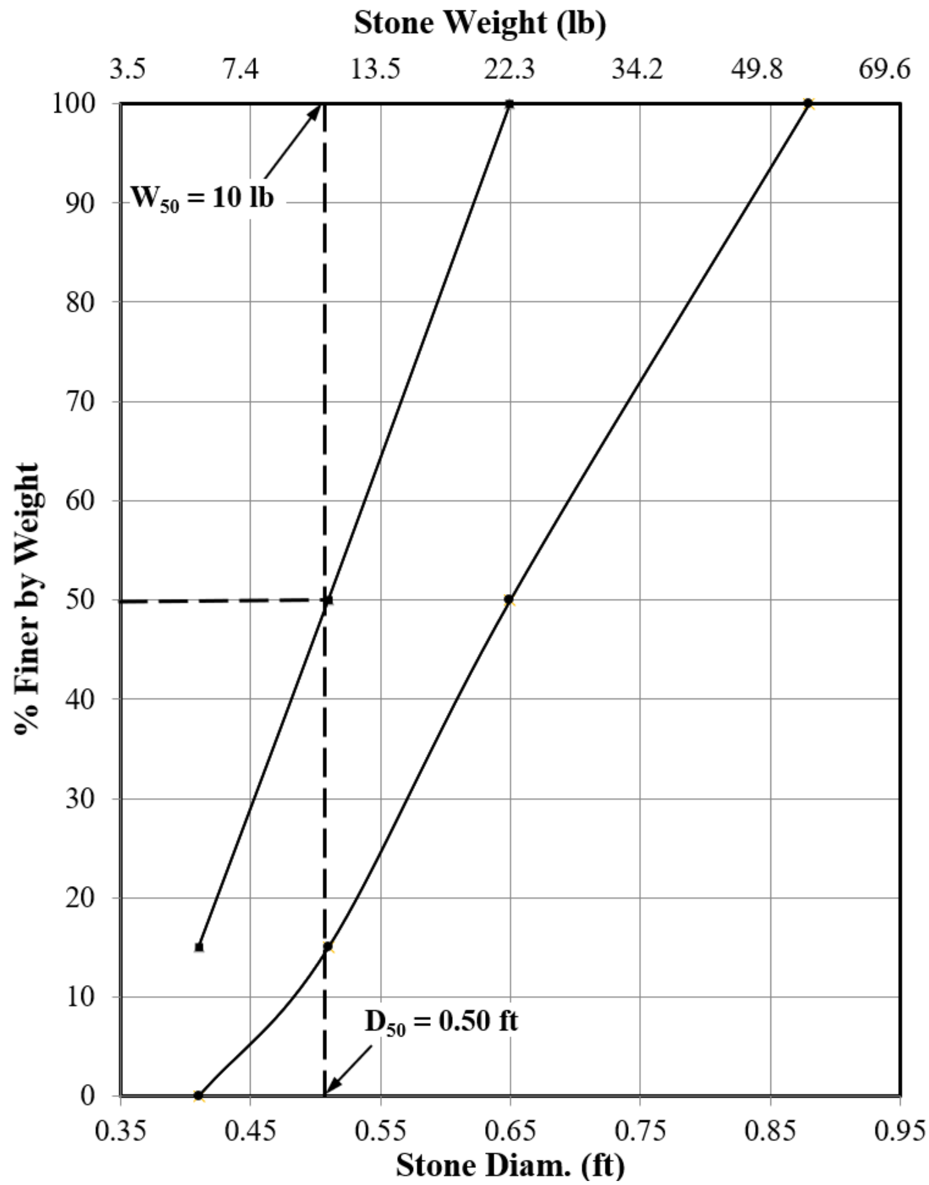


Figure 6.6-12: Gradation Curves for LADOTD 10 LB Class Riprap

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-ft 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 0-ft NAVD88. 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 an may impact the armoring requirements for outfall and channel features. An analysis ws compelted 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: <u>Elevations for Design of Hurricane Protection Levees and Structures Report</u>. The report provides data for sections along the non-Federal NOV levee as well as many other Levee systems. The 50- 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 6.6-9**.

Table 6.6-9: 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 6.6-10**.

Table 6.6-10: 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 6.6-11**.

Table 6.6-11 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 details of the analysis are available in the BODR **Appendix H.11**. The results for each diversion component is summarized in the flowing Tables. These armoring rip-rap sizes have been compared to those developed using the design flow conditions and it was determined that the rip-rap sizes required from the design flow conditions control the rip-rap sizes.

Table 6.6-12: Summary of Wave-based Rip-Rap Size for Levee for 50 Year Design Conditions

Water Depth(ft)	Wave Height Hs (feet)	Wave Period (seconds)	Deepwater Wave Length (ft)	Wave Breaking	Rip Rap Size (lbs)	Applicable Range in Channel (measured from the outfall)
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 6.6-13: Summary of Wave-based Rip-Rap Size for Levee for 100 Year Design Conditions

Water Depth(ft)	Wave Height Hs (feet)	Wave Period (seconds)	Deepwater Wave Length (ft)	Wave Breaking	Rip Rap Size (lbs)	Applicable Range in Channel (measured from the outfall)
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 6.6-14: Summary of Wave-based Rip-Rap Size for Channel Slope for 100 Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.1	4.1	33.1	0 – 1100
1.05	4.1	8.3	1100 - 2500
0.52	4.1	2.1	2500 - 3500

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

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.9	4.8	68.3	0 – 1100
1.45	4.8	17.0	1100 - 2500
0.73	4.8	4.3	2500 - 3500

Table 6.6-16: Summary of Wave-based Rip-Rap Size for Channel Bottom for 50 Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.1	4.1	<0.25	0 – 1100
1.05	4.1	<0.25	1100 - 2500
0.52	4.1	<0.25	2500 - 3500

Table 6.6-17: Summary of Wave-based Rip-Rap Size for Channel Bottom for 100 Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.9	4.8	<0.25	0 – 1100
1.45	4.8	<0.25	1100 - 2500
0.73	4.8	<0.25	2500 - 3500

Table 6.6-18: Summary of Wave-based Rip-Rap Size for Channel Berm for 50 Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.1	4.1	0.5	0 – 1100
1.05	4.1	<0.25	1100 - 2500
0.52	4.1	<0.25	2500 - 3500

Table 6.6-19: Summary of Wave-based Rip-Rap Size for Channel Berm for 100 Year Design Conditions

Wave Height (feet)	Wave Period (seconds)	Rip Rap Size (lbs)	Applicable Range in Channel (measured form the outfall)
2.9	4.8	52.0	0 – 1100
1.45	4.8	0.63	1100 - 2500
0.73	4.8	<0.25	2500 - 3500

Table 6.6-20: Summary of Wave-based Rip-Rap Size for Outfall Ramp for 50 Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Rip Rap 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 6.6-21: Summary of Wave-based Rip-Rap Size for Outfall Ramp for 100 Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Rip Rap 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 6.6-22: Summary of Wave-based Rip-Rap Size for Outfall Ramp Side Slopes for 50 Year Design Conditions

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

Table 6.6-23: Summary of Wave-based Rip-Rap Size for Outfall Ramp Side Slopes for 100 Year Design Conditions

Ramp Location	Water Depth (feet)	Wave Height (feet)	Wave Period (seconds)	Rip Rap 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-ft from the end of the Conveyance Channel 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-foot 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 reduces, 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 6.6-13** and **6.6-14** present the modeling results. The cases are described in **Figure 6.6-15** 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.

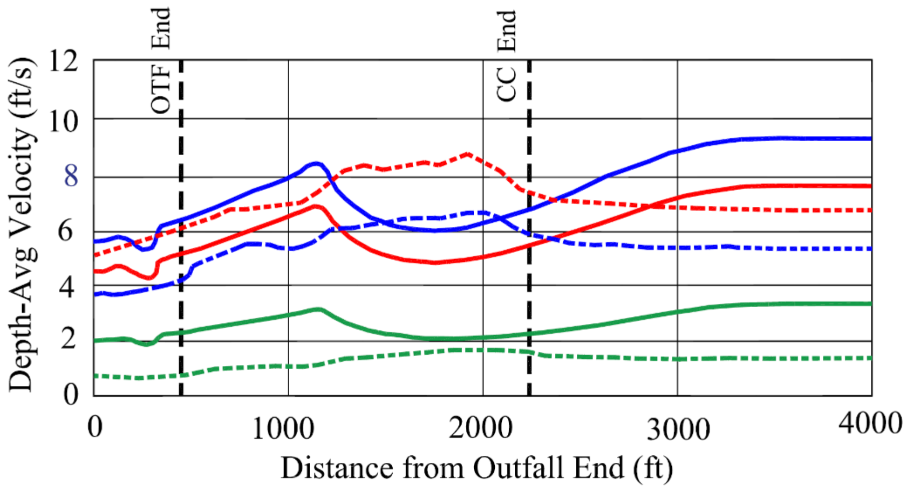


Figure 6.6-13: Depth-Averaged Velocities in the Outfall section for various model cases

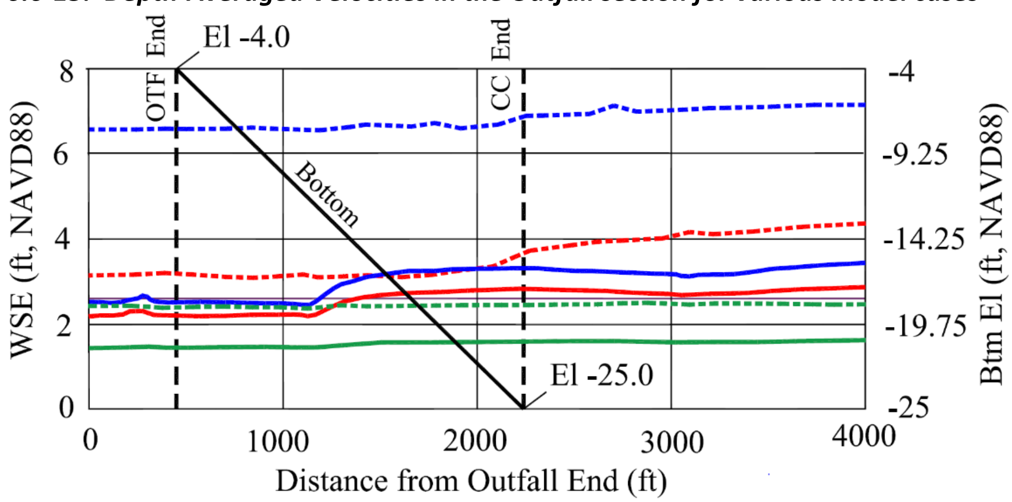


Figure 6.6-14: Water Surface Elevations in the Outfall section for various model cases

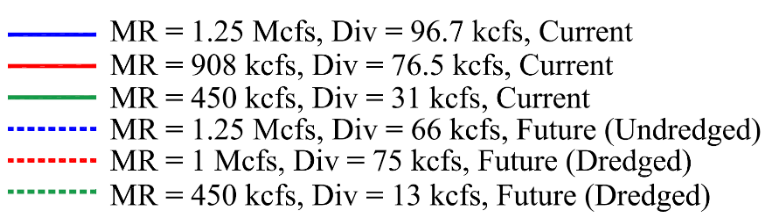


Figure 6.6-15: Legend for Figures 6.6-10 & 6.6-11 describing the various model cases

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 recommend for erosion protection. However, the depth of water in the Outfall section is less that it is throughout the Conveyance Channel because, as shown on **Figure 6.6-15**, 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 6.6-24 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 6.6-24: 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 6.6-6** 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 6.6-13** and **6.6-14** 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 for the 30% Design.

