APPENDIX O: BIOLOGICAL ASSESSMENT & BIOLOGICAL OPINION

O1: Biological Assessment

O2: Biological Assessment Correspondence

(to be provided in the FEIS)

O3: USFWS Biological Opinion

(to be provided in the FEIS)

O4: NMFS Biological Opinion

(to be provided in the FEIS)

O1: Biological Assessment

Mid-Barataria Sediment Diversion
BIOLOGICAL ASSESSMENT

Prepared for:

Louisiana Trustee Implementation Group and the Coastal Protection and Restoration Authority

January 2021

146 N Canal St, Suite 111 • Seattle, WA 98103 • www.confenv.com

Mid-Barataria Sediment Diversion
BIOLOGICAL ASSESSMENT

Prepared for:

Louisiana Trustee Implementation Group and the
Coastal Protection and Restoration Authority
Baton Rouge, Louisiana

Authored by:

Confluence Environmental Company
Seattle, Washington

January 2021

This report should be cited as:

LA TIG (Louisiana Trustee Implementation Group). 2021. Mid Barataria Sediment Diversion: Biological Assessment. Prepared for CPRA and LA TIG, Baton Rouge, Louisiana, by Confluence Environmental Company, Seattle, Washington.

EXECUTIVE SUMMARY

The Coastal Protection and Restoration Authority of Louisiana is proposing to construct, operate, and maintain the proposed Mid-Barataria Sediment Diversion Project. The proposed Project consists of a multi-component river diversion system intended to convey sediment, fresh water, and nutrients from the Mississippi River at approximate River Mile (RM) 60.7 in the vicinity of the town of Ironton, Plaquemines Parish, Louisiana to the mid-Barataria Basin. After passing through a proposed intake structure complex at the confluence of the Mississippi River and proposed intake channel, the sediment-laden water would be transported through a conveyance channel to an outfall area in the mid-Barataria Basin located in Plaquemines and Jefferson Parishes.

Following construction of the Diversion Project it would be operated for 50-years based on the flows measured at the Mississippi River gage at Belle Chase. When Mississippi River flows exceed 450,000 cubic feet per second (cuffs), flows through the diversion will increase from a base flow target of 5,000 to a maximum of 75,000 cfs. The maximum diversion flow will occur when the Mississippi River at the Belle Chase reaches 1,000,000 cfs.

The Louisiana Trustee Implementation Group has evaluated the Mid-Barataria Sediment Diversion Project to determine how the proposed action will affect any threatened or endangered species or designated critical habitat potentially occurring in the action area defined for this biological assessment (BA). This BA summarizes the available information on the potential effects of the project on ESA-listed species and critical habitat within the action area. This BA addresses 10 ESA-listed species: pallid sturgeon (*Scaphirhynchus albus*), Eastern black rail (*Laterallus jamaicensis jamaicensis*), piping plover (*Charadrius melodus*), Rufus red knot (*Calidris canutus rufa*), West Indian manatee (*Trichechus manatus*), green sea turtle (*Chelonia mydas*), hawksbill sea turtle (*Eretmochelys imbricata*), Kemp's ridley sea turtle (*Lepidochelys kempii*), leatherback sea turtle (*Lepidochelys kempii*), and loggerhead sea turtle (*Caretta caretta*).

Potential effects of the project construction and operations on ESA-listed species and designated critical habitat include construction effects associated with disturbance from underwater noise associated with piling installation and habitat effects during dredging and sediment placement. During operations the project may entrain fish, nutrients, sediment and water from the Mississippi River into Barataria Basin, cause changes in water temperature and salinity in Barataria Basin and cause indirect effects to wetland, fish and invertebrate populations and habitat quality in Barataria Basin. The project will implement minimization measures to reduce effects on listed species and designated critical habitat. Table ES-1 summarizes the listed species and proposed and designated critical habitat addressed in this BA and the effect determinations.

Table ES-1. ESA Species Effect Determinations

Listed Species	Status	Effects Determination	
		Species	Critical Habitat
ESA Listed Fish			
Pallid Sturgeon (*Scaphirhynchus albus*)	E	LAA	NA
Eastern Black Rail (*Laterallus jamaicensis jamaicensis*)	PT	NLAA	NA
Piping plover (*Charadrius melodus*) - Atlantic Coast, Great Lakes, and Northern Great Plains population	T	NLAA	NE
Red knot (*Calidris canutus rufa*)	T	NLAA	NA
West Indian manatee (*Trichechus manatus*)	T	NLAA	NA
Green sea turtle (*Chelonia mydas*) - North Atlantic DPS - South Atlantic DPS	T	LAA	NA
Hawksbill sea turtle (*Eretmochelys imbricata*)	E	NLAA	NA
Kemp's ridley sea turtle (*Lepidochelys kempii*)	E	LAA	NA
Leatherback sea turtle (*Dermochelys coriacea*)	E	NLAA	NA
Loggerhead sea turtle (*Caretta caretta*), - Northwest Atlantic DPS	T	LAA	NE
Sources: NMFS 2018a, USFWS 2018a Abbreviations: DPS = Distinct Population Segment; E = Endangered; T = Threatened; PT = Proposed Threatened NA = not applicable; LAA = likely to adversely affect; NLAA = may affect, not likely to adversely affect; NE = no effect			

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

ACRONYMS AND ABBREVIATIONS

°C	degrees Celsius
°F	degrees Fahrenheit
µg/L	micrograms per liter
µPa	micropascal
AHP	above Head of Passes
BA	biological assessment
BiOps	Biological Opinions
BMP	best management practice
BU	beneficial use
BUDMAT	beneficial use of dredged material
CEC	Confluence Environmental Company
CEMVN	U.S. Army Corps of Engineers New Orleans District
CFR	Code of Federal Regulations
cfs	cubic feet per second
Chl A	chlorophyll A
COC	contaminants of concern
CO-OPS	Center for Operation Oceanographic Products and Services
CPRA	Coastal Protection and Restoration Authority of Louisiana
CRMS	Coastwide Reference Monitoring System
CWPPRA	Coastal Wetlands Planning, Protection, and Restoration Act
cy	cubic yard
CZMA	Coastal Zone Management Act
dBA	decibels, A-weighted
dBpeak	decibels, peak
dBrms	decibels root mean square
DDT	dichlorodiphenyltrichloroethane
DO	dissolved oxygen
DOI	U.S. Department of the Interior
DPS	distinct population segment
DWH	Deepwater Horizon
DWH PDARP	The Deepwater Horizon Oil Spill Final Programmatic Damage Assessment and Restoration Plan
EFH	essential fish habitat
EIS	Environmental Impact Statement
EPP	Environmental Protection Plan
ERL	Effects Range Low
ERM	Effects Range Median
ESA	Endangered Species Act
FHA	Federal Highway Administration
FP	fibropapillomatosis disease
FR	Federal Register
FWOP	future without Project

FWP	future with Project
GEC	Gulf Engineers & Consultants, Inc.
GIWW	Gulf Intracoastal Waterway
HUC	Hydrologic Unit Code
Hz	hertz
kHz	kilohertz
LA	Louisiana
LA TIG	Louisiana Trustee Implementation Group
LADOTD	Louisiana Department of Transportation and Development
LCA	Louisiana Coastal Area
LDEQ	Louisiana Department of Environmental Quality
Ldn	average equivalent sound level over a 24-hour period
LDNR	Louisiana Department of Natural Resources
LDWF	Louisiana Department of Wildlife and Fisheries
LMR	Lower Mississippi River
MAM	monitoring and adaptive management
MAMP	Monitoring and Adaptive Management Plan
MBSD	Mid-Barataria Sediment Diversion
mg/L	milligrams per liter
MHHW	mean high high water
MR&T	Mississippi River and Tributaries
MRL	Mississippi River Levee
MSA	Magnuson-Stevens Fisheries Conservation and Management Act
NAVD88	North American Vertical Datum 1988
NEPA	National Environmental Policy Act
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NOGC	New Orleans Gulf Coast Railway
NOV-NFL	New Orleans to Venice Non-Federal Levees
NPDES	National Pollutant Discharge Elimination System
NRDA	Natural Resource Damage Assessment
NTU	nephelometric turbidity unit
OCM	Office of Coastal Management
OCS	Office of Coast Survey, NOAA
ODMDS	Ocean Dredged Material Disposal Site
OPA	Oil Pollution Act
OTF	outfall transition features
PAHs	polycyclic aromatic hydrocarbons
PBFs	physical or biological features
PCBs	polychlorinated biphenyls
PCEs	primary constituent elements
Phase I RP	Final Strategic Restoration Plan and Environmental Assessment #3: Restoration of Wetlands, Coastal, and Nearshore Habitats in the Barataria Basin, Louisiana
ppt	parts per thousand
psi	pounds per square inch

psu	practical salinity unit
PVC	polyvinyl chloride
RIA	Regional Implementation Agreement
RM	river mile
SAV	submerged aquatic vegetation
SEL	sound exposure level
Services	U.S. Fish and Wildlife Service and National Marine Fisheries Service
SLR	sea level rise
SPCC	spill prevention, control, and countermeasures
SPL	sound pressure levels
SR	State Route
SWPPP	stormwater pollution prevention plan
TDA	threshold discharge area
TEDs	turtle excluder devices
TEL	Threshold Effect Level
TESC	temporary erosion and sediment control
TOC	total organic carbon
TSS	total suspended solids
TWI	The Water Institute
USACE	U.S. Army Corps of Engineers
USC	United States Code
USCG	U.S. Coast Guard
USDA	U.S. Department of Agriculture
USDOT	U.S. Department of Transportation
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service
WQC	Water Quality Criteria
WQS	Water Quality Standards

1.0 BACKGROUND AND HISTORY

The Coastal Protection and Restoration Authority of Louisiana (CPRA) is proposing to construct, operate, and maintain the proposed Mid-Barataria Sediment Diversion (MBSD) Project. This Project is proposed to maintain and rebuild eroding upland, and freshwater and coastal marsh habitat within the Barataria Basin. The Project is also intended to restore injuries to natural resources caused by the Deepwater Horizon oil spill. The Deepwater Horizon Oil Spill Final Programmatic Damage Assessment and Restoration Plan (DWH PDARP) was developed collaboratively under the Oil Pollution Act (OPA) by federal and Gulf Coast state natural resource trustee agencies (DHNRDAT 2016). The DWH PDARP includes a suite of coastal restoration objectives, including the use of sediment diversions to help maintain and rebuild coastal habitats. Furthermore, the Louisiana Trustee Implementation Group (LA TIG) published the Final Strategic Restoration Plan and Environmental Assessment #3: Restoration of Wetlands, Coastal, and Nearshore Habitats in the Barataria Basin, Louisiana (Phase I RP), consistent with OPA and the DWH PDARP. The proposed Project is a preferred alternative for restoring DWH Oil Spill injuries through restoration in the Barataria Basin. This Project is being evaluated for funding under the DWH PDARP restoration planning process by the LA TIG, who will make the final funding decision.

The LA TIG funding decision and the U.S. Army Corps of Engineers (USACE) permitting review process are collectively referred to as the proposed MBSD Project, proposed Project, or Project for the purpose of this this Biological Assessment (BA). The Project constitutes a major federal action with the potential to significantly affect the quality of the human environment. The Project is, therefore, being evaluated under the National Environmental Policy Act (NEPA) through a detailed, interdisciplinary Environmental Impact Statement (EIS) supporting both the USACE and LA TIG decision processes. The Project is also being evaluated under the OPA and DWH PDARP through the development of the Phase II Restoration Plan (RP). This BA analyzes the Project's potential effects to federally listed threatened and endangered species (see Appendix A). The EIS and RP contain additional details and background on the Project description, Project history, and direct, indirect and cumulative environmental impacts. The information presented here is consistent with the EIS, RP, and supporting reports provided by CPRA. Where appropriate, this document will refer to sections of the EIS or RP for additional information and incorporate that information by reference.