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. For the 30% Design the protection within the Outfall section will be a 2-ft 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. The optimum configuration will be further investigated in the 60% Design phase 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 -4 feet. 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 Beneficial Use of Materials (BUM)

The AECOM Design Team (DT) performed a conceptual analysis of two BUM placement areas during the 15% BOD Phase for the Mid Barataria Sediment Diversion (MBSD) Project. The two sites included the Wilkinson Canal Marsh Creation Area and the Bayou Dupont Marsh Creation Area. As a part of the 30% Design Phase, CPRA has asked the DT to assess additional alternative Beneficial Use of Excess Material (BUM) placement areas to assist the Environmental Impact Statement (EIS) Team. These alternatives included additional placement areas within Barataria Basin, the fast lands within the protected side of the New Orleans to Venice (NOV) levee, and filling in the borrow pits. The existing areas evaluated during the 15% BOD Phase was updated based on refined quantities. The alternative placement areas are listed below and shown in **Figure 6.7-1**.

- Shallow Water Placement Areas within Barataria Basin
    - Bayou Dupont Marsh Creation Area
    - Wilkinson Canal Marsh Creation Area
    - Outfall Transition Feature Marsh Creation Area (North & South)
    - Shell Marsh Creation Area
- Fast Lands Placement Areas
    - Northern Fast Land Placement Area
    - Southern Fast Land Placement Area
- Borrow Pit Placement Area

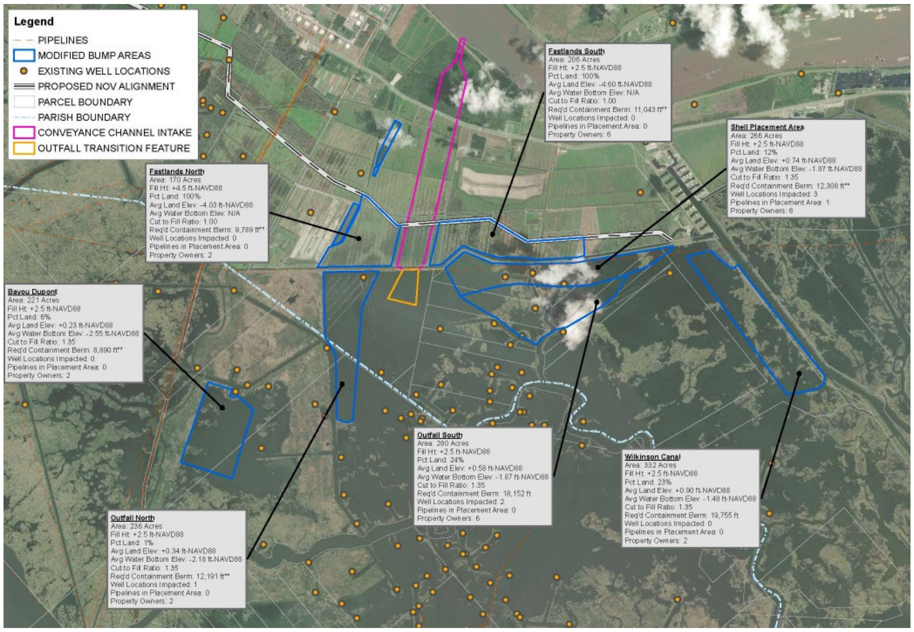


Figure 6.7-1: BUM Alternative Placement Areas

Prior to moving forward a specific placement area for final design and construction, the DT will await direction from the CPRA on the preferred location.

6.8 Utility Relocations

During the 15% BOD Phase for the Mid Barataria Sediment Diversion (MBSD) Project, the AECOM Design Team (DT) identified the utilities within the MBSD Project footprint. 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 – XXkw distribution line located within Hwy 23 right of way and conveyance channel
- Plaquemine's Parish Water Line – 20-inch AC line located within Hwy 23 right of way

Meetings were held with all four utility companies during 2019. During these meetings, the DT, CPRA and CPRA PMT were in attendance and informed each company with information about MBSD and how MBSD affects their utility. This document will discuss the relocation approach provided by the four major utility companies.

6.8.1 Shell Pipeline Company

Shell Pipeline operates a 20-inch crude pipeline within the Barataria Basin which is located approximately 25 feet from the flood side toe of the NOV levee. The Shell line will be affected by the dredging within the outfall transition feature 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. At a minimum, Shell will be required to relocate their line to a depth that is 15 feet outside of the lowest excavation of the outfall channel or outside the potential scour area, whichever is deeper. **Figure 6.8-1** shows a conceptual plan and profile developed by the DT for budgetary purposes only. Shell will develop their own company plan for the relocation.

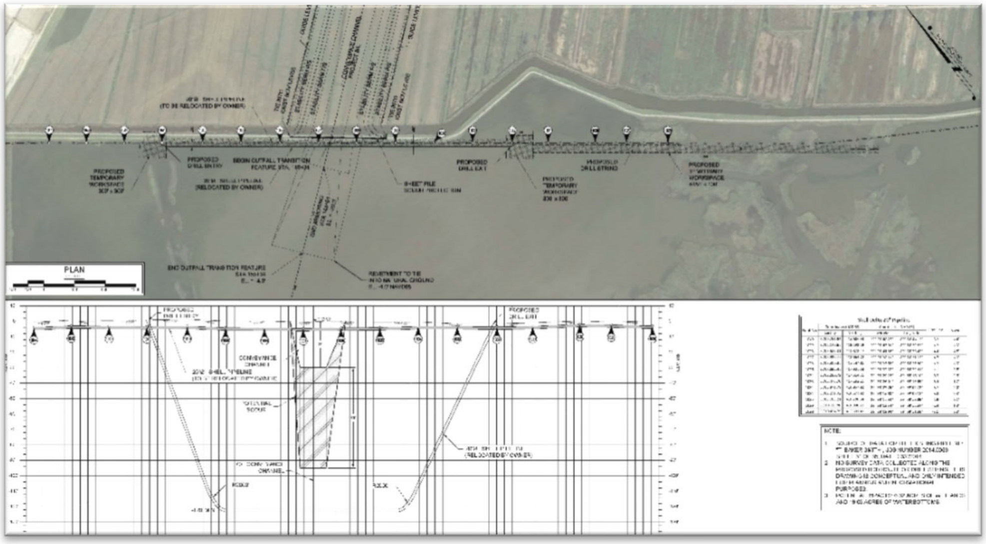


Figure 6.8-2: Conceptual HDD Plan & Profile

Based on our meeting held on January 15th, 2019, Shell anticipates executing their relocation in three phases after CPRA provides Shell with a Notice to Proceed.

- Identify Phase – 2 months
- Design & Permitting Phase – 4 months (this does not include agency review time for approved permit)
- Construction Phase – 4 months

6.8.2 Energy 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 for relocation as a part of the Class 5 estimate provided. Meeting notes from original engagement with Entergy and the Class 5 estimate can be found in Appendix XX. Based on the Class 5 estimate received, it is anticipated that the relocation will take approximately 18 months to complete. **Figure 6.8-3** shows the schematic for relocating Entergy's transmission line.

Note: Entergy Transmission stated in the meeting that they will require an 8 week lead time prior to any shut downs of the transmission line. CMAR Contractor will be required to coordinate any schedule shutdowns based on their construction schedule. See meeting notes in appendix.