USACE and the LA TIG will use this BA to support individual requests to initiate formal Section 7 consultation under the federal Endangered Species Act (ESA) with the U.S. Fish and Wildlife Services (USFWS) and National Oceanic and Atmospheric Administration (NOAA) Fisheries (National Marine Fisheries Service [NMFS]). It contains the information required under 50 CFR 402.12 and 50 CFR 402.14(c) for conducting a BA and initiating formal consultation, respectively. We anticipate that the USFWS and NMFS will each conduct formal consultations and will issue separate Biological Opinions (BiOps) for the species under their respective jurisdictions, with each BiOp addressing specific USACE and LA TIG actions as appropriate. This will also serve to inform consultation under the Magnuson-Stevens Fisheries Conservation and Management Act (MSA), Section 305(b)(2), for assessment of effects to Essential Fish Habitat (EFH) for both agency actions. This BA therefore contains the information necessary to satisfy the requirements of 50 CFR 600.920(e)(1). USACE and the LA TIG anticipate that NMFS will consolidate the EFH consultations for both actions and issue a single set of conservation recommendations, if necessary.

1.1 Project Background

Sediment diversion projects have been included as a critical component of the state's Coastal Master Plans since 2007 (CPRA 2007, CRPA 2012, CRPA 2017a). Previous studies examining sediment diversions from the Mississippi River, which informed the development of the proposed Project, can be found in EIS Section 1.2.2.1. Louisiana's 2017 Master Plan objectives applicable to the proposed Project include harnessing the natural processes that built Louisiana's coastal landscape, sustaining Louisiana's unique cultural heritage, and ensuring that Louisiana's coast continues to be both a sportsman's paradise and a hub for commerce and industry (CPRA 2017a). The DWH PDARP also features sediment diversion projects as the primary approach to restore and preserve Mississippi-Atchafalaya River processes; these diversion projects are intended to increase the long-term resilience and sustainability of deltaic wetlands by reducing widespread loss of existing wetland area (DHNRDAT 2016).

1.2 Barataria Basin and Birdfoot Delta History

The Barataria Basin was formed over 1,000 years ago as part of the Lafourche delta complex and they form a sub-estuary within the Mississippi River deltaic plain (USFWS 1987). Historically, the Mississippi River deposited sediment, fresh water, and nutrients into the Barataria Basin during annual overbank flooding cycles; these deposits nourished and sustained wetland habitats. Levees and channelization of the Mississippi River altered natural sediment transport from the river into the basin, eliminating the source of sediment and fresh water that built and maintained wetlands and marshes. As a result, the basin is suffering from significant coastal habitat loss (Couvillion et al. 2011, CPRA 2012). The Barataria Basin lost approximately 29% of its total land area between 1932 and 2016 (Couvillion et al. 2017).

Land loss occurs due to a complex mix of natural and human causes, and the Barataria Basin has been impacted by multiple events and forces (described further in EIS Chapter 3), including the following:

- storm and hurricane events;
- erosion, subsidence, and sea-level rise;
- industrial, commercial, and residential development;
- additional flood risk management and drainage efforts; and

- the Deepwater Horizon (DWH) oil spill.

Various agencies and nongovernmental organizations have implemented coastal protection, restoration, and rehabilitation projects within the basin in response. The State of Louisiana has adopted a Coastal Master Plan that includes 124 projects Louisiana that are expected to build or maintain more than 800 square miles of land over the 50-year planning horizon for the plan (CPRA 2017a). Additional information on past, present and reasonably foreseeable CPRA projects within the Project area can be found in the EIS Chapter 4. Additionally, the LA TIG has singled out the Barataria Basin as a key restoration target and has signaled its restoration intentions in this basin via a Strategic Restoration Plan (LA TIG 2018); the restoration plan also identifies a large-scale sediment diversion as a critical aspect of holistic ecosystem restoration in this area.

1.3 Project Characteristics

The proposed Project is the construction, operation, and maintenance of a controlled sediment and freshwater inlet diversion structure, conveyance channel, and discharge system that will discharge sediment, fresh water, and nutrients from the Mississippi River into an outfall area within the mid-Barataria Basin. The diversion structure would be located in Plaquemines Parish on the right descending bank of the Mississippi River at river mile (RM) 60.7. The conveyance system will cut west through Plaquemines and Jefferson parishes to discharge into estuarine marsh habitat on the east side of mid-Barataria Bay (Figures 1.4-1 and 1.4-2).

The design elements of the proposed Project are separated into 3 categories:

- Diversion Complex – The diversion complex will comprise features that form the basic structural elements for water inlet and conveyance from the Mississippi River to the basin outfall area.

- Basin Outfall Area – This is the basin side of the outfall area within the action area (Section 2.4), where initial delta formation is anticipated; features will be constructed here that have been determined to increase the efficiency of water and sediment accumulation.

- Auxiliary Features – These are Project elements that accommodate existing or future services and infrastructure, including road, rail, and utilities and drainage systems. These features are considered to be interrelated and interdependent and will be addressed in Section 2.3.7 below.

The proposed Project will require, at a minimum, 3 to 5 years of construction, depending on the extent of needed ground modifications and soil stabilization measures. Based on preliminary plans, construction will likely occur in several phases.

The proposed Project includes a diversion operations plan. Operation of the large-scale sediment diversion will be triggered with gates opening for flow when the Mississippi River gage at Belle Chasse reaches 450,000 cubic feet per second (cfs) and reducing to a base flow of 5,000 cfs when flow at the Belle Chasse gage falls below 450,000 cfs. When Mississippi River flows exceed 450,000 cfs, flow through the diversion will vary, with a maximum diversion flow of 75,000 cfs. Flow rates will increase proportionately to flow in the Mississippi River until the Mississippi River gage at Belle Chasse reaches 1,000,000 cfs, at which point flow through the diversion will be capped at 75,000 cfs. Operations will be maintained in a manner to prevent reverse flow from the Barataria Basin to the Mississippi River. Diversion operations will be suspended prior to and during major storm events.

1.4 Project Location

The structural features of the proposed Project are located in south Louisiana on the west bank of the Mississippi River at RM 60.7, just north of the town of Ironton, and the anticipated outfall area for sediment, fresh water, and nutrients conveyed from the river is located within the mid-Barataria Basin (see Figure 1.4-1). The proposed Project area comprises the area within the hydrologic boundaries of the Barataria Basin and the western portion of the lower Mississippi River Delta Basin. The proposed Project area also includes the Mississippi River itself beginning near RM 60.7 and extending to the mouth of the river. Detailed information regarding the proposed Project features and the MBSD Project area can be found in the EIS Chapter 2 and Section 3.1, respectively.

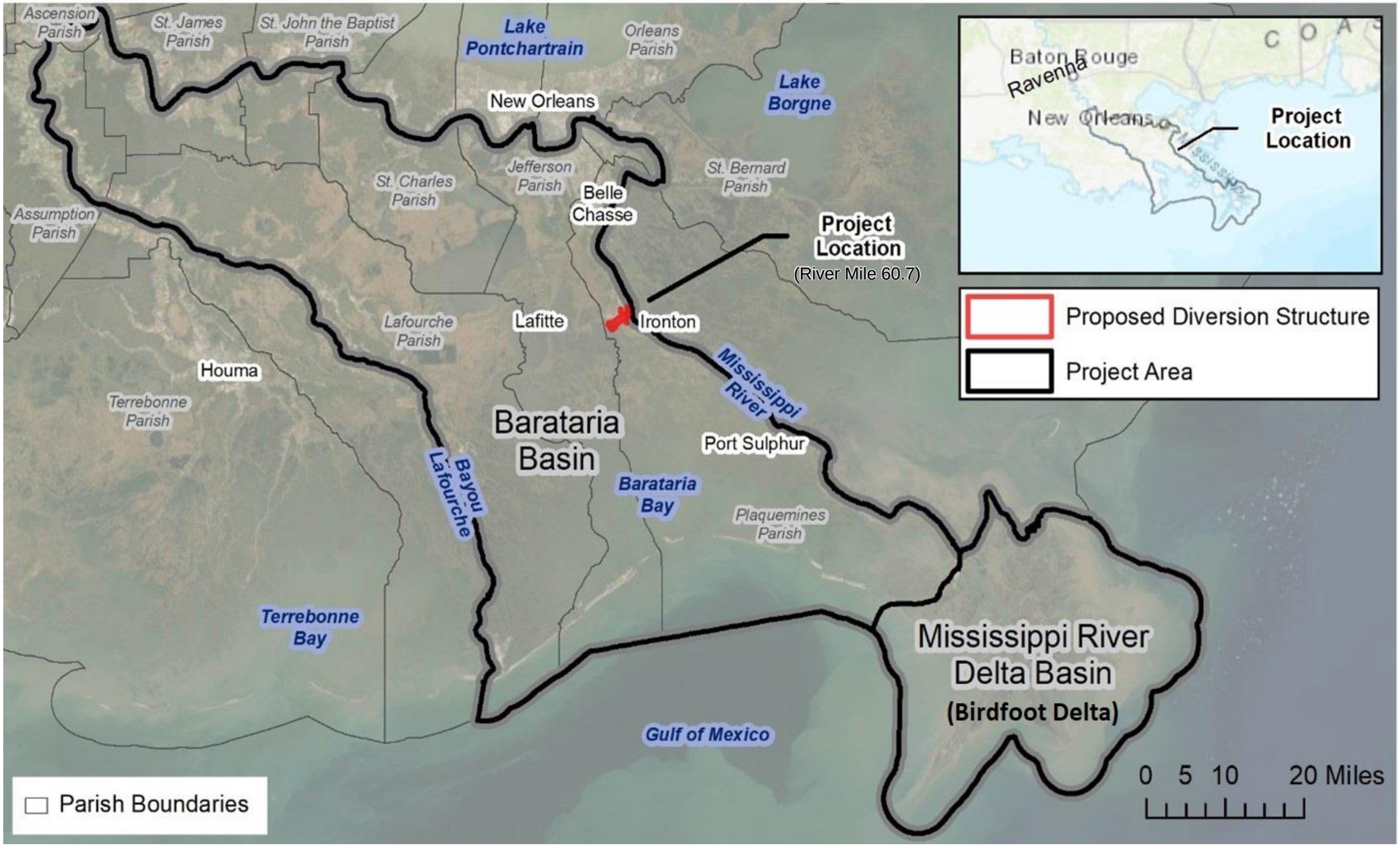


Figure 1.4-1. Location of Project Area (Barataria Basin, western portion of the lower Birdfoot Delta Basin, the Mississippi River from RM 60.7 to the mouth, and a portion of the northern Gulf of Mexico

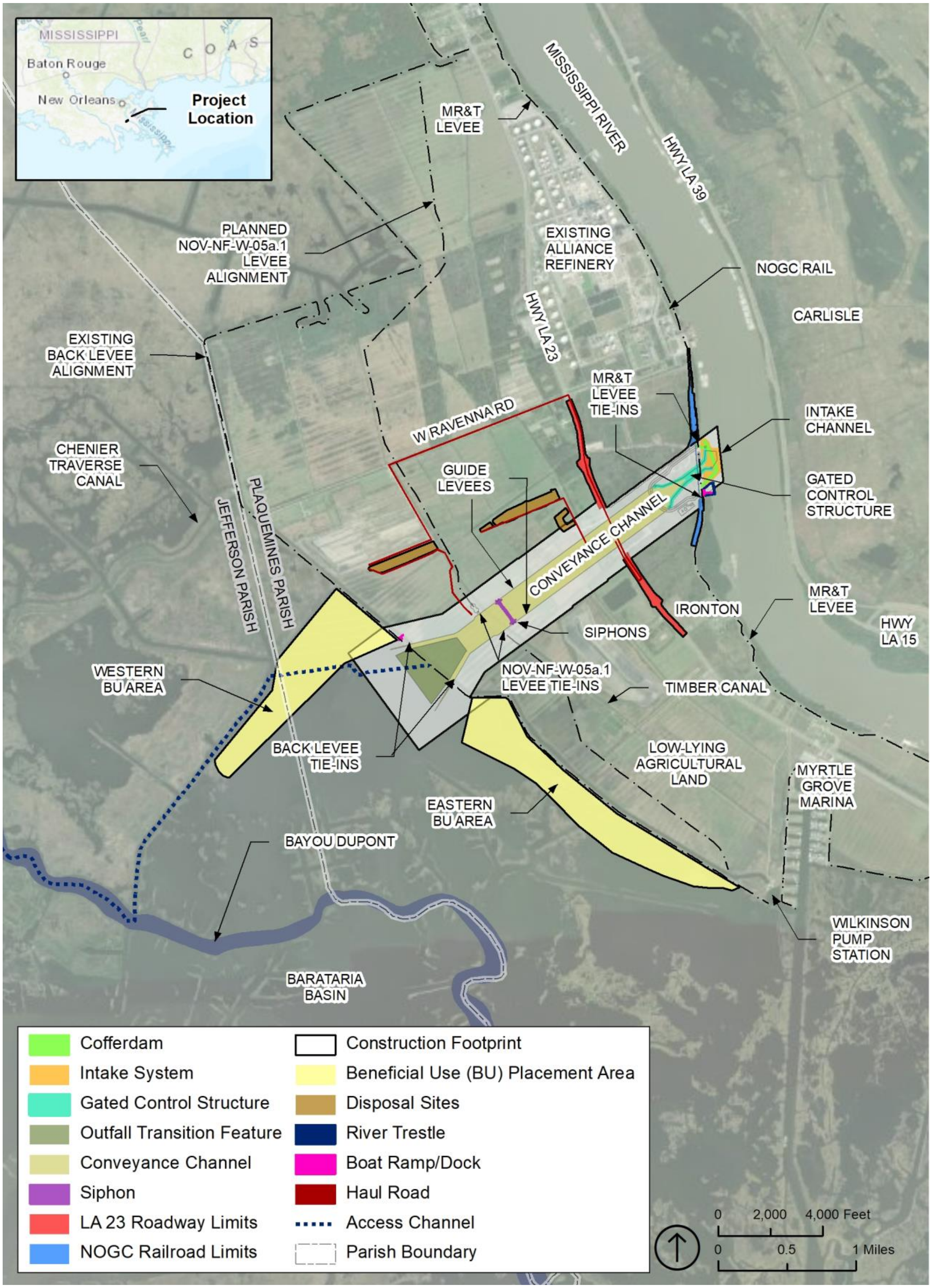


Figure 1.4-2. Project Design Features and Construction Footprint

1.5 Pre-Consultation Technical Assistance

Federal agencies including CPRA, the USACE, and the USFWS and NMFS (the Services) have been participating in a pre-consultation technical assistance process. The goal of this process is to facilitate collaboration between regulatory entities as the Project progresses through NEPA, ESA and EFH consultation, project design, acquisition of permits, and definition of mitigation. This process has provided a forum for the following:

- Ensuring project consistency with regulations
- Sharing information with regulatory agencies in real time
- Clarifying regulatory agency preferences for Project design
- Identifying issues early enough to avoid costly redesigns and schedule delays
- Providing feedback to the project team about how best to comply with anticipated permit requirements
- Testing potential courses of action and airing assumptions in a collaborative environment
- Identifying where regulatory agency requirements differ and developing approaches for reconciling these differences
- Building collaborative relationships

The LA TIG, which is responsible for restoring the natural resources and services within the Louisiana Restoration Area that were injured by the DWH oil spill, has also been involved in this process. The LA TIG has convened numerous working groups to address various aspects of the Project that are relevant to the ESA consultation. These include the following:

- A MBSD Monitoring and Adaptive Management Plan (MAMP) development working group
- A combined state and federal working group called the UFT comprised of the USACE, Federal Coordination Team (comprised of USACE, NOAA/NMFS, DOI/USFWS, EPA, and USDA/NRCS) and LA TIG (including CPRA). This group meets monthly and includes invited or contracted technical participants. This working group has several subject-specific working groups to address technical issues related to project including:
    - Modeling Working Group that addressing the various models being used to evaluate Project impacts, including Delft 3D, Ecosystem Models, and ADCIRC
    - EFHA/ESA Working Group that facilitates pre-consultation coordination efforts with representatives and technical experts from NMFS and USFWS that provide agency input on draft analyses and technical documentation

1.6 Recent Consultations and Existing Information

Prior consultations with the Services regarding projects that overlap the geographic area, activities, species, or habitats may provide guidance for many facets of the current ESA consultations. During the pre-consultation technical assistance process, the Services identified the following consultations and processes that help inform the current biological assessment:

- Framework Biological Opinion on Deepwater Horizon Oil Spill Final Programmatic Damage Assessment and Restoration Plan and Final Programmatic Environmental Impact Statement (SER-2015-17459) (NMFS 2016)
- USACE Projects
  - Louisiana Coastal Area (LCA) Small Diversion at Convent/Blind River (USFWS 2009)
  - LCA Medium Diversion at White Ditch (USFWS 2010)
  - Bonnet Carré Spillway 2011 and 2016 Emergency Operations (USFWS 2018)
  - Bonnet Carré Spillway 2018 Emergency Operations (USFWS 2020a)
  - Bonnet Carré Spillway 2019 Emergency Operations (in press)

The Biological Opinion listed above analyzes the Project area for restoration actions resulting from the DWH PDARP—a framework for a comprehensive programmatic restoration plan that will guide the development of Project-level actions. While the MBSD Project is a component of the DWH PDARP, it was recognized that sediment diversion projects will require independent evaluations.

The 5 USACE projects listed above represent existing Mississippi River flow diversion activities. Each project contains similarities to the MBSD; however, the MBSD occurs at a different section of the Mississippi River, discharges into different basins, and has different planned operational characteristics.

2.0 DESCRIPTION OF THE PROPOSED PROJECT AND ACTION AREA

The following section describes the proposed Project, which will define the proposed Project and action area for the biological assessment.

Information in this section is consistent with the EIS, RP, and supporting information provided by CPRA. Where appropriate, this section will refer to sections of the EIS or RP for additional information.

2.1 Discussion of Federal Action and Legal Authority

The regulatory authority of the USACE for this Project includes Section 10 of the Rivers and Harbors Act and Section 404 of the Clean Water Act (CWA) (collectively referred to as "Section 10/404"), as well as Section 408 of the Rivers and Harbors Act of 1899. USACE approvals and permissions under these authorities constitute a federal action that may affect ESA listed species. This BA, as noted provides the information required pursuant to the ESA and implementing regulations 50 CFR 402.14 to prepare a BA and initiate formal Section 7 consultation. This BA is submitted to the Services by the U.S. Army Corps of Engineers New Orleans District (CEMVN) to initiate formal consultation regarding effects to threatened and endangered species from the MBSD Project. This BA is promulgated in accordance with Section 7 of the ESA and the implementing regulations referenced above.

In addition to the USACE permitting review, Natural Resource Damage Assessment (NRDA) funds arising from the DWH oil spill settlement are being considered as a potential funding source for this Project. These funds are managed by the LA TIG, which includes several federal agencies (NOAA, U.S. Department of the Interior [DOI], the U.S. Environmental Protection Agency [USEPA] and U.S. Department of Agriculture [USDA]). NOAA serves as the lead Federal natural resource trustee for the DWH PDARP. The federal trustees' approval of funds for this Project also constitute a Federal action for the purpose of Section 7 consultation. This BA is submitted to the Services by the federal trustees on the LA TIG pursuant to the same implementing regulations, requesting initiation of formal consultation regarding impacts to threatened and endangered species from the Project.

2.2 Project Purpose

The purpose of the proposed Project is to restore for injuries caused by the DWH oil spill by implementing a large-scale sediment diversion in the Barataria Basin that will reconnect and reestablish sustainable deltaic processes between the Mississippi River and the Barataria Basin through the delivery of sediment, fresh water, and nutrients to support the long-term viability of existing and planned coastal restoration efforts (LA TIG 2019, GEC 2019). This project purpose is consistent with LA TIG's Strategic Restoration Plan and Environmental Assessment #3 and the 2017 Louisiana Coastal Master Plan, and as stated by CPRA in their Section 404 permit application. The proposed Project is needed to help restore habitat and ecosystem services injured in the northern Gulf of Mexico as a result of the DWH oil spill.

2.3 Project Description[1]

This section will provide a detailed description of the Project, including the major Project elements from construction through operation and maintenance. Interdependent and interrelated actions will also be described.

Development of the Sediment Diversion will include the following Project elements:

- Diversion Complex
  - Intake System
  - Gated Control Structure
  - Conveyance Channel
  - Guide Levees
- Outfall Area
  - Outfall Transition Feature
- Auxiliary Features
  - Linear Infrastructure
  - Beneficial Use Placement Areas
  - Mitigation

An overview of the Project Description is captured in Table 2.3-1.

[1] The project description is based on early design (15% to 30% design) and will be updated prior to issuance of the draft EIS to match the project description in the dEIS (60% design)

Table 2.3-1. Mid-Barataria Sediment Diversion Project Activities

Project Element	Project Feature	Project Phase	Project Action	Habitat (Aquatic/ Terrestrial)	Aquatic Interaction (River/Basin)
Diversion Complex	All Features	Diversion Operations	Baseline Diversion Flow (5,000 cfs diverted)	A	R,B
Diversion Complex	All Features	Diversion Operations	Intermediate Diversion Flow River between 450,000 and 1,000,000 cfs (Between 5,000 and 75,000 cfs diverted)	A	R,B
Diversion Complex	All Features	Diversion Operations	High Diversion Flow River >1,000,000 cfs (75,000 cfs diverted)	A	R,B
Diversion Complex	All Features	Construction Activities: Phase 1 (Site Prep)	Clearing and grubbing the limits of terrestrial construction	T	R, B
			Access: Haul road excavation and construction, unloading areas, parking pads, fencing	T	R, B
			Staging: constructing and/or stabilizing staging areas	T	R, B
			Clearing and grubbing the limits of aquatic construction	T	R
			Access: dredging for barge access (river side)	T	R
			Access: dredging for barge access (basin side)	T	B
			Staging: barge unloading area	T	R
		Construction of Foundation Systems: Phase 1-3 (Construction)	Pile Driving	A	R
			Surcharge area with excess fill to consolidate sediments	T	R, B
			Dewatering/rewatering	A	R, B
			Excavation	A	R, B
			In-the-dry construction: Sheet pile installation/removal	A	R

Project Element	Project Feature	Project Phase	Project Action	Habitat (Aquatic/ Terrestrial)	Aquatic Interaction (River/Basin)
	Intake System, Gated Control Structure	Construction Activities: Phase 1-3 (Construction)	In-the-dry construction: Dewatering/rewatering	A	R
			Sediment excavation and disposal	A	R
			Concrete placement	A	R
			Staging during construction. Barge delivered materials	A	R
	Conveyance Channel	Construction Activities: Phase 1-3 (Construction)	Sediment excavation and disposal	A	R
			Sediment excavation - mechanical	A, T	R
			Grading and top soil spreading	A, T	R
	Guide Levees	Construction Activities: Phase 2-3	Sediment placement	T	R, B
			Establish vegetation	T	R, B
			Install wells	T	R, B
Basin Outfall Area	Outfall Transition Feature	Construction Activities: Phase 1 (Site Prep)	Clearing and grubbing the limits of aquatic construction	A	B
			Staging (barge landing)	A	B
			Staging (pier)	A	B
			Staging during construction. Barge delivered materials	A	B
Auxiliary Features	Linear Infrastructure	Railway (NOGC)	Railway bridge construction	A, T	R
		Highway LA 23	Raised and relocated	T	N/A
		Utilities - Power	Relocation of existing power right-of-way (ROW)	T	N/A
		Utilities - Fiber Optic	Relocation of existing Fiber Optic ROW	T	N/A
		Utilities - Water	Relocation of 16-inch water main for Plaquemines Parish	T	N/A
		Utilities - Shell Pipeline	Relocation of existing Shell Pipeline	A, T	B