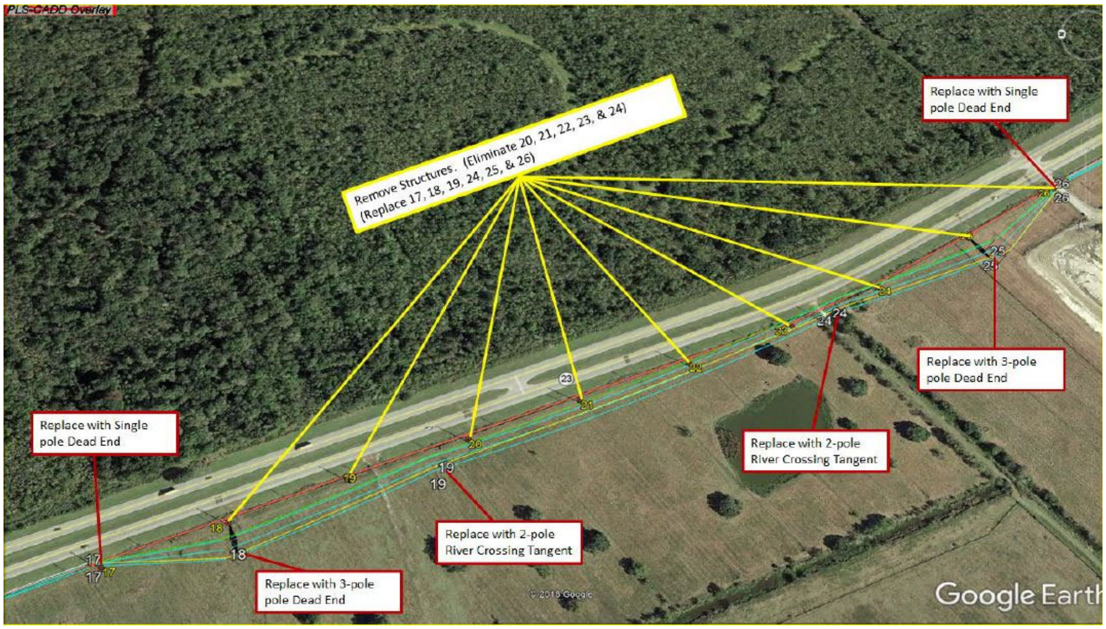


Figure 6.8-3: Entergy Transmission Relocation Layout

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. Due to the inhouse equipment that Entergy Distribution uses to maintain their infrastructure, the preferred option for relocation would be an HDD under the channel. Location and depth of the HDD will need to consider the Hwy 23 flood walls along the conveyance levee which will have a sheet pile cutoff wall and pile supported foundation. The DT will provide this info to Entergy Distribution for their design and construction needs. Based on our kickoff meeting on January 15th, 2019, it is anticipated that Entergy will install two (2) conduits for each existing line for a total of four (4) conduits under the conveyance channel. Each conduit is estimated at 6-inch diameters. The meeting notes with Entergy Distribution is located in Appendix XX. No formal schedule or estimate has been provided by Entergy Distribution to date.

6.8.4 Plaquemine Parish Waterline

Plaquemine 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. PPG preferred the relocation under the channel as stated in the meeting notes shown in Appendix XX.

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 on each side of the highway.

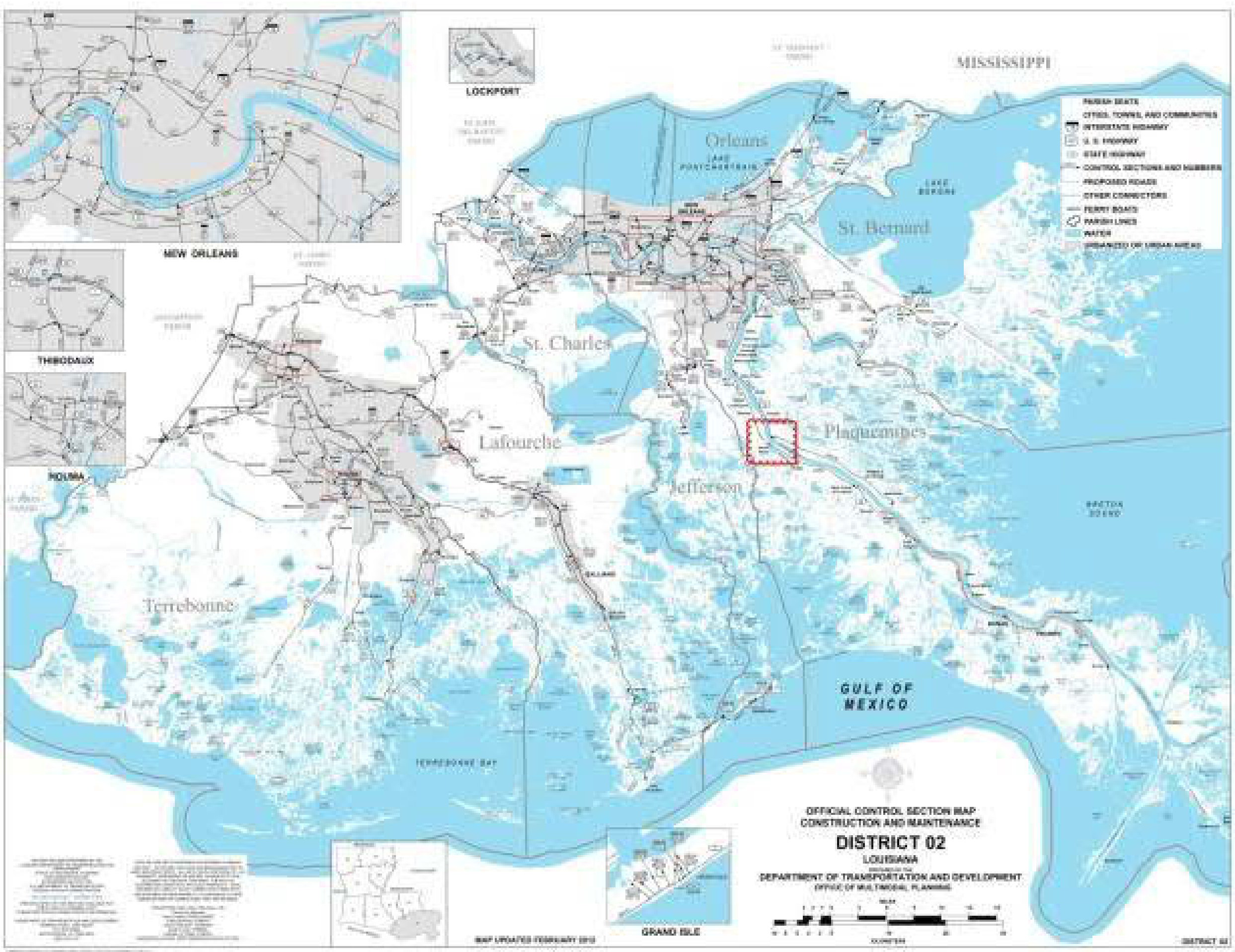


Figure 7.1-1: Hwy 23 Location Map

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. As of November 2019, the 90% Preliminary Plans for the proposed Hwy 23 Bridge are under review by LADOTD.

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. The traffic study is currently under review by LADOTD.

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 to 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 feet) 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.

7.5 Maintenance of Traffic

A preliminary sequence of construction has been developed that utilizes the southbound pavement to maintain both directions 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 will occur in coordination with the CMAR as design progresses.

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 is currently under review by CPRA, and will be submitted to LADOTD for review.

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. Each pile bent consists of three 30-inch pipe piles tipped at approximately EL -135 feet NAVD88 supporting a reinforced concrete pile cap. Pile bents are spaced at 50 feet. The approach spans consist of pre-stressed concrete box girders supporting ballast decks.

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), eliminating the need for a flood-proof bridge. The bridge spans over the U-frame intake structure consist of steel girders supporting ballast decks. Three 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 maintenance access and emergency bypass of the intake structure. Access ramps from the levee crowns provide continuous access across the intake structure for wheeled vehicles. The second parallel bridge has the same structural design as the RR bridge and can accommodate a future second track if needed.

High river conditions throughout the 30% design phase prevented a complete geotechnical investigation for the RR bridge alignment. The 30% design relied upon geotechnical assumptions based on investigations for other features of the project. Future design phases will rely on geotechnical investigations specific to the railroad bridge alignment.

Temporary spur tracks are included to maintain rail service during construction. Further details will be added in the 60% submittal.

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 30% 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 in a reinforced concrete abutment that transitions to the pile-supported portion of the bridge.

In future design phases, the Design Team will re-evaluate the embankment using geotechnical data specific to the railroad bridge alignment.

8.2.3 Approach spans and bents

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

Three 30-inch by 0.5-inch steel pipe piles tipped at approximately EL -135 feet NAVD88 support each pile bent. Each pile is filled with concrete from the existing ground elevation to 20 feet below existing ground elevation. The piles support a 20-foot L x 3.5-foot W x 4-foot D reinforced concrete pile cap. The spacing of the pile bents is 50 feet. In future design phases, the Design Team will re-evaluate the pile tips elevations using geotechnical data specific to the RR bridge alignment.

Each span of the approach bridge consists of four 3.5-foot W x 4.5-foot D pre-stressed box girders. One and one/fourth inch-diameter steel tie rods spaced at 16'-2" connect the box girders transversely. The girders rest on the pile caps with 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.

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 W36x395 steel 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 58'-2 $^{13}/_{16}$" to 67'-5 $^{5}/_{8}$" to accommodate the skew. "Steps" in the outer walls and three interior concrete piers support the bridge spans over the intake structure on elastomeric bearing pads.

The girders support a ballasted RR deck. 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 maintenance access and emergency bypass of the intake structure. The second parallel bridge has the same structural design as the RR bridge and can accommodate a future second track if needed.

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 tainter 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 span.

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 Access ramps

Reinforced concrete access ramps provide access for wheeled vehicles from the MRL crown to the second bridge over the U-frame intake structure. Reinforced concrete spread footings on the side slopes of the MRL support reinforced concrete columns. Concrete bent caps at the tops of the piles support precast concrete bridge decks, which span 20 feet.

The ramps tie in to the levee crown at approximately +16.65 feet NAVD88 and tie in to the railroad bridge at approximately +20.3 feet NAVD88. Precast concrete grade crossings on the first bridge allow wheeled vehicle access over the tracks and onto the second bridge.

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.

The 60% design will include detailed design of the spur tracks with consideration given to NOGC's current and future operations, the planned PLT facility, and potential cultural resources.

8.2.8 Drainage

Drainage swales, ditches, and culverts have a design capacity based on the 25-Year, 24-hour design storm.

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 sections below provide general design criteria information, and Appendix A includes additional details.

8.3.2 Geometry

The 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% (1.25% preferred).

8.3.3 Loads

The Cooper E-80 train governs the design loading. 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.

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 maintenance access and emergency bypass along the Mississippi River Levee alignment and can accommodate a future second track. 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 alignments.

9. SECONDARY SITE FEATURES

Secondary site features include the site reservation buildings including the Administration Building, Safe House, Operations and Maintenance (O&M) Building, boat garage and storage building emergency generators, fuel tanks, sewer lift station, and other appurtenant features related to the operation of the diversion facility. Associated disciplines include architectural, structural, electrical and instrumentation, and mechanical and plumbing.