Project Element	Project Feature	Project Phase	Project Action	Habitat (Aquatic/ Terrestrial)	Aquatic Interaction (River/Basin)
		Drainage System	Siphon drain option	A, T	B
	Beneficial Use Placement Areas	Beneficial Use Placement Areas	Beneficial Use Placement Areas	A	B
	Mitigation	Wetland and aquatic mitigation	Construction of wetland and aquatic mitigation	A	R, B
Diversion Complex	All Features	Maintenance of Sediment Diversion	Debris management	A	R, B
			Channel repairs/modifications	A	R, B

2.3.1 Site Preparation

Site preparation for construction of the major Project features includes clearing and grubbing, stockpiling and placement of material, excavating and constructing of haul roads (including drainage channels, cross-drain structures, and access fencing), hauling of material, grading and paving, dredging, pumping of dredged material to prepared disposal site(s), installation of sediment and erosion control measures and slope protection, permanent and final stabilization, and extension of utilities to serve the proposed Project. A more detailed description of the basis of design for construction of the proposed Project is provided in Appendix B.

Various types of equipment would be present and operating throughout the construction of the Project, including excavators, trucks, loaders, dozers, rollers, scrapers, pile drivers, cranes, barges, and well point drill rigs for dewatering. The means and methods of the construction contractor will determine what equipment would be on site. A concrete batch plant will be placed in the proposed construction footprint to produce the large volumes of concrete needed for the large structures. A temporary offloading facility may be constructed by the contractor on either the river or basin (or both) to accommodate safe materials transfer.

Staging Areas

Areas associated with Project construction activities will be located within the overall footprint of the construction limits. Staging areas and construction yards will be about 8 acres. An additional 4 acres will be used for a concrete batch plant. The contractor will select the final size and locations of these areas. Staging areas will include the following:

- Haul and access roads
- A concrete batch plant
- Barge offloading facilities located on the Mississippi River and in the Barataria Basin
- A staging area for barge-delivered materials
- Construction yards
- A laydown area for drying and processing clay borrow from excavations

Transport/Access Routes

Access routes will be used to transport construction equipment and to dredge the outfall transition feature. There is one planned access route from the north to the proposed outfall area, as shown in Figure 2.3.1-1. This route follows a route used for previous restoration projects that had similar required draft for vessels. The route can be accessed from the Gulf Intracoastal Waterway via the Barataria Bay Waterway. The Project will also utilize the Mississippi River, which is navigable by ocean-going vessels up to Baton Rouge and by barge traffic all the way to the Port of Minneapolis, Minnesota.

The Basin-side routes may be adjusted based on survey of bathymetry and presence of underwater obstructions, or oil and gas infrastructure. The route avoids most pipelines in the area; however, pipelines parallel to the existing New Orleans to Venice Non-Federal Levees (NOV-NFL) may be unavoidable and these pipelines may need to be lowered to facilitate dredge access for the Project. Approximately 303,000 cubic yards are projected to be excavated for the channel. The barge and equipment access route includes dredging a bottom withd of approximately 50 feet and bottom elevation of -9.00 feet North American Vertical Datum 1988 (NAVD 88) to provide flotation clearance during the construction phase. The current channel has an average depth of approximately -4 ft NAVD88. Excavated materials will be deposited adjacent to the channel and deposition areas are projected to create a crest that would be exposed to approximately +2 ft (NAVD88) above the mean tide line

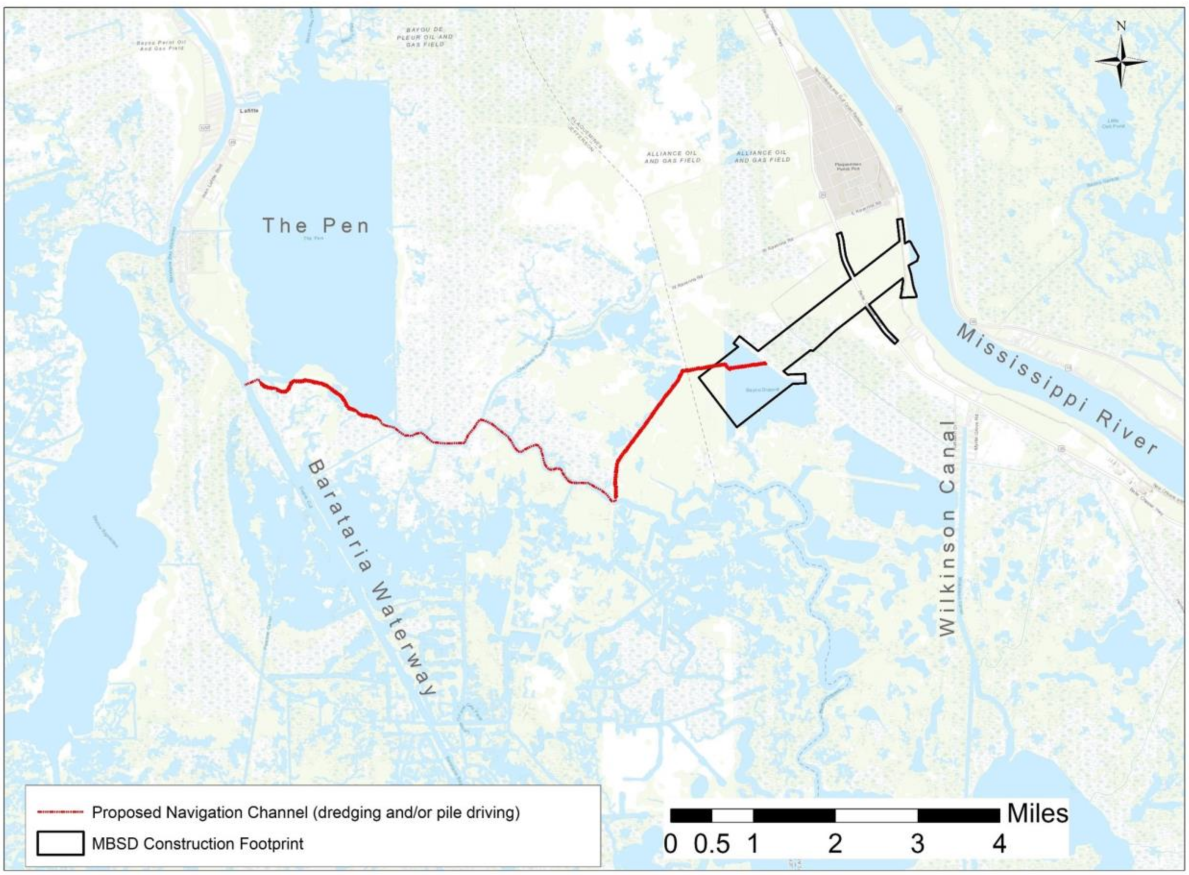


Figure 2.3.1-1. Potential Access Routes (white) that may be dredged for barge access

2.3.2 Sediment Diversion Construction

The proposed Project would require, at a minimum, 3 to 5 years of construction, depending on the extent of needed ground modifications and soil stabilization measures. Construction would likely occur in several phases.

The design elements of the proposed Project are separated into 3 categories: (1) diversion complex, (2) basin outfall area, and (3) auxiliary features (Figures 2.3.2-1, 2.3.2-2, 2.3.2-3, 2.3.2-4, and 2.3.2-5). Design elements of the diversion complex and basin outfall area are described below. Auxiliary features are described as interdependent and interrelated actions (see Section 3.3.7).