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, IBC 2015

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 and will thus require necessary buildings like an administration office, operation shops, safe house and control house 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/ramp. 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 Plaquemine 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 transportation surface 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 450 feet length by 260 feet width and will require approximately 6 feet of fill/ embankment to bring the final parking/surface drive grade from existing (assumed EL 4.0 +/-) to approximate EL 10.5 around the buildings and to EL 10.3 at perimeter with the low point for drainage structures at the Base Flood Elevation (BFE) (EL 10). The entire reservation fill area will be considered for surcharging or wick drained to be determined by geotechnical analysis. Location of the reservation area in relation to the construction cofferdam levee and excavation was evaluated and due to the large amount of fill required to backfill the excavation and the effect it will have on the design of the reservation buildings foundations if the footprint extends beneath the building foundations it was determined to shift entire site further to the south. This will require an additional 100 feet of permanent R/W beyond the already established 800 feet permanent R/W from project centerline which will then transition back to the original line in approximately 200 feet. The CMAR haul road will be re-established along the south R/W as the permanent access roadway at 24 feet width with the drainage ditch to parallel it. The river end (east) will have 24 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 Station 1122+75 and over the MRL to access the CMAR and permanent reservation site boat launch (see 9.6) in the river. The northbound access road from the turnout will provide access to the gated structure via 24-foot width 2-inch 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 48 inches diameter) throughout the site area to a drainage ditch outfall running parallel within the south R/W along access road then connecting to LA 23 ditch drain system, utility service such as sewer (treatment plant and lift station as per the building and occupant requirements), water service line to tie in with parish water distribution system via min. 12-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 (8-inch chain link and 12 foot long gates at entrance areas), parking lot (light pole standards) lighting through limits of the parking and access roads and separate building lighting, 12 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, 3-inch asphalt binder course on 12-inch stone aggregate and 12-inch compacted sand subbase with swale drainage from Hwy 23.

9.3 Safe House

The Safe House is a rectangular raised structure with two access stairways connected by a wrap-around exterior walkway. The Safe House has two parts: the living quarters and the generator room. The building area is 853-square-feet, not including exterior walkways and stairs.

The living quarters contain a small kitchenette and living room, two beds, bathroom with shower, and control panels for the gate structure. These living quarters are conditioned space. The generator room is unconditioned and houses a generator on a raised platform and a large fuel tank. The wall dividing these two parts is a 3-hour rated fire barrier.

The building is concrete masonry and the conditioned areas have an insulated stud and cavity wall. The entire roof is insulated and will have standing-seam metal panels that match the other buildings on the

Reservation in style and color. The roof drains with a gutter and downspout system. The building is sprinklered. The exterior walkways are part of the concrete structural system and the two stairs will be pre-engineered metal stairs.

Note that only the structural design of this building meets the requirements of FEMA P-361 and ICC 500 documents for designing Community Storm Shelters. The architectural and mechanical requirements for Community Shelters are not in line with the purpose of this building; they do not account for sleeping quarters and are always required to be fully accessible. This facility is intended to house three workers during a high-water event, providing a location from which to operate the gate controls. This facility is not intended to serve as a Community hurricane or tornado shelter.

The safe house is designed at the BFE level above a 100-year hurricane flood stage and designed for a 200-mph wind speed. The safe house building is constructed of impact resistant reinforce masonry walls, with a metal and concrete roof deck anchored for design wind speeds.

The roof consists of metal and concrete deck roof over steel joists, framed to reinforced masonry bearing walls.

The lateral force resisting system consists of reinforced masonry shear walls.

Safe House Operating Floor

The primary structural floor at EL +22.5 NAVD 88. is the operating floor. The electrical and mechanical and control equipment is mounted on this floor, subjecting it to the dead weight of the control equipment, and the vibration generated during the generator runtime operation. The floor is designed as a system of beam sections and precast slabs laid out about the various wall locations.

9.4 Administration Building

The Administration Building is a single-story office building. roughly rectangular in shape, the floor plan has minor setbacks for exterior alignment and a minor overhang at the main entrance and a flat roof pitched in both directions. The building is designed with three sections, with the conference room and office areas separated by an open lobby and eating area. The building area is 2,400-square-feet. The building is sited with the conference room overlooking the gate structure.

The conference room is sized to comfortably seat 20-24 people around a communal, non-hierarchical table. There will be automatic shades to block out daylight from all Conference Room windows for easy viewing of a large screen on one wall. The ceiling is a sloped wood plank system with hidden acoustical treatment, which also covers a portion of one wall. There is millwork for display and for refreshments.

There are three offices, one is larger for the administrator. The copier and millwork storage are in the communal entry corridor for all three offices. If the copier is not needed, there are other storage options that could be in the space. The mechanical and data closets are between the offices. Each office contains a window.

Between these two areas is the Lobby and Reception space, including a receptionist desk. The Lobby is open in plan to the Lunchroom. There are two adjacent restrooms. The Lobby has a lot of daylight, including clerestory windows facing north. The ceiling in the Lobby and kitchenette have the same acoustical treatment as in the Conference Room. The building entrance is into the Lobby, facing the receptionist desk, clerestory windows, and dividing wall. This dividing wall is an exciting opportunity for a bold graphic or decorative treatment relating to gate structure, wetlands building, the Mississippi River, or other topic. The interior lobby walls are one-hour fire-rated because the building is not sprinklered.

The building exterior walls are insulated 6-inch metal stud framing with a combination of masonry veneer and fiber cement board siding or panel finishes. There is a large amount of missile impact glazing within an aluminum storefront system for daylighting. All doors and the main entry have pre-engineered aluminum canopies which will match those of the Operations & Maintenance Building. The front entry ramp and the mechanical pad are screened by gabion walls, which will be welded wire cages filled with river rock in imitation of the sediment and river setting.

The building has a compound-pitched standing-seam metal panel roof, which will match the other buildings on the Reservation in style and color. The conference room roof has a parapet around the front and side walls. The entire roof drains with a gutter and downspout system.

Gravity framing consist of steel tubing (HSS) beams and square tube (HSS) columns. The upper roof consists of light gauge metal deck with insulation and a drop ceiling in some areas.

The ground floor is a concrete slab with grade beams and pile support.

Foundations consist of continuous grade beams and piles under structural walls and columns. The lateral force resisting system consist of concentrically braced steel braced frames with HSS braces. Interior braced frames will require foundation grade beams.

9.5 Operations and Maintenance Building

The Operations and Maintenance Building is a rectangular pre-engineered metal building with two parts: an open vehicle garage and a conditioned work area. The building area is 6,498-square-feet.

The vehicle garage contains two bays with 18-foot x 18-foot garage doors on both end walls. The interior clear height ranges from 24 feet to over 30 feet. The vehicle garage is open to the pre-engineered structure and vinyl-backed insulation. The walls are 8-inch Z-girts with metal panel exterior. The roof is sloped with standing-seam metal panels over Z-girts. One bay contains a 5-ton overhead crane. The floor of the vehicle garage bays slopes to a central trench drain with an oil/water separator. The end user needs to provide more information on the intentions for the crane during the next phase of development.

The conditioned work area contains an Office, Soils Laboratory, Shop, Lunchroom, Conference Room, Restrooms, and support spaces. This portion of the structure is a lean-to to the main portion and has a lower roof line. The exterior walls are non-bearing concrete masonry with metal panels above. These rooms are conditioned and all but the shop have acoustical tile ceilings. The end user needs to provide more information on the intentions for the Soils Laboratory and Shop during the next phase of development. The designers have included doors, utility sinks, limited millwork, and some power and lighting but needs feedback and detail from the end user.

The building is sprinklered. The wall separating the garage and work areas does not require fire-rating. The doors between these spaces are sliding doors to avoid interference with crane operation.

The final steel layout, member sizes, and all aspects of the structure will be calculated and designed by a Pre-Engineered Metal Building Manufacturer based on the specifications and will be reviewed by the

Architect and Engineer (A/E). There will be portal framing in bays selected by the manufacturer in coordination with the A/E.

The lateral force resisting system consists of concentrically braced steel frames with braces.

Interior braced frames will require foundation grade beams. Roof elevation changes will be arranged to occur at braced frame lines that are continuous vertically from the roof to the foundation without a horizontal offset.

The maintenance building has a 5-ton overhead crane in the interior bay.

9.6 Pole Shed

The Pole Shed for equipment storage is an open-air pre-engineered metal building with metal panels on three sides. There are 5 bays for storing equipment. The roof is sloped with standing-seam metal panels over Z-girts.

The final steel layout, member sizes, and all aspects of the structure will be calculated and designed by a Pre-Engineered Metal Building Manufacturer based on the specifications and will be reviewed by the Architect and Engineer (A/E). There will be portal framing in bays selected by the manufacturer in coordination with the A/E.

The lateral force resisting system consists of concentrically braced steel frames with braces.

Interior braced frames will require foundation grade beams. Roof elevation changes will be arranged to occur at braced frame lines that are continuous vertically from the roof to the foundation without a horizontal offset.

9.7 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 decreases any resulting environmental impacts and issues.

The top of the launch ramp is the upper-most part of the v-grooved concrete launch ramp and includes a portion of the vertical curve. The top of the launch ramp is a minimum of 1 foot above DHW. The top of the launch ramp above the waterline is cast-in-place concrete.

The toe of the launch ramp is the lower end of the v-grooved concrete launch ramp. The launch ramp toe extends below the DLW level to provide a hard surface for the trailer to travel on during launch and retrieval. The launch ramp toe is constructed of precast concrete panels below the waterline.

The Launch lane is 25 feet wide.

DESIGN WATER ELEVATIONS

General

The Ordinary Low Water (OLW) elevation was used as the basis for launch ramp toe design.

The Ordinary High Water (OHW) elevation, at minimum, was used for the top of launch ramp design.

Design Slope

Launch Ramp Slope

Preferred: 14%
Minimum: 12%
Maximum: 15%

Boarding Pier

Five-foot-wide Boarding piers (dock) are designed as means to help safely and efficiently launch and retrieve boats, and load and unload boaters at the launch facilities.

Design

Dead Load

Weight of construction materials

Live Load

Preferred: 20 lb/ft2

Minimum: 20 lb/ft2

Maximum: N/A

Freeboard (Dead load only)

Preferred: 12 inches

Minimum: 10 inches

Maximum: 15 inches

Pile Placement

Piles are generally placed internally unless the boarding piers have limited or no access to one side.

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, with the exception of the Safe House yard which is Risk Category IV.

- 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. The Safe House generator will store 500 gallons of diesel fuel.
- Maintenance and washing facility structures are subject to normal maintenance activities such as; minor equipment loads, tool cart impact and wash downs.
- Floor slabs in the maintenance building are to be designed for HS-20 traffic loads, and light or 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, uncoated. Structural steel deck and light gauge materials are to have standard minimum galvanized finish from the manufacturer, 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 recommendations.
- Metal stairs, metal stud curtain wall and window wall systems, pre-manufactured steel joists, 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 was 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.

Final design of metal buildings 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 = 130 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 appendix.

Metal Building System

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 new bridge crane building to determine preliminary column reactions. A foundation analysis review was performed for the existing 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
- Wind Speed = 151 MPH Risk Category IV (200 MPH Safe House)

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

990.00 lbs	Trolley Weight
4000.00 lbs	Crane Beams Weight
9,082.00 lbs	Max Wheel Load Static
10,000.00 lbs	Hook Load
11,352.00 lbs	Vertical + .25 Impact
2,200.00 lbs	Transverse + .2 Breaking / Running
33,640.00 lbs	Long Direction+ .1 Breaking / Running

Equipment Loads

TBD 30%

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 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 30 inches maximum at corner, edge, and ridge zones and at 5 foot 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 ½ inch on 12-inch 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 Design Documentation Report, 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.

For the Administration Building, the HVAC system designed is a variable refrigerant flow heat pump system with heat recovery. The high-efficiency system consists of a single 8-ton remote condenser connected to vertical air handling units and ceiling cassette type units. The vertical air handling units serve the Conference Room and Lobby area, are located in an equipment closet, and feature ducted supply, return, and outside air connections. The ceiling cassette units are provided to allow independent temperature zones to each office area. All restroom ventilation is provided via ceiling mounted exhaust fans.

The Operation & Maintenance Building is fully conditioned within the occupied space and ventilated within the Vehicle Bay/Garage. The HVAC system features a 7.5-ton heat pump system with supplemental electric heat. The vertical air handling unit is located in an equipment closet and features ducted supply, return, and outside air connections. The corridor ceiling cavity serves as a return air plenum to save cost over a fully ducted return air system. 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.

For the Safe House Building, the HVAC system consists of a 3-ton heat pump with supplemental electric heat. The vertical air handling unit is located in an equipment closet and features ducted supply, return, and outside air connections. All restroom ventilation is provided via ceiling mounted exhaust fans. Electric heat is provided for the Sprinkler Riser closet freeze protection. To mitigate heat build-up, a discharge wall louver and ducted plenum is designed for the generator radiator fan.

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 Operation & Maintenance

Building and the Safe House Building, the domestic water service is brought in at the Sprinkler Riser Room. At the Administration Building, the domestic water service is brought in near the mechanical 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.

Electric, high efficiency, storage tank, commercial grade, domestic water heaters are specified to serve each building. These have been selected to minimize the load on the electrical system. Local thermostatic mixing valves are provided to maintain 110-120oF 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 Operation & Maintenance Building, sediment traps are specified for all Work Area/Shop and Soils Lab sinks to prevent solids from entering the sanitary sewer system.

The majority of buildings do not require interior storm drain systems. However, the Vehicle Bay/Garage within the Operation & Maintenance 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 Operation & Maintenance 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 Operations & Maintenance Building and the Safe House Building. In order to supply the sprinkler systems, a 6-inch fire water service is brought into the Sprinkler Riser Room at each protected building. Double check backflow preventers and meters will be installed in the site service near the property line to protect the City's municipal water supply system. The water meters and backflow preventers 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 Operations & Maintenance 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 Diversion Gates

The tainter gate hoist machinery will consist of a 15 hp, 1150 rpm electric motor driving a worm gear reducer with a ratio of 30:1. The single worm gear (primary reduction) will then drive two separate parallel shaft reducer (secondary reduction) with a ratio of 70.6:1. The final drum reduction will have a ratio of 5:1 with a 17-inch pinion pitch diameter and the bull gear will have a 85-inch pitch diameter.

The drums will be of the stacking rope design with 6 ropes per side and with a diameter of 1 ¼" and a 6x37 Class with an IWRC, Stainless Steel 304. The drum has an allowance for 2 dead wraps, starting with an initial static drum diameter of 37.5 inches and after the two dead wraps, the working diameter is 42.5 inches when the gate starts to raise. The net drum (D) to wire rope (d) is 34 to 1.

The details of the general arrangement are as follows:

1. The motor is mounted directly to worm gear.
2. A worm gear is self-locking, thus the choice.
3. The motor will have a disc brake on the back side.
4. The worm gear will have two input shafts and two output shafts.
5. The outboard worm shaft will have a shoe brake rated to 150% of the motor rated torque.
6. The two outputs of the primary reduction will go to the secondary reduction on the control deck and the second output will go to the idler side via a supported shaft (three spherical bearings) outside of the walkway between the enclosures.

A digital 4-20-amp encoder with limit switches will be mounted to the drum shaft on the control deck. Back-up limit switches and their location are still under consideration. A rotary limit switch would be the simplest to install, but their long-term reliability, has proven to be questionable. The use of mechanical limit switches has proven to be sound but, given the gate travel and the configuration of the piers locating the limit just upstream of the trunnion for the fully open position appears to be a better choice. For the full closed back-up position we are considering a proximity switch just above the gate in the full closed position. This has shown to be problematic in icing condition but given the project location it should not be an issue. The lower limit switch would be normally open in the closed position and remain open during the gate travel. The upper limit switch would be normally closed and will open when the gate reaches the full gate travel.

<u>Alternative Designs</u>

Hydraulic cylinders were considered but given the complexity of the State-of-the-Art Design used at Folsom Dam Aux Spillway and other projects, further consideration did not continue.

11. ELECTRICAL, INSTRUMENTATION AND CONTROLS

For the purposes of this Design Documentation Report, the Electrical and Instrumentation / Controls designs are addressed together, since decisions regarding the Electrical Systems impact the Instrumentation / 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 Instrumentation / 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 Basis of Design Report.

11.2 Electrical Site Distribution

Electrical service (utility power) to the Reservation and Gate Structure will be coordinated with Entergy after the 30% submittal. However, the intent, as illustrated on the drawings, is to request that Entergy provide overhead service (13.8 kV or 25 kV, nominal) all the way through the Reservation and to the Gate Structure. Each building on the Reservation and the Gate Structure will have a dedicated electrical service and meter. Pole-mounted utility transformers will be located adjacent to each building on the Reservation and in close proximity to the Gate Structure. This approach provides significant cost savings as compared to a primary metered service or pad-mounted transformer(s) at the Reservation. Since service to the Reservation from Entergy will be overhead, maintaining overhead service throughout provides only minimal increased risk of an electrical outage.

Service voltage to each building was selected according to equipment requirements. Where possible, 208-volt, 3-phase service was selected to eliminate the need for transformation within the building.

Communication service to the Reservation will be coordinated with AT&T after the 30% submittal. However, the intent, as illustrated on drawings, is to request that AT&T provide underground fiber communication lines from Hwy 23, throughout the Reservation for service to each building. Standard Data / Communications service will be brought to the Administration Building and the Operations and Maintenance Building. Metro Ethernet service will be established at the Safe House, where the Master PLC will be located.

11.3 Lighting

LED lighting was selected throughout. LED fixtures have come down considerably in cost over the past few 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 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 Roadway.

Exterior Entry / Exit egress lighting for the Operation and Maintenance Building and the Administration Building will be powered by inverters. Downlights recessed in canopies were selected in lieu of wall packs to provide a more residential feel.

Where wall packs are provided, they 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.

For site safety and security, site lighting will be fed from the Safe House electrical panel so that it may be operational when utility power is lost.

Marine Grade, industrial light fixtures were selected for the Gate Structure. Decision to specify marine-grade fixtures was based solely on selecting fixtures that are appropriate for the use, considering limited access and close proximity to the river. Building-mounted fixtures at the Reservation (Administration Building and Operation and Maintenance Building) will not be marine-grade. The additional cost for marine-grade fixtures in these locations could not be justified.

Lighting calculations for the Gate Structure will be performed after the 30% submittal.

The need for obstruction lighting at the top of the Gate Structure is still being evaluated.

11.4 Power

Current panel schedules reflect known and/or anticipated electrical loads. Several loads are still unknown, particularly in the Operation and Maintenance Building. Current design illustrates dedicated 480Y/277-volt panels in the Shops area and Maintenance Garage for this reason.

Estimates of available short circuit current will be calculated later in the design. Voltage drop calculations and conduit fill calculations will be performed after the 30% submission.

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 motors will originate from motor control centers located within the machinery room.

11.5 Standby Generators

Standby power is currently illustrated for the Gate Structure and the Safe House. The standby generator dedicated for the Safe House will also serve the Sewer Lift Station / Treatment Plant and site lighting at the Reservation. The standby generator dedicated for the Gate Structure will also serve the access roadway lighting.

Currently, standby power is not included in the design for the Operation and Maintenance Building or the Administration Building. However, current design does include a service-entrance rated “storm switch” for each building, which will provide a means for connecting a portable generator set during extended outages.

Several factors contributed to the decision to include two generator sets in the design. The major factors are listed below.

Voltage: The Safe House does not require 480-volts. However, if the Safe House generator were utilized for the Gate Structure as well, a 480-volt generator would be required, and additional transformation within the Safe House would also be necessary, adding cost, and requiring additional space for transformation.

Distance: The Safe House is located at the Reservation, and the Gate Structure is roughly 800 feet away. Underground feeders would need to be grossly over-sized to limit voltage drop at the Gate Motors.

Minimum size requirements: Gate Structure hoist motors are currently estimated to be 30 HP. This was a conservative requirement that is being reduced as the gate machinery design progresses. The calculations in this version of the DDR reflect the 30HP demand. Generator sizing calculations confirmed that a minimum of 60 kW of available generator power would be necessary for starting and operating a single gate. This result did not include any other power requirements at the Gate Structure. Once the additional loads were included in the calculations, it was determined that an 80 kW generator set is necessary for the Gate Structure alone. Conversely, current calculations indicate that only 50 kW of generator capacity is required for the Safe House loads. Increasing the Safe House Generator from 50 kW to 150 kW (next standard size above 130 kW) would increase the footprint dramatically and cause the generator to operate at a very light load (less than 30% of rating) which can lead to generator wet-stacking and reduced generator performance.