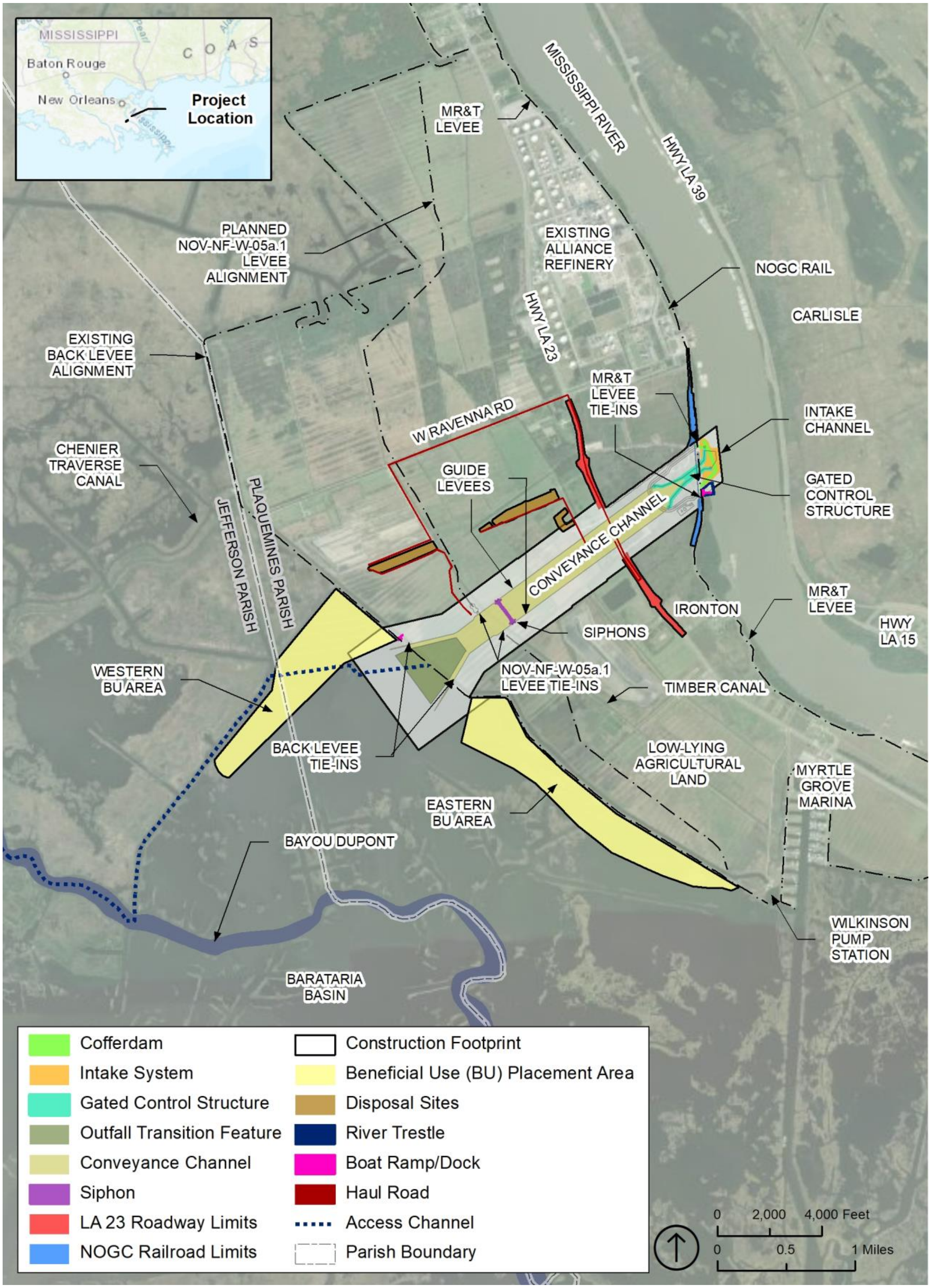


Figure 2.3.2-1. Project Construction Footprint

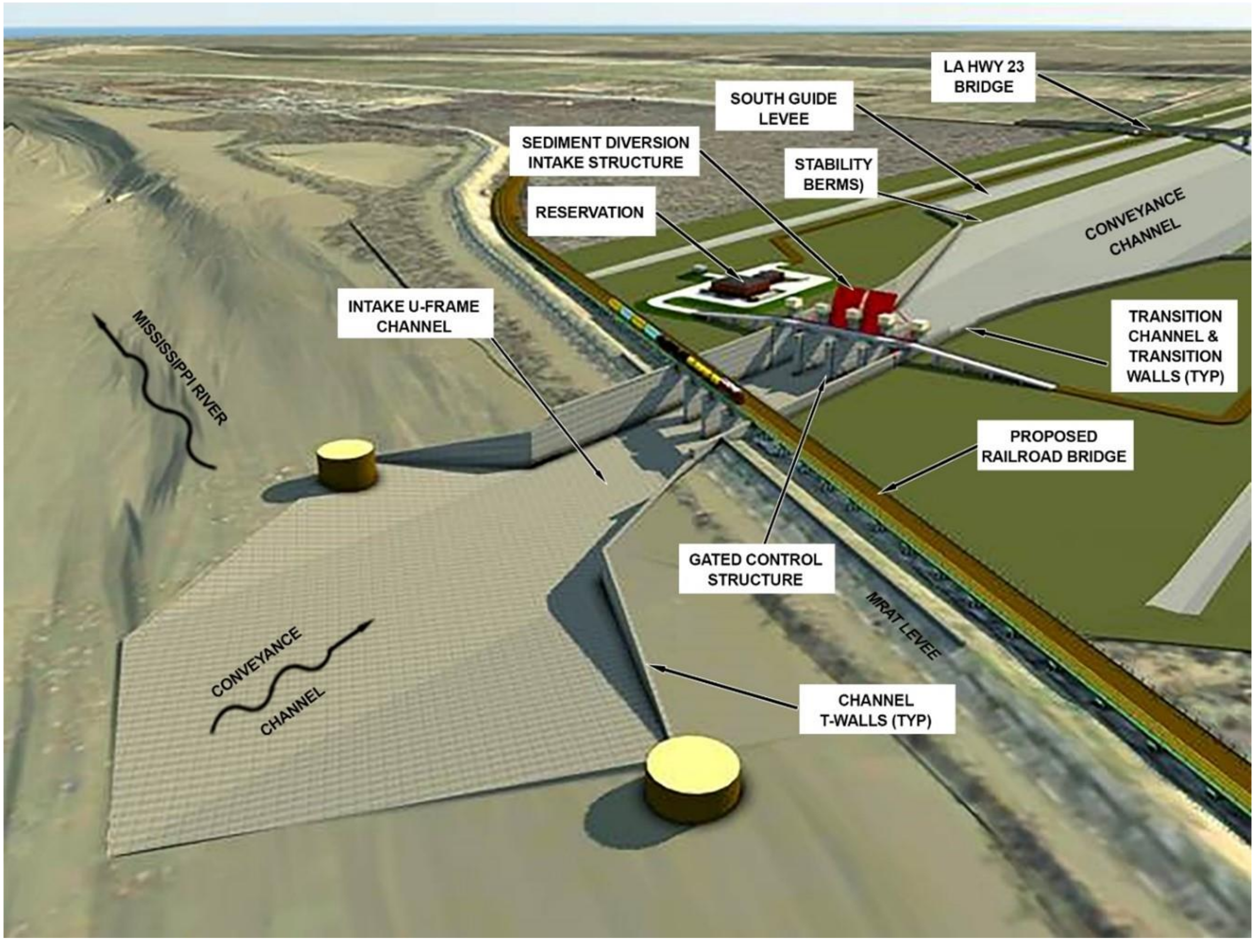


Figure 2.3.2-2. Proposed Project Design Features as Viewed from the Mississippi

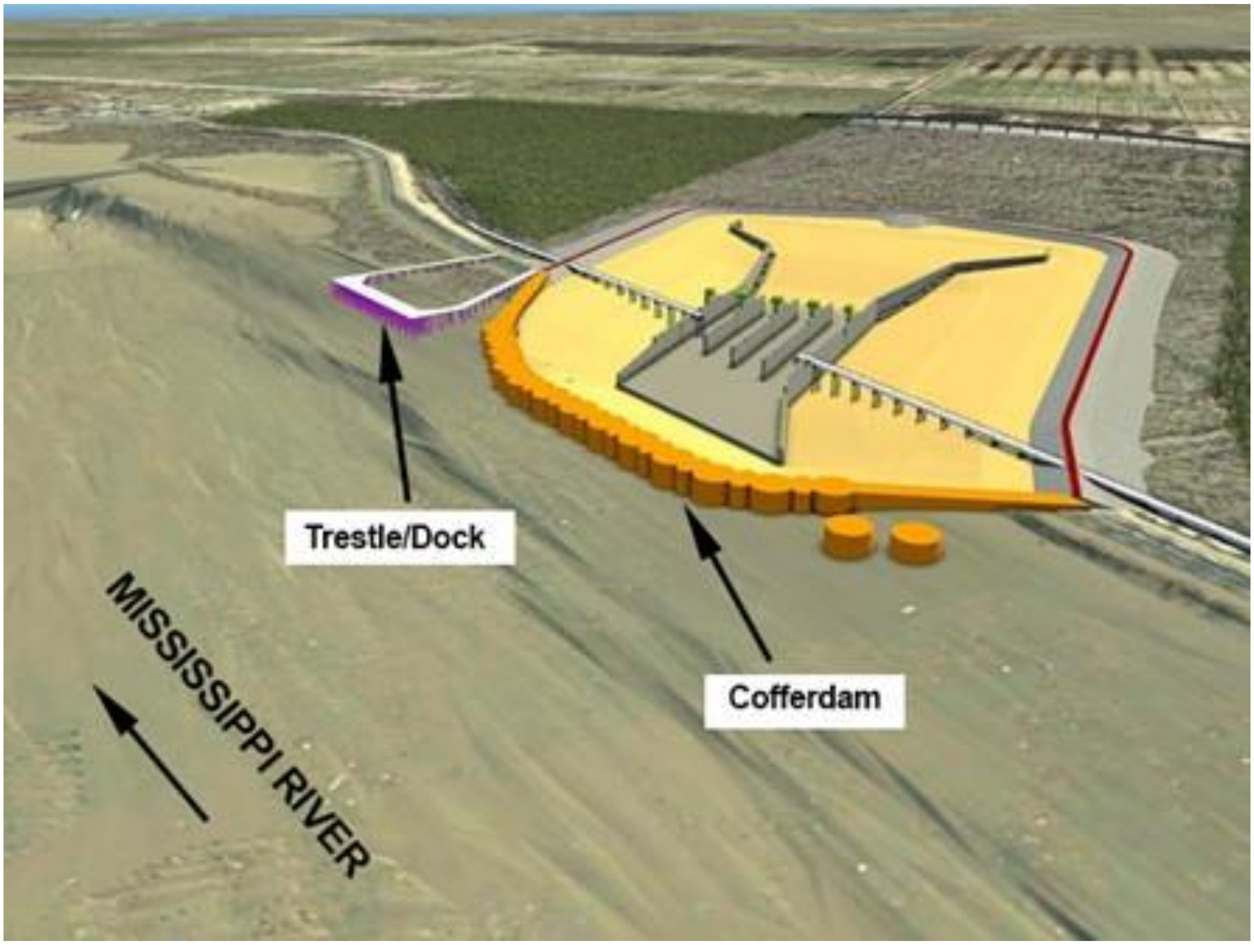


Figure 2.3.2-3. Proposed Trestle and Construction Cofferdam Overview

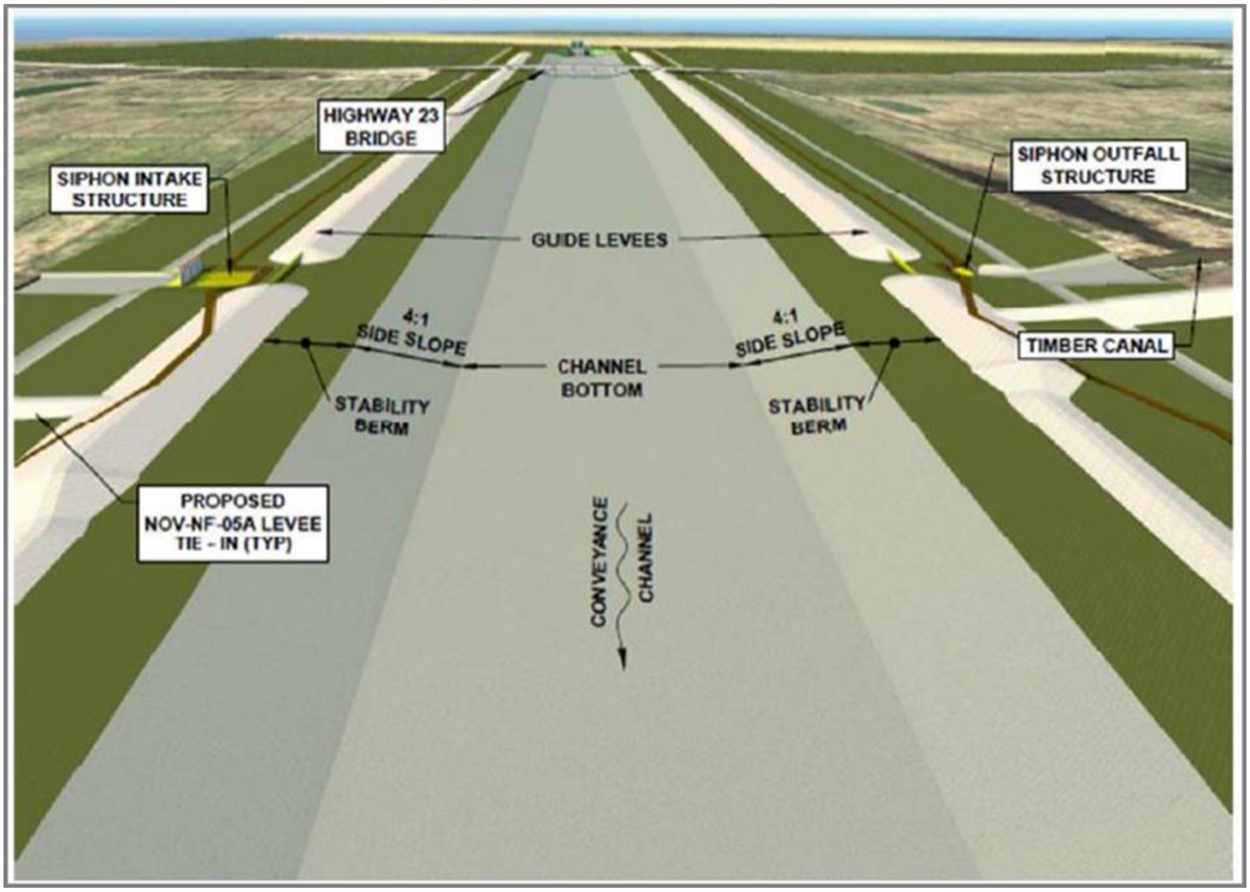


Figure 2.3.2-4. Proposed Conveyance Channel, Guide Levees, Stability Berms, and Siphon