Generator set controls for both generator sets will interface the PLC Controls for the Gate structure so that personnel can be notified of generator alarms (battery charger malfunction, low fuel, etc.).

Gate Structure Standby Generator:

Currently, 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. Consideration is still being given by the team to housing the generator within a separate structure.

Sub-base fuel tank capacity will be sized to provide minimum run time to operate each gate for at least a complete cycle (up and down). Per NFPA 110, that capacity will be multiplied by 1.33 to determine the minimum tank size required. By keeping fuel capacity to a minimum, a fuel polishing system for this generator should not be necessary, provided that the generator set is exercised weekly, under load.

Generator will start whenever utility power is lost, whether gates require operation or not. This sequence will insure that cameras, controls, and lighting are functional regardless of gate operation.

Safe House Standby Generator:

Generator will start whenever utility power is lost. This sequence will insure that cameras, controls, and lighting are functional regardless of gate operation.

Fuel tank capacity will be sized to provide minimum run time of 72 hours. Per NFPA 110, that capacity will be multiplied by 1.33 to determine the minimum tank size required. For the current basis-of-design generator, minimum fuel tank capacity is as follows:

Minimum Vol Req. = (fuel consumption) x (hours)x1.33
= 4.0 gal/hr. x 72 hrs. x 1.33 = 383 gallons

The sub-base fuel tank for the basis of design generator is 389 gallons, which meets the 72-hour requirement. Consideration continues to be given to specifying a separate 660-gallon tank (the largest allowed in the Safe House per NFPA) in lieu of the 389 gallon sub-base tank, thus providing, essentially, 7-days of run time at full load. However, a 3-day tank is being proposed at present, with the following considerations.

There is no standard, off-the-shelf, manufacturer-provided 660-gallon tank. Typical tank sizes jump from 500-gallons to 1000 gallons. A custom tank could be specified, but at an additional cost.

The 389-gallon tank mounts below the generator (sub-base type) and comes with the generator set packaged unit. Any tank larger than this capacity will require that a separate tank be provided. A separate tank would necessitate additional fuel piping (supply and return), and possibly a fuel pump. Piping would need to be double-wall for spill prevention.

Actual run-time provided by the 389-gallon tank is 4 days (if the tank is full; this includes the 133% required by NFPA 110), assuming that all of the electrical loads ran continuously for those 4 days, which will certainly not be the case. For example, the range will only be used when cooking; the water heater and AC will cycle on and off; the site lighting is only on at night.

To prevent fuel from spoiling, a fuel polishing system will be required, regardless of which tank size is selected. Without the polishing system, it is not anticipated that the fuel consumed during normal weekly testing will be enough to deplete the tank within the anticipated storage life.

11.6 Grounding and Lightning Protection

Details of the grounding system will be provided after the 30% submittal. Future designs will illustrate ground loops around buildings on the Reservation, a ground loop around the generator set for the Gate Structure, and ground rods on each side of the Gate Structure. 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.

Lightning Protection System specifications will be included after the 30% submittal.

11.7 Fire Alarm and Mass Notification Systems

The Operation and Maintenance Building, the Administration Building, and the Safe House will each 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 for each building (no speakers or voice paging), 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 a MNS does result in a significant project cost. A 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 Control System layout and details will be provided after the 30%. Input from End User is necessary if access control is required. Note that if system needs to remain operational for longer than 4 hours, additional generators may be required to insure power will be available to the equipment.

11.9 Gate Structure Power and Controls

Gate control wiring diagrams, PLC point list, and Sequence of Operation will be provided subsequent to the 30% submittal. The following narrative discusses decisions made up to this point regarding the Gate Structure Controls.

Gates 1 and 2 will be operated from electric motors located in Machinery Room No. 2. Gates 3 and 4 will be operated from electric motors located in Machinery Room No. 4. Machinery Room No. 1 will house the ATS and main power distribution equipment for the Gate Structure.

Diversion gates will be powered and locally controlled from Motor Control Centers located in Machinery Room Nos. 2 and 4. This approach provides a centralized location for operating a pair of gates while being able to observe the associated motors in operation.

Pan-Tilt-Zoom (PTZ) surveillance cameras will be mounted to the Gate Structure, positioned for viewing the gates and intake. From the Safe House, Administration Building, or any other PC with network access, required software, and password authorization, including remote PCs, it will be possible to call up camera images and control the pan, tilt, and zoom of each camera.

Gate controls will be a combination of PLC-based and hard-wired per the below.

> Gate control power will be 24-volts DC; power will be backed up by UPS and generator.
>
> Proximity sensors and limit switches will be used in conjunction to positively determine gate open or closed status.
>
> Should the PLC system fail, hard-wired controls within Machinery Room 2 will be able to open or close Gates 1 and 2 only. Similarly, hard-wired controls within Machinery Room 4 will be able to open or close Gates 3 and 4 only. Hard-wired controls will include safety interlocks, and a key-switch will be necessary to bypass the PLC and enable the hard-wired controls. Hard-wired controls will include pushbutton interfaces for gate open and close control, LED indicators for gate

open and closed status, control power available indication, gate power available indication, and an emergency stop switch.

PLC control equipment will be located in the Safe House and in the control sections of MCCs located in Machinery Room Nos 2 and 4. Communication between the master PLC in the Safe House and the Remote PLCs at the Gate Structure will be via fiber optic communication. An additional communication link from the Master PLC to the Administration Building will also be provided for direct connection of a dedicated PC.

A touchscreen will be provided in the controls section of each MCC located in Machinery Room Nos. 2 and 4. From the touchscreen, it shall be possible for an Operator to open or close any of the 4 gates. At this time, it is anticipated that a clear view of all 4 gates will be available from either Machinery Room No. 2 or 4. After evaluating final lines of sight, if it is determined that only two gates can be observed from Machinery Room 2 or 4, then PLC control within those Machinery Rooms will be limited to the gates associated with those Machinery Rooms.

A touchscreen will be provided in the Master PLC enclosure located in the Safe House. From the Safe House, it will be possible for an operator to monitor and control the Diversion Structure.

Via a hard-wired communication link, it will be possible for an authorized person to monitor and control the Gate Structure from a PC in the Administration Building. Authorization for control of the structure can be denied via programming.

Via the Metro Ethernet System, it will be possible for any authorized person with network access, required software, and password authorization to monitor and control the Gate Structure from a PC. Authorization for remote, off-site control of the structure can be denied via programming.

Proximity sensors and limit switches will be used in conjunction to positively determine gate open or closed status.

Prior to gate operation, an audible alarm will sound at the Gate Structure and at the Diversion channel outfall to warn of impending gate operation. Length of alarm prior to beginning gate operation is yet to be determined.

The Gate Controls PLC will also function as a SCADA System. The following will be monitored.

1. River level
2. Intake basin water level
3. Outfall basin water level
4. Gate Structure Generator Alarms
5. Gate structure ATS position and sources available
6. Safe House Generator Alarms
7. Safe House ATS position and sources available
8. Position of each gate

12. REAL ESTATE

CPRA will acquire both temporary and permanent rights-of-way for the construction and operation of the MBSD Project. Preliminary 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 4,000 acres, and the total estimate for temporary right-of-way is approximately 100 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 and design 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:

- a temporary circular cell cofferdam (Drawing XYZ)
- cofferdam deflector (Drawing 101)
- dewatering systems (Drawing 102 to be added for 60%)
- temporary levee system (Drawing 101)
- braced excavations for bridge piers (Drawing 103 to be added for 60%)
- braced cofferdam for inverted siphon installation (Drawing 104 to be added for 60%)
- excavation of the Conveyance Channel (Drawing 105 to be added for 60%)
- temporary protection across the Conveyance Channel when the backside levee is breached (Drawing 106 to be added for 60%)
- river trestle dock facility and fleeting area (Drawing 107 to be added for 60%).

Construction of the intake structure requires excavation to an EL -52, to permit the structure invert to be built to EL -40. Temporary structures to protect against river flooding from a high-water event on the Mississippi River consists of a two-fold approach: a steel sheet pile cellular cofferdam system in the river with a combination retaining structure on the river batture; and a temporary earthen levee to ring the landside of the excavation tying into the Mississippi River Levee (MRL). These temporary structures will provide continuous protection during construction activities in the excavation. The temporary levee will be a line of MRL Flood Protection for estimated 3 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.04**. The MR cellular cofferdam is a redundant line of riverine flood protection. The design criteria for the MR cellular cofferdam are presented in **Appendix J.03**.

Temporary river armoring will be placed against the outboard sheet of the coffer cells to prevent erosion and scour of the river bed soils around the coffer cells. The size of the stone will be determined using the velocities as determined by the DT using the physical model for the Headworks structure. Stone will be continuous from the cofferdam deflector to the downstream cell number 15.

The deflector system for the cofferdam will consist of cells 1, 2 and 3 with connecting panels. The deflector system will deflect the water velocities away from the cofferdam and serve as the point of protection for the cofferdam to prevent erosion of the river soils next to the cells. Testing of the deflector will be accomplished at a future date in the physical model with results provided by the DT.

An Emergency Evacuation Plan will be assembled at a future date after the Hurricane Evacuation Plan is developed. The evacuation plan will be developed during the 60% Design Phase and it will consider the time it will take to rewater the Headworks excavation and provide for securing the site and safely removing personnel from the project site.

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 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 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.

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 will be evaluated for river flows utilizing the DT physical hydraulic model that is being used for modeling of the inlet structure. Results of the physical model testing will be completed during the 60% Design Phase.