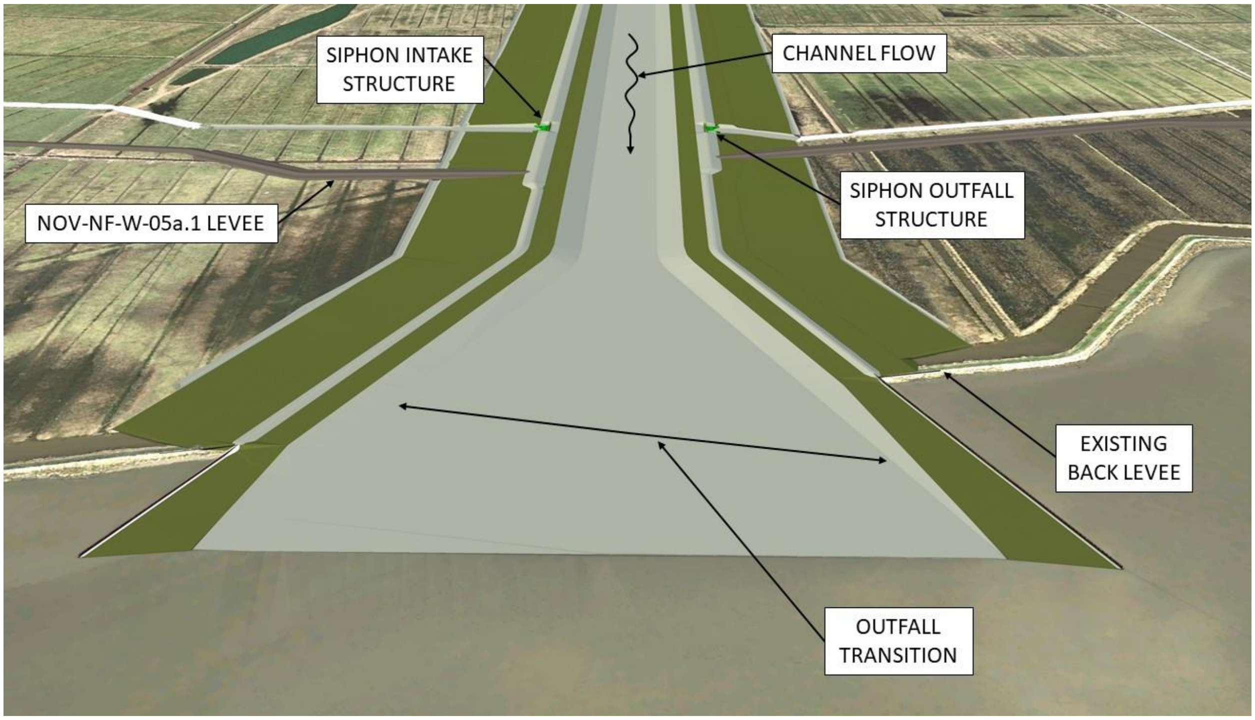


Figure 2.3.2-5. Proposed Outfall Transition Feature

Diversion Complex

The diversion complex would consist of the following features: intake system structure, gated control structure, conveyance channel guide levees, and stability berms. These features would be designed to convey sediment, fresh water, and nutrients from the Mississippi River to the Barataria Basin by way of a control structure confined by guide levees and with enough velocity to prevent buildup of siltation in the channel and to protect against scour. During construction a pile supported trestle with a total surface area of approximately 36,000 s.f. would be installed just downstream of the intake along the Mississippi River for material transfer (Figure 2.3.2-2). The proposed construction limits for the diversion complex would be approximately 1,015.4 acres.

Intake System

The intake system consists of an intake structure (with two flared training walls and an intake channel), a gated control structure, and a transition channel that would connect to the larger conveyance channel (Figures 2.3.2-1 and 2.3.2-2). The training walls and intake channel would be located on the Mississippi River bed slope and adjacent to the sand bar, which occurs at an approximate depth elevation of -50 feet to -70 feet. The training walls would extend into the Mississippi River about 950 feet shoreward (west) of the Mississippi River navigation channel limits.

The training walls would direct the flow of sediment from the river into the intake and restrict riverbank soils from filling in the channel. The walls would be inverted pile-founded T-walls that would gradually increase in elevation from 0.0 and -13.0 feet, respectively, in the river to approximately 16.4 feet where they would connect to the intake channel walls. A temporary cofferdam system would be built around the proposed training walls to dewater the area during construction. It is estimated the cofferdam will be in place for up to 3.5 years. After construction, the cofferdam system would be removed.

Gated Control Structure

The gated control structure would consist of 4 45-foot-wide steel tainter gates with an invert elevation of -40 feet and a top-of-wall elevation of 16.4 feet. Water flow would be regulated by raising or lowering the gates. The river side of the structure would tie into the current Mississippi River & Tributaries (MR&T) Project Levee alignment, with 4 machine rooms and a maintenance bridge across the top. The gates would be operated with commercial power; diesel generators would be used as back-up. For seepage control, subsurface cutoff walls and drainage systems would be incorporated.

From the gated control structure, water would be funneled through a U-shaped transition channel with widths increasing from the gated control structure to the trapezoidal conveyance channel. The transition wall system under consideration would be pile-supported inverted T-walls.

Construction methods for the gated control structure are provided in EIS Section 2.8 and include the following: construction of subsurface seepage cutoff walls and drainage systems; construction of a temporary setback levee to reduce the risk of flooding until the gated control structure is completed; and construction “in-the-dry” behind the existing MR&T Levee.

Conveyance Channel

The conveyance channel, lined with bedding stone and riprap, would convey sediment-laden river water from the gated control structure and transition channel to the Barataria Basin. From the gated control structure, water would be funneled through transitional widths. The conveyance channel would have a 300-foot bottom width with an invert elevation of –25 feet, setback berms between the top of channel and toe of the guide levees, and guide levees (see Figure 2.3.2-3). The total width of the conveyance channel, stability berms, and guide levees would measure 734 feet and would occupy approximately 563 acres, including the guide levees. The channel would cut through a complex geologic environment that includes point bar deposits, marsh deposits, and abandoned distributary channels.

Construction of the conveyance channel would include clearing and grubbing of the site. The wooded area east of LA 23 would be cleared of trees, since these are within the vegetation-free zones around the levees and stability berms, and are not permitted by USACE guidelines

(USACE 2019). Mechanical and hydraulic excavation methods would be used to excavate the channel. Two USACE-approved and environmentally cleared levee clay borrow sites located contiguous to the proposed conveyance channel would be used for fill material for embankments/levee construction if needed, in addition to material generated from channel excavation. Construction methods are detailed in the EIS Section 2.8.

Guide Levees

Earthen guide levees would be constructed along both sides of the conveyance channel as a linear feature designed to constrain project flows (Figure 2.3.2-3). Drain systems would be incorporated into the levees to expedite soil consolidation and settlement. It is anticipated that multiple lifts and construction sequences would be needed to bring the guide levees to their final design height. The guide levees would also serve as hurricane flood protection against storm surges and would be built to an elevation of 15.6 feet, which is the USACE Design Grade for the proposed upgraded NOV-NFL levee. The levees would include a 10-foot-wide levee crown topped with a gravel access road. The levees would be constructed from soil material excavated for construction of the intake channel and conveyance channel.

Basin Outfall Area

The outfall area is defined as the area on the basin side of the outfall channel that will receive the sediment, fresh water, and nutrients from the Mississippi River via the conveyance channel. This area is delineated by Cheniere Traverse Bayou to the north, Wilkinson Canal to the south, and the Barataria Bay Waterway to the west, and is approximately 676 acres (Figure 2.3.2-1). The area largely consists of degraded wetland, shallow open water, and oil and gas canals. It is anticipated that a delta will form in the outfall area. Details about Project-induced land building in the basin are provided in the EIS Section 4.2.

Modeling efforts indicate that, upon proposed Project initiation, sand and coarse-grained sediments would be deposited within the outfall area in an initial delta formation with deposition of finer-grained sediment extending farther gulfward in the basin, forming a subaqueous delta just below the low-tide water level. Over time, the subaqueous delta will evolve into a subaerial delta above the low-tide water level as vegetation becomes established and encourages additional deposits of sediment. This would in turn extend the formation of new subaqueous delta farther gulfward into the basin. Fine-grained sediments transported by the diversion will travel farther from the outfall area and be dispersed throughout the proposed Project area.

Outfall Transition Feature

The Project design includes the creation of an outfall transition feature (OTF) to increase the efficiency of water and sediment delivery. To create the OTF, the receiving basin surrounding the outlet will be dredged to create a gradual gradient from the diversion channel invert elevation of -25 feet (the bottom grade elevation of the channel) to the existing bed elevation of the receiving basin (-4 feet). The OTF is designed to provide sufficient bed topography for the diversion to flow at maximum capacity, expediting initial delta formation. The OTF will be created by dredging bottom sediment from the open water area within about 640 acres (1 square mile) of the outfall transition walls of the diversion structure. These sediments will be placed at designated beneficial use locations in the receiving basin shown in Figure 2.3.2-1. The bottom of the OTF will be armored with riprap.

Pile Driving

Temporary cofferdams would be used during Project construction for dewatering in-water work areas, controlling groundwater, and to provide structural support. Areas where cofferdam installation would likely occur are shown in Figure 2.3.2-5. Installation methods may include impact, auguring, vibrating, or other methods. In general, upland pile driving may use either impact or vibratory pile drivers without noise attenuation. Sheet piles will be installed using vibratory methods to the extent practicable. In-water pilings may be driven with impact or vibratory pile drivers. Estimated quantities, pile types, and duration of pile driving by location are shown in Table 2.3.2-1.

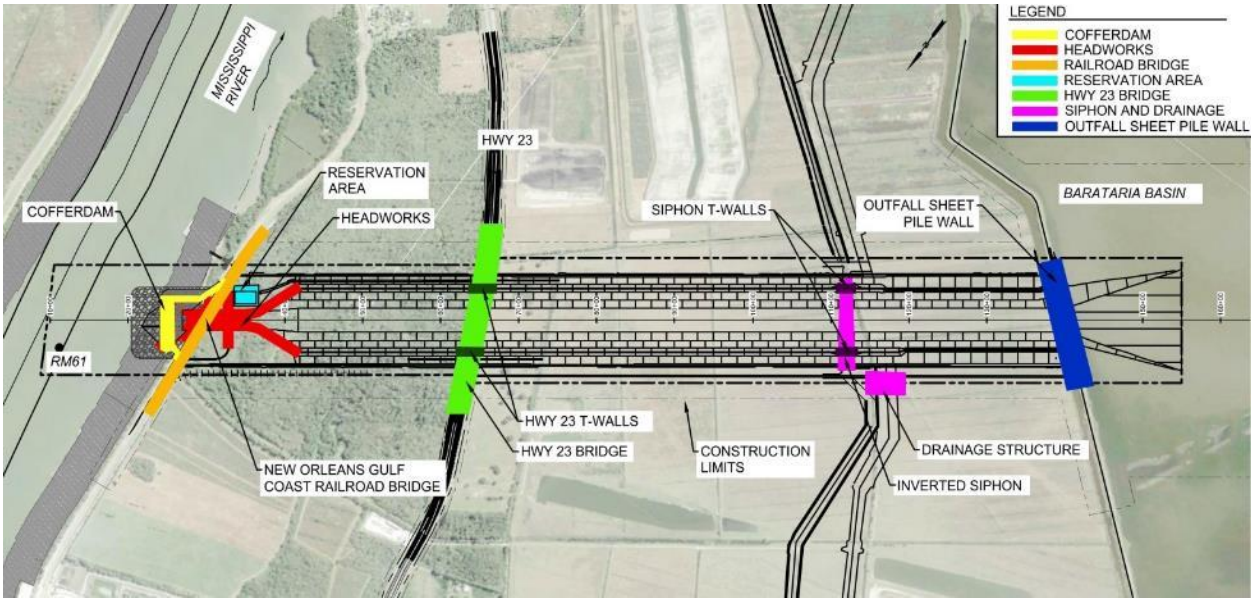


Figure 2.3.2-5. General Locations where Pile Driving is Planned

Table 2.3.2-1 Proposed Design Pile Installation Information for Mid-Barataria Sediment Diversion

Project Area	In Water?	Pile Type	Installation Method	Pile Depth (ft)	Pile Count (# or footage)	Blows/ Pile (#)	Installation Duration (months)	Hours/Day
Cofferdam (cofferdam cells, protection cells, and combi wall)	Yes	Sheet (Steel)	Vibratory Hammer	85-100	420,000 square feet (SF) for Mississippi River cofferdam cells (Steel)	NA	5-10	8-12
		Sheet (Steel)	Vibratory Hammer	85-100	105,000 SF for permanent protection cells (Steel)	NA	3-6	8-12
		Steel Piling	Impact, Cushioned	85-100	15,000 linear feet (LF) of pipe or H/I-shaped king piles for combi-wall	500+	2-6	8-12
Headworks (intake, gate, and transition monoliths)	Yes*	Sheet (Steel)	Vibratory Hammer	40-85	120,000 SF of sheet piles	NA	4-8	8-12
		Concrete or Steel H Piling	Impact, Cushioned	100-200	175,000 LF of square, pipe, and/or H-piles	500+	12-15	8-12
New Orleans & Gulf Coast (NOGC) Railroad Bridge	No	Concrete or Steel Piling	Impact, Cushioned	50-100	50,000 LF of square, pipe, and/or H-pile	500+	6-10	8-12
Highway 23 Bridge and T-wall	No	Concrete or Steel Piling	Impact, Cushioned	50-100	50,000 LF of piles	500+	4-6	8-12
		Steel H Piling	Impact, Cushioned	50-100	20,000 LF of H-piles (T-wall)	500+	1-3	8-12
River Trestle	Yes	Steel Pipe Piling	Impact, Cushioned	75-100	132 piles 36-inch	500	2-3	8-12
Inverted Siphon, Sluice Gate, and T-walls	No	Timber, Concrete, or Steel Piling	Impact, Cushioned	60-100	40,000 LF of piles (siphon headworks – Timber, Concrete, or Steel)	250-500	1-3	8-12
		Steel H Piling	Impact, Cushioned	50-100	20,000 LF of H-piles (T-wall – Steel)	500+	1-3	8-12
		Sheet (Steel)	Vibratory Hammer	40-85	40,000 SF of sheet pile (Temporary Retaining Structure & cutoff wall)	NA	1-3	8-12

Project Area	In Water?	Pile Type	Installation Method	Pile Depth (ft)	Pile Count (# or footage)	Blows/ Pile (#)	Installation Duration (months)	Hours/Day
Canal Cut-Off	Yes (Timber Canal)	Sheet (Steel)	Vibratory Hammer	25-80	20,000 SF of sheet pile	NA	1-3	8-12
Outfall	Yes (Basin)	Sheet (Steel)	Vibratory Hammer	50-100	30,000 SF of sheet pile	NA	2	8-12
Boat Pier	Yes (Basin)	Timber piling	Impact	20	30 timber piling (12-inch diameter)	20	5 days	8-12
Navigation Markers	Yes (Basin)	Timber (piling)	Press Installation	20	TBD timber piling (12-inch diameter)	NA	1-2	8-12
Secondary Site Features	No	Timber, Concrete or Steel Piling	Impact, Cushioned	25-100	40,000 LF of pile	250-500	1-3	8-12

*This will be behind the cofferdam during construction. Note: *Information contained in Table 1 is based on information available from the Basis of Design Report and engineering judgement.*

2.3.3 Operation of Sediment Diversion

Lower Mississippi River Conditions & Historic Flows

The Mississippi River carries sediment-rich flows south to the Gulf of Mexico. At the Project location, the depth of the river is approximately 120 feet and a sand bar exists at a depth of about 50 feet. The top width of the river is approximately 2,000 feet. Near the Project inlet, the Mississippi carries a flow ranging from 425,000 cfs to 1,250,000 cfs during typical annual peak flow events. Transported sediment consists of clay, silt, and sand particles. The dominant hydraulic processes in the vicinity of the diversion are longitudinal, transverse, and vertical velocities due to the upstream river bend, suspended sediment transport through the water column, and bed load transport along the sand bar present at the proposed diversion.

Planned Operations Summary

The proposed Project includes a diversion operations plan based on initial sediment transport and deposition modeling. A monitoring and adaptive management plan will be implemented concurrently to observe and evaluate system performance and environmental response. The plan may prescribe operational changes where necessary to improve system performance or if certain threshold environmental conditions are reached.

The diversion operation plan currently calls for initial opening of the sediment diversion gates when the Mississippi River gage in Belle Chasse reaches 450,000 cfs. Once operational, the gates will be operated to maintain controlled diversion rates ranging from a target minimum of 5,000 to a maximum of 75,000 cfs, scaled to flow conditions in the main river. The maximum diversion flow of 75,000 cfs will occur when the Mississippi River gage in Belle Chase exceeds 1,000,000 cfs. The target baseflow diversion rate of 5,000 cfs would occur when Mississippi River flows drop below 450,000 cfs at the Belle Chase gage. The diversion rate between these threshold flows will be controlled by the difference in water surface elevation between the Mississippi River and the Barataria Basin (the "head differential"). When the Mississippi River flow and stage are high, the increased head differential will push a higher volume of water and sediment through the diversion into the Barataria Basin. When the Mississippi River flow and stage are low, there will be less energy to push water and sediment through the diversion. Figure 2.3.3-1 illustrates this variable flow rate for a representative Mississippi River hydrograph from 2011, a high spring flow year (data derived from The Water Institute of the Gulf 2014).

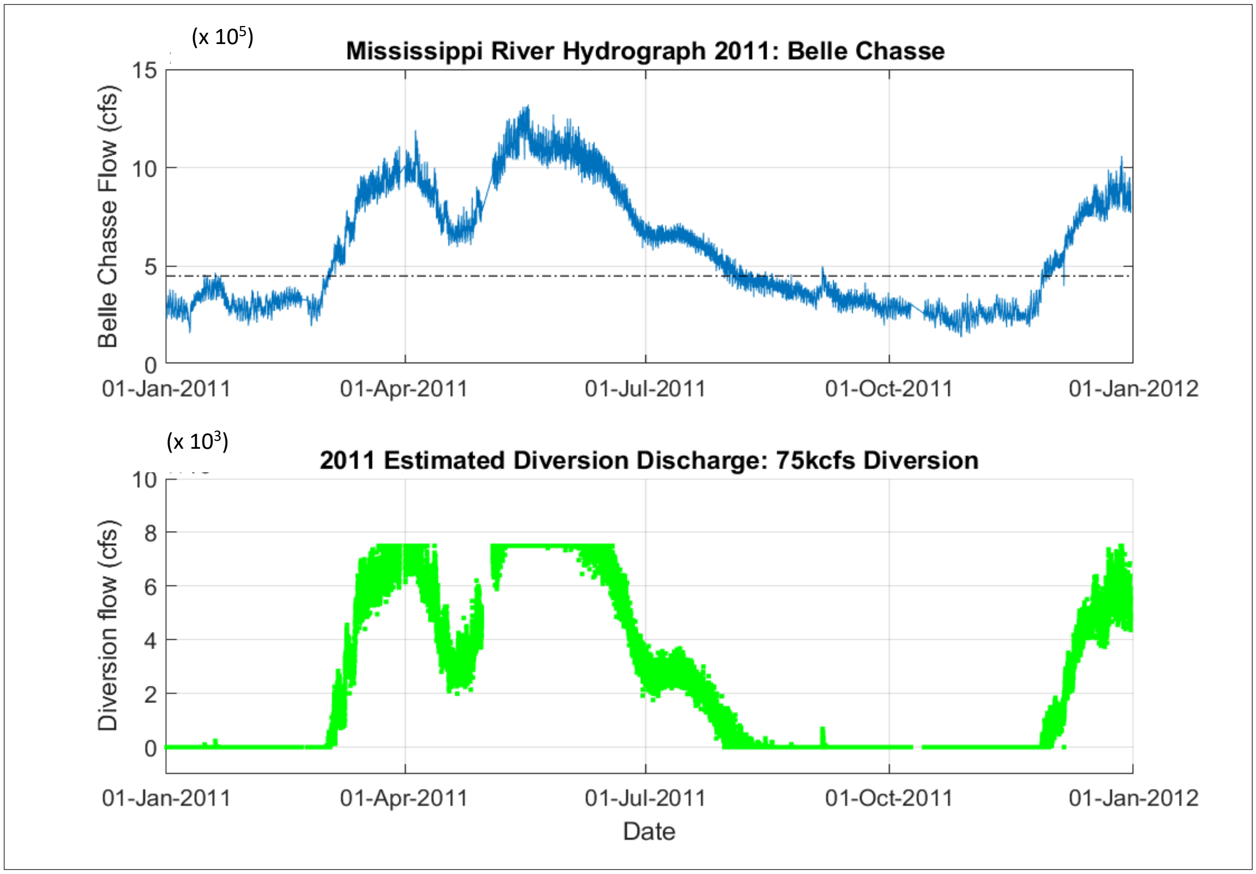


Figure 2.3.3-1. Variable Flow for 75k Diversions (bottom plot) Driven by 2011 Mississippi River Discharge (top plot) with a 450,000 cfs Operational Trigger in the Mississippi River (Water Institute of the Gulf 2015)

The diversion would be operated to maintain the target baseflow diversion rate of 5,000 cfs when river flows drop below 450,000 cfs. The Project proposes to use diversion gates or other alternative methods to maintain sufficient baseflow from the Mississippi River to meet the diversion target, but this may not be possible under all conditions. The diversion rate could theoretically fall to zero when high tides coincide with low baseflows in the main river. The diversion will be operated to prevent backflow from the Barataria Basin towards the Mississippi River.

2.3.4 Maintenance of Sediment Diversion

The sediment diversion may require periodic maintenance activities. These may include the following:

- Periodic inspections of diversion components
- Periodic maintenance of diversion components
- Clearing of vegetation

- Dredging in the outfall area to maintain flows or provide site access/beneficial reuse of sediment

Post-construction operations monitoring, and maintenance will be addressed within the following plans:

- Operations and Maintenance Plan
- Monitoring and Adaptive Management Plan

2.3.5 Description of Auxiliary Features

CPRA identified several auxiliary actions to the proposed Project, which are described in detail Section 2.8.1.2 of the EIS. These actions include the development of road and rail crossings, and other improvements necessary to maintain existing infrastructure that crosses the Project footprint. These activities are addressed in this BA as interrelated and interdependent actions and are described in Section 2.3.7.

2.3.6 Description of Proposed Conservation Measures

Proposed conservation measures include environmental protection measures and best management practices (BMPs) that would be implemented during the construction of the Project to avoid or minimize potential environmental effects.

CPRA will develop an Environmental Protection Plan (EPP) that details the procedures for the prevention and/or control of pollution and habitat disruption that may occur during construction. As part of the EPP, the Plan shall detail the action which the contractor shall take to comply with all applicable federal, state and local laws and regulations concerning environmental protection and pollution control and abatement, as well as any additional specific requirements. The EPP will include an approved Spill Control Plan, Waste Management Plan, Contaminant Prevention Plan, and Environmental Monitoring Plan.

Many of these BMPs are standard approaches that will apply universally to many Project construction or operation activities. This section discusses provisional BMPs that CPRA anticipates will be included as construction or operation commitments for the Project.