Two cell placement crews will be used to place templates, set sheets and drive sheets at two independent locations. Crew one will start at cell 1 and crew two will start at cell 10. Additional pile driving crews will be used to set connecting arc templates, place and drive the sheets. Filling of the cells and arcs will be accomplished by a separate operation consisting of clamming cranes, loaders, push boats and flat 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

Sequence of cell placement begins with cell 1, and progresses sequentially to cells 2, 3, and 4. The Z-pile structural deflector wall will be installed at an upstream off-set from cell 1 to cell 2. At the same time cell 1 is being placed a separate crew will begin placement of the upstream Z-pile tie in walls that will connect the MRL to cell 3, once cell 3 is completed. This will prevent any potential for erosion of the levee structure. As placement of the Z-pile tie in wall is in progress, cell 4 will be installed. Once the Z-pile tie in wall to the MRL is in place and cell 4 is complete, the structural deflector wall from cell 2 to cell 4 will be installed. Cells 5, 6, 7, 8 and 9 will follow in sequence with connecting arcs installed as work area permits.

At the onset of placing cell 1 the second cell setting crew, crew two, will begin placement of cell 10 and proceed downstream to cell 13. The placement of the cells 15 to 19 will proceed toward the MRL making closure with the downstream Z-pile tie in system from cell 19 to the MRL. 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. Additionally, the placement of the inside stability berm will be completed where required. The Z-pile wall tie in structures will also be filled with sand and will be capped with the same size stone as the cells. In addition, the surfaces will be prepared for equipment traffic by placing a topping of smaller road stone.

13.2.4 Cell Fill

Sand for cell fill will be obtained from the designated borrow source in the river. Classification of the sand will be an SP to provide minimal permeability of the fill but allow drainage of the cell fill for dewatering purposes. Cell fill will be obtained from the borrow area using a floating plant and crane equipped with clamming capability. Sand will be placed on flat deck barges and transported to the cells being filled.

13.2.5 Upstream Deflector

The purpose of the deflector is to divert the river current away from the riverside leg of the cofferdam to prevent scour near the cofferdam. Only the angled portion of the deflector is needed to accomplish the diversion. In addition, the deflector will provide a sheltered area to aid in the construction of the river arm of the cofferdam. The deflector walls will be designed as steel sheet pile walls supported by steel frames on steel pipe pile foundations. The steel frames will be constructed in advance; then used as templates for driving the foundation support pile. Drawings for the deflector wall structure will be provided at the 60% design stage.

The deflector walls will be designed during the 60% design phase using a swell head determined from the physical model studies or a value determined by hydraulic analysis. The swell head will be combined with the velocity head for determining the loading on the wall. The 60% design phase will present the analysis and design.

Current river bottom elevation will be used for determining length of sheet pile for proper embedment to protect against potential scour. The frame support piles will be driven to an elevation to be determined in the 60% design phase that will provide axial support as well as lateral support of the frame and sheet pile wall. The deflector panels that will be connected to cells 1, 2 and 4 will be constructed of a structural frame made up of I-beam sections attached to vertical pipe acting as sleeves for pipe pile to be placed through and driven into the river bottom. Once in place, the frame will be faced with Z piling. Stone protection will be placed at the deflector and along the river arm of the cofferdam to prevent scour next to the base of the cells. The scour protection will consist of rock sized according to the velocities obtained from the DT physical model test for the cofferdam system. A stone berm section outside the cells will be evaluated for a section with dimensions of 20 feet wide at the crest, 10 feet in thickness and will be 45 feet wide at the base with a face slope of 1V:2.5H.

The cell diameter is currently set at 61.68 feet for initial design, which will be confirmed in the 60% design phase. 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, there will not be cranes located on adjacent cells or arcs that have the potential to increase interlock stresses.

13.2.7 Water Loading in the Cell

The water level inside the cell has been set at 16.65-foot for the outboard side of the cell, with a slope of 2H:1V to the inboard side of the cell. This water profile is the maximum water loading used for design and determination of the interlock stresses. Dewatered elevation of the interior excavation of the cofferdam is set at 5 feet to 10 feet below excavation grade line inside the cofferdam as the excavation progresses to the required depth of -50 feet. During the 60% Design Phase cell fill, and subsurface soils will be evaluated for drainability when responding to the draw down from the dewatering system. As such, minimum provisions such as block out pipe for installation of dewatering wells will be placed in each cell on the inboard side of the cell as required to lower the phreatic surface.

13.2.8 Interior Stability Berm

The interior cofferdam stability berm was designed using procedures and guidance contained in EM 1110-2-1902, Slope Stability. Detailed analysis was performed using SLOPE/W, GeoStudio 2018 R2. The interior cofferdam stability berm fill will consist of SP or SM sand depending on the borrow source location for the cell fill. The height of the berm will be at -10 with a berm width of 10 ft at the top and with slopes set at 1V:4H. The layout of the cellular cofferdam shown on Drawing 101 will accommodate the required slopes. The cross sections of the cofferdam slopes are shown on Drawings 101A thru 101K. Erosion of the slopes during rewatering will be prevented by use of pipes extending to the base of the excavation that will be connected to the rewatering pumps. 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 to a predetermined elevation to be established during the 60 % Design Phase. The floodway will consist of 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 berm grade lines. The interior of the floodway will be lined with geotextile and then filled to the top of the sheet pile, with the interior filled with 400-pound rip rap stone.

13.2.9 Subsurface Materials

Subsurface information is being 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 are currently being determined and the results will be provided in the 60% Design Phase in **Appendix J**.

13.2.10 Impact Loading to the Cofferdam Cells

The deflector arm and river arm cells and arcs will be protected from vessel impact by energy dissipating devices shown on Drawings TBD and in **Figure 13.2-1**. Protection of the river cells will be accomplished using berthing features that are commonly used for commercial dock facilities. The energy absorbing features will consist of rubber pneumatic energy absorbers that will absorb the impact energy from a standard size barge with a capacity of 2272 tons. The fender 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. The energy absorbing design and details will be included during the 60% Design Phase.

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 vessel size used in the design was an inland rivers standard hopper barge fully loaded with a gross tonnage of 2272 Tons. 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 60% Design Phase following evaluation of the 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.

The downstream arm of the cofferdam will be protected by mooring piles. These piles will prevent barges from rubbing the interlocks of the sheet pile during unloading of materials. The piles will be designed for number and size during the 60% Design Phase. The downstream arm of the cofferdam will be utilized to moor barges for offloading materials such as foundation piles to be placed inside the cofferdam. The mooring piles will be designed using the procedures contained in the Handbook on Design of Piles and Drilled Shafts Under Lateral Loads, FHWA-IP-84-111. The mooring pile will be designed for allowable loads and deflections using fully loaded hopper or flat top barges.

13.2.11 MRL Tie-In

Tie-in to the MRL will be accomplished using two parallel single Z-pile walls separated 30 feet. The two walls will be tied together with internal tension members such as DYWIDAG high tension rods attached to walers placed on the outside of the sheet pile walls. Drawing 109 depicting this system will be added during the 60% Design Phase. The area between the walls will be filled with sand SP, with 5% or less fines. The termination of the Z pile wall will occur in the MRL embankment. Soil conditions will be evaluated for stability, settlement and through seepage in the 60% Design Phase. Surcharge loading will be included in the analysis based on the equipment weight being used in the surcharge analysis of the coffer cells.

13.2.12 Cofferdam Removal

Removal of the cofferdam cells, arcs and MRL tie-in walls will begin with the removal of the tie-in sheet pile walls at the downstream end of the cofferdam. The cells will be removed beginning at cell 19 and proceed up to cell 10. Removal of cell 9 will begin at the same time cell 19 is being removed and proceed up to connecting arc 3A. Cells 2, 1 and the deflector walls will be removed followed by removal of cell 3 and the upstream sheet pile tie in walls. Sheet pile will be cut off at 2 feet below the finish grade line and back fill will be placed for the levee embankment and for stone armoring. Divers will be utilized to cut the sheets for removal. Final design grading and placement of stone protection will follow the removal of the cells and connecting arcs.

13.2.13 Cofferdam Instrumentation and Monitoring

Instrumentation of individual cells of the cofferdam will consist of survey monitoring points and inclinometers. Location Drawing 110 will be added during the 60% Design Phase. The survey points will be located on the inboard and outboard of cells 3, 5, 9, 13, 15 and 19. The survey markers will be read daily to determine any movement. Red-line deflections will be determined during the 60% Design Phase.

In addition, inclinometers will be installed on cells 3, 5, 9, 13, 15 and 19 to correlate with the top of cell surveys. 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. River surveys along the cofferdam will be performed monthly to detect any developing scour close to the coffer cells.

13.2.14 Cofferdam Flood Gates

Two flood gates approximately 15 feet wide and with a crest at elevation of 12.0 feet will be provided for the cofferdam in the event river stages higher than 16.65 occur during construction of the inlet structure. The flood ways will consist of a concrete paved sluiceway, steel beams, timber needles and a riprap protected spillway. Associated drawing 111 will be provided during the 60% Design Phase. Erosion on the sides of the spillway will be controlled by use of sheet piling. The flood gates will be located between cells 13 and 15 and 15 and 16 on the downstream arm of the cofferdam. The gates will be designed to enable filling of the cofferdam within a yet to be determined time period. The design of the flood gates will be provided in the 60% Design Phase.

13.2.15 Dewatering System and Under Seepage Control

A dewatering system, consisting of deep pumped wells and shallow well points will be used to draw down the water table to maintain a dry excavation bottom. The dewatering system will consist of high capacity deep wells estimated at 200 GPM and low capacity wells with a capacity of 50 GPM. Permeability values for the course point bar material and SM/CL material will be determined by doing deep well pump tests in predetermined locations. Borings have been obtained to determine the stratigraphy delineating the course materials from the fine materials. The intent of the dewatering test is to evaluate the drainability of the upper SM/CL materials as the lower course material are being pumped. The results of the borings and pump test will be provided in the 60%Design Phase documents.

The system will be designed with the high capacity wells located around the perimeter of cofferdam transitioning into the low capacity wells at the MRL levee and proceeding around the perimeter of the excavation for the inlet structure. Two to three levels of eductor wells will be installed as the excavation is carried to the -52 elevation. The high capacity wells and educator system will be powered through an electrical distribution system with a generator back up system to maintain continuous pumping capability. The abandonment of the wells will be in accordance with local, state and federal guidelines. Associated drawing 112 and details of the system will be provided in the 60% Design Phase.

13.2.16 Construction Activities Inside the Cofferdam

The cofferdam will be designed to permit pile driving operations inside the cofferdam at all river elevations up to elevation 15.0 feet on the Carrolton gage. The cofferdam will also be designed for installation of the dewatering system up to a river elevation of 13 feet. The maximum river elevation at the project site is currently 11.82. As such, any elevation greater than the current record is assumed to be brought about by a surge from a tropical storm. Thus, the dewatering system will be inoperable since the Headworks excavation will be rewatered to an elevation to be determined in the 60% Design Phase.

13.3 Overview of Sequence of MBSD Project Construction

The sequence of construction may best be explained by referencing the attached Linear Schedule in conjunction with this narrative.

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 following explains the work sequence as Landside Work and Riverside/Marine Work.

Landside Work:

Construction of the earthen interim levee on the east side of Highway 23 (Hwy 23) will begin immediately using suitable on-site material that is processed, placed and compacted in controlled lifts. Relocation of the NOGC Railroad track must be completed as the interim levee is constructed in order to tie the interim into the existing MRL. Once the interim levee has been brought to EL16 and is authorized, the first stage of the dewatering system will be installed as excavation of the Intake 'bowl' proceeds to EL-52. Excavation of the bowl is restricted by USACE rules regarding river level per Carrollton Gage in New Orleans.

Excavation of the bowl will be concurrent with the installation of the staged dewatering system. As the bowl is excavated to the final elevation of EL-52, the work area will be protected with a rock material course to provide a stable work area for pile driving.

The Intake U-Wall structure will then be constructed beginning with support foundation piles, followed by foundation footings and intake walls. Gate structure, Intake training walls, transition walls, and levee flood walls will proceed in the same sequence.

Once the Intake Structural elements have been completed, the bowl area will be backfilled with controlled lifts to final grade.

On the west side of Hwy 23, the wick drain system, beginning with the sand drainage-blanket, will begin as soon as the initial clearing is complete. As the wick drain blanket and wick drains are installed the excavation of the proposed conveyance channel will begin with excavated material processed and placed on the drainage blanket to begin the surcharging/consolidation period for the guide levees. Successive layers of clay levee material will be placed up to the final elevation for the guide levees with appropriate consolidation periods allowed between lifts.
Siphon construction will begin with braced excavation for the pipe installation and soil improvement at the intake and outfall structures in the early stages of the project.

Once the guide levees and flood protection T-walls along the length of the conveyance channel have been brought to grade, the proposed NOV-5a Levee will be removed at the location of crossing with the MBSD channel. This will ensure that protection from back flooding will be maintained at all times. 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 installed 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 deflector system, 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 Interim earthen levee is tied into the existing MRL, the contained area between the cofferdam an existing MRL will be dewatered as described in Section 13.2.

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 construction of permanent piling and concrete features.

The Interim Levee will remain in-place until the final concrete structural features are in place, affording full permanent-grade protection to Plaquemines Parish and Hwy 23.

13.3.1 Headworks Construction & Interim Earthen Levee Sequencing Description

The following paragraphs describe the sequence of landside construction in the Headworks area, including the Interim Earthen Levee (IEL). This write-up is accompanied by a set of drawings depicting each phase.

Phase 1 of the Headworks Area (Land):

The construction of the Interim Earthen Levee (IEL), which will serve as the main line of protection during construction of the MBSD Headworks, is the key element of Phase 1. The interim levee must be in place and fully functional as the main line Mississippi River flood protection before the MRL can be breached to complete 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 interim levee. The interim levee will be built up over a 9-month period and split into approximately three equal lifts in order to achieve the target build elevation at the end of construction. The lifts will be built up with successive courses (12 inches loose/8 inches compacted) to specified Proctor soil density.

All interim levee lifts will be built from excavated and tested on-site material. The first lift of the interim levee will comprise the stability berm for the earthen levee. The successive lifts will form the sloped levee section. All material used will be processed and density-controlled as per USACE standards (I will contact Mark Woodward and identify the specific spec). Levee construction will be accomplished using trucked-in material, dozers, compactors, motor graders, etc. Included in the levee construction will be instrumentation (TBD) to measure settlement and stability of the completed levee. It is projected to take 9-months to achieve the final levee grade, including the base-course levee road.

Phase 1 concludes with the inspection, and certification by the USACE, of the IEL. 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 IEL 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 in order to begin construction of the Intake Structures. The completed excavation slopes of the ‘bowl’. Dewatering of the bowl area will be in three stages and occur concurrently with the excavation. Final excavation elevation for the bowl will be EL-52.

Construction duration for the Intakes Structures is projected to take approximately 32 months, during which time the dewatering system will be operational.

As the Headworks Structure is completed, the structure and bowl will be back-filled and dewatering system withdrawn.

Dirt processing and compaction equipment used for the back-filling operation will typically consist of dozers, pad-foot compactors, etc.

Phase 3 of Headworks Construction

Phase 3 consist of removal of the IEL once the Intake Structure is complete and approved. The final structural elements of the Intake Structure will provide permanent protection for the Plaquemines Parish at that time.

13.3.2 Interim Earthen Levee (IEL)

Project design criteria requires that an IEL be the main line of interim flood protection for Plaquemines Parish to guard against river flooding and, therefore, must be designed and constructed to the USACE's standards for the existing MRL. The design section for the MRL consists of a 10-ft wide levee crown with 1:3 floodside slope and 1:4 landside slope.

It is anticipated that the construction duration for the intake structure will be approximately three (3) years. Therefore, for the duration of construction, the elevation of the interim levee crown must 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).

A dewatering system, consisting of deep pumped wells and shallow well-points will be used to draw down the water table to maintain a dry excavation bottom. **Table 13.3-1** shows the proximity of the excavation and interim levee. Due to anticipated settlement, the landside slope will be constructed to 1:3.5 and settle down to 1:4. Likewise, based on the results of the analyses, and assuming a minimum 10-foot wide crown, 1V:3.5H side slopes on the Land-Side and 1V:3H side slopes on the Flood-side, it is determined that the interim levee should be built up to EL 18 feet., NAVD88 in order to stay above project grade for the duration of construction. The interim levee elevation will be monitored during construction (using what time of monitors or monitoring regime), and material and equipment will be readily available to add fill back to design grades, as needed.

Table 13.3-1: Proximity of Excavation and Interim Levee

Soil Reach	Ground Elevation (ft., NAVD88)	Target Crown Elevation at End of Construction (ft., NAVD88)	Crown Elevation At year 4 (ft., NAVD88)
Reach 3	+4	+18	+16.9
Reach 4	+3	+18	+16.7
Reach 5	+3	+18	+16.6

Based on preliminary analysis calculations, settlements by the end of construction are expected to range between 12 and 20 inches. This anticipated settlement will be taken into account when determining the required volume of placed material. Total long-term settlements (immediate, primary, and secondary) are expected to range between 33 and 48 inches. It is noted that these values do not account for subsidence. The calculation package for these analyses are included in **Appendix B**.

Although the computed consolidation settlements indicate that the crown of the interim levee will remain above the required project grade for the duration of construction, there is a possibility that some areas along the alignment may settle more than predicted. Monitoring of settlement will be maintained, and threshold triggers will be established for providing additional lifts of fill on the interim levee. Should the assumed project schedule differ significantly from that which was analyzed, additional analyses may be required to refine settlement estimates. Iterative analyses were performed to evaluate the crown elevation at various years past the end of interim levee construction in order to confirm that the interim levees will remain above the design grade for at least three (3) years past end of construction. This target build elevation was incorporated into the slope stability analyses, as is deemed the worst case since the subsoil have not had a chance to gain any strength due to consolidation settlements, and as it imposes the greatest loading.

13.3.3 MRL Levee Crossing & Levee Access for Plaquemines Parish Government (PPG) During Construction

Crossing of the MRL will be limited to the area where the interim levee 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 inlet 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.

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 and police patrols during construction. Since construction of the channel is going to disrupt current access along the current MRL crown, the interim earthen levee crown will connect to the MRL and include continuation of the crown road allowing access around construction.

13.3.4 Armoring

Armoring of the interim levee, cofferdam and tie-in structures will follow guidelines as presented in the design documents listed in Section 6.6 Armoring. The required armoring will be established based on velocities determined from model studies related to these features of the project. Once velocities are determined the size and thickness of the armoring will be established.

13.4 Excavation and Installation of the Sheet Pile Retaining Structure for Siphon Under the Channel

The excavation for installation of the siphon pipe(s) under the channel will be accomplished by open cut excavation to EL -25 with excavation slopes being evaluated for a FS of 1.2 with slopes anticipated to be 1V to 6H. The width of the excavation at EL -25 will accommodate the required width for installation of the siphon pipe plus 40 feet on each side to accommodate equipment access. A sheet pile retaining structure will be installed from EL -25 to the required depth below EL -38 which is the maximum depth of the excavation. The sheet pile retaining wall will be 15 feet in height and will be designed as a braced sheet pile wall with one level of bracing to minimize impacts to placement of the concrete pipes. A dewatering system will be installed to control ground water in the excavation.

13.5 Cofferdam Structures for Highway 23 Bridge Pier Installation

The excavation for installation of the bridge piers will be accomplished with a braced sheet pile cofferdam structure that will be installed to provide driving of the foundation piling to a tip elevation of -130. The interior of the cofferdam will be excavated to an elevation of -25 prior to driving the piles. Piling will be removed or left in place and cut off 3 feet below grade line. A dewatering system will be installed to control ground water in the excavation as needed.

13.6 Temporary MR Trestle Dock

The trestle will be located downstream of the cofferdam and will provide access for offloading of materials. The temporary trestle foundation design will follow standard procedures for design of pile foundations as presented in USACE, EM 1110-2-2906, Design of Pile Foundations. Soil-pile properties will be determined during the design process of the trestle. Loads to be supported will be 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 will be applied to determine individual pile capacity for the structure.

13.7 Maintenance of Back Hurricane Protection

The planned NOV-5a 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-5a Levee at the points where the two levees intersect along both sides of the Conveyance Channel. The NOV-5a Levee will then be removed in between the two Guide Levees once they are brought to design grade and the T-walls at Hwy 23 are complete.

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.