Environmental protective measures presented below include those protecting land and water resources. BMPs for biological resources are being developed as impacts are more clearly understood.

Inwater Work – Best Management Practice

Timing Restrictions

The Project will coordinate with natural resource agencies to identify construction activities and timing restrictions applicable to this Project. Given the large amount of in-water construction associated with the Project, it may not be feasible to avoid construction when fish, turtles, or marine mammals are potentially present.

Pile Driving Noise Attenuation

The Project will develop a pile-driving plan to guide pile-driving operations. This plan will identify locations, approximate timing and installation methods including any noise attenuation methods.

West Indian Manatee Protection Measures

All personnel involved with Project-related in-water work in potential manatee habitat shall be fully instructed and trained in measures for avoiding and minimizing manatee impacts. These measures will include, but are not limited to, understanding the potential presence of manatees, maintaining manatee speed zones, and other appropriate measures for avoiding collisions with and injury to manatees. All personnel shall be advised of applicable civil and criminal penalties for harming, harassing, or killing manatees. Additionally, personnel will be instructed not to attempt to feed or interact with manatees. Passively taking pictures or video for the purpose of documenting incidental take avoidance and minimization is acceptable.

The following conservation actions shall be undertaken during construction to reduce the risk of impacts to manatees:

- All on-site personnel are responsible for observing water-related activities for the presence of manatee(s).
- All work, equipment, and vessel operation will cease if a manatee is spotted within a 50-foot radius (buffer zone) of the active work area. In-water work may resume once the manatee has left the buffer zone on its own accord (manatees must not be herded or harassed into leaving), or after 30 minutes have elapsed with no manatee(s) sighted in the buffer zone.
- If a manatee(s) is sighted in or near the Project area, all vessels associated with the Project should operate at “no wake/idle” speeds within the construction area and at all times while in waters where the draft of the vessel provides less than a 4-foot clearance from the bottom. Vessels should follow routes of deep water whenever possible.
- If used, siltation or turbidity barriers should be properly secured, made of material in which manatees cannot become entangled, and be monitored to avoid manatee entanglement or entrapment.
- Turbidity barriers will be placed so they do not impede manatee movement.
- Temporary signs concerning manatees should be posted prior to and during all inwater Project activities and removed upon completion.

Collisions with, injury to, or sightings of manatees should be immediately reported to the USFWS Louisiana Ecological Services Office (337-291-3100) and the Louisiana Department of Wildlife and Fisheries (LDWF), Natural Heritage Program (225-765-2821).

Sea Turtle Protection Measures

Vessels supporting construction activities may encounter sea turtles in the vicinity of the material transport routes, dredging areas, and construction areas. Vessels operating in these areas will follow NMFS Vessel Strike Avoidance Measures and Reporting for Mariners (NMFS 2008) and Sea Turtle and Smalltooth Sawfish Construction Conditions (NMFS 2006) to limit the potential for adverse interactions with sea turtles. These conservation measures require construction and vessel operators to take the following steps:

- Vessel operators will be notified of the potential presence of ESA protected sea turtles in the project areas and instructed on the need to avoid collisions with sea turtles.
- Siltation barriers shall be made of material in which a sea turtle cannot become entangled. The barriers shall be properly secured, and be regularly monitored to avoid protected species entrapment.
- Vessel operators and crews shall maintain a vigilant watch for sea turtles to avoid striking sighted protected species.
- Vessels shall operate at “no wake/idle” speeds at all times while in the outfall construction area and while in water depths where the draft of the vessel provides less than a 4-foot clearance from the bottom. Vessels will preferentially follow deep-water routes (for example, marked channels) whenever possible.
- When sea turtles are sighted within 100 yards of active vessel movement or operations, the vessel operator shall attempt to maintain a distance of 50 yards or greater between the animal and the vessel whenever possible.
- When an animal is sighted in the vessel’s path or close to a moving vessel and when safety permits, the vessel operator shall reduce speed and shift the engine to neutral. Engines will not be reengaged until the animals are clear of the area.
- Vessel crews will report sightings of any injured or dead sea turtles immediately to the NMFS Southeast Regional Office at 727-824-5312.

Upland Work – Best Management Practices Strategy for Temporary Stormwater Management

Stormwater Pollution Prevention Plan

The stormwater pollution prevention plan (SWPPP) is prepared to meet National Pollutant Discharge Elimination System (NPDES) permit requirements for stormwater discharges from construction sites. The SWPPP will address the following:

- Planning and organization
- Site assessment

- BMP identification
- Implementation
- Evaluation and monitoring

Temporary Erosion and Sediment Control Plan

A temporary erosion and sediment control (TESC) plan is required to prevent erosive forces from damaging Project sites, adjacent properties and the environment. A TESC plan will be prepared and implemented to minimize and control pollution and erosion due to stormwater runoff. The TESC plan may be a component of the SWPPP.

Spill Prevention, Control and Countermeasure Plan

A spill prevention, control and countermeasure (SPCC) plan is prepared by the contractor to prevent and minimize spills that may contaminate soil or nearby waters.

Operations Plan

CPRA will develop an operation plan to guide overall operations of the MBSD. This will include standard and emergency procedures guiding operation of the diversion structure. The operations plan will guide how flow conditions are regulated and how any emergency closures or regular maintenance activities occur. The operations plan will include measures to prevent backflow of water from Barataria Basin to the Mississippi River during storm events and guide how flows through the diversion will change. Freshwater input from the Mississippi River has the potential affect species and habitats adapted to the current salinity and flow regime. The operations plan will guide how freshwater from the diversion is introduced to these habitats on an initial and ongoing basis.

Monitoring and Adaptive Management Plan (MAMP)

CPRA is developing a MAMP in association with the Project that will guide field monitoring of species, habitats and water quality considerations during operation of the MBSD. This plan will include monitoring efforts and management actions that may affect operations based on identified thresholds and planning processes.

Mitigation Measures

The Project is a component of the DWH PDARP and a priority of Louisiana's Comprehensive Master Plan for a Sustainable Coast (CPRA 2017a). Project components including the Beneficial Use Placement of dredged materials to support and maintain wetlands are examples of mitigation measures incorporated into Project design. Mitigation measures identified as conservation recommendations from the ESA consultation will be combined with mitigation measures resulting from the MAMP and other regulatory reviews into the Mitigation Plan for the Project.

2.3.7 Interdependent and Interrelated Actions

This section will discuss interdependent or interrelated actions or activities associated with the proposed Project, if any. These are actions that would not occur "but for" the proposed Project.

Installation of the MBSD will cross existing linear transportation infrastructure, utility, and drainage systems that serve the adjacent communities. These systems will need to be modified to accommodate the Project. In addition, Project construction will generate a large amount of excavated soil and sediment. This material may be repurposed for beneficial use placement at selected locations in Barataria Bay. The required infrastructure modifications and planned beneficial use of overburden and dredged material are referred to as "Auxiliary Activities and Structures" and are described below. See EIS Section 2.8 for additional details.

Specifically, the segments of state highway LA 23 and the New Orleans and Gulf Coast (NOGC) Railway crossing the proposed conveyance channel would need to be raised and relocated. In addition, linear public and private utilities located along the LA 23 corridor, including electric, water, communications, and cable lines will need to be relocated. These features will be temporarily relocated during Project construction and permanently replaced once the conveyance channel is complete.

Auxiliary Activities and Structure development will adhere to the same construction BMPs described above for the MBSD.

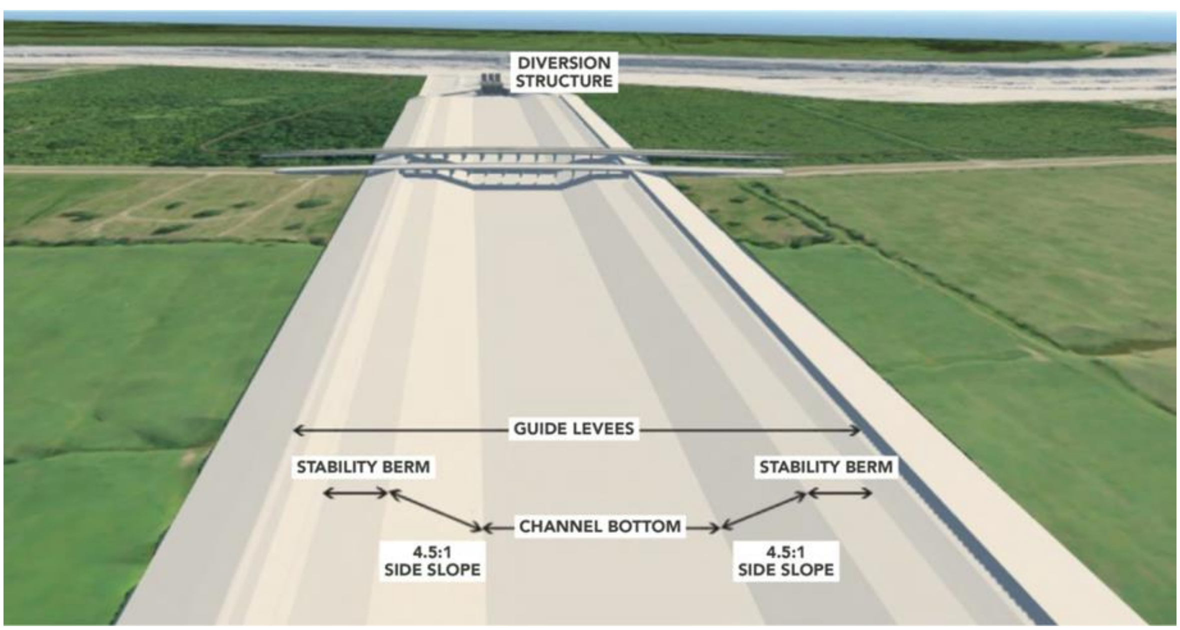


Figure 2.3.7-1. Conveyance Channel with Proposed Railway and Highway Bridges

Linear Infrastructure

<u>**New Orleans Gulf Coast Railway**</u>

The NOGC, a subsidiary of the Rio Grande Pacific Corporation, operates a 32-mile-long railroad that traverses the west bank of the Mississippi River immediately adjacent to the Project. The NOGC currently serves more than 20 switching and industrial customers who produce a variety of fishing, agricultural, petroleum, chemical, and steel products. The railroad line terminates approximately 1,500 feet south of the centerline of the proposed conveyance channel. NOGC plans to extend the rail line farther south pending future service agreements. Construction of the conveyance channel would require that a portion of the NOGC Railroad right-of-way be raised and relocated over the conveyance channel (Figure 2.3.2-2). The proposed railroad modifications include maintaining the existing railroad alignment, constructing a bridge over the proposed conveyance channel with a bottom elevation of 16.4 feet, and extending the track by 600 feet to comply with bridge approach design standards. Further details on railroad modifications may be found in the EIS Chapter 2.

The preliminary construction sequence for the railroad modification includes the following:

- Construct temporary marshalling track along the north conveyance channel levee.
- Remove portion of existing track crossing the conveyance channel.
- Install turnout at intersection and lockout mechanism to prevent trains from accessing removed track segment.
- Place embankment approaches on each side of the conveyance channel.
- Construct bridge spans following construction of the concrete conveyance channel.
- Install replacement bridge and approaches including ballast, track, and train bumping post or hill.
- Remove temporary track and turnout.

Railroad bridge and track construction will adhere to the BMPs for upland and inwater work described in Section 2.3.6. Preventative maintenance and inspection measures will follow typical intervals for similar railroad bridges.

The Project will coordinate with NOGC to ensure appropriate emergency response plan is in place for any incidents along the portion of the rail line crossing the conveyance channel to protect the Barataria Basin and Mississippi River from potential spills.

Highway Louisiana 23

State highway LA 23 is the principal transportation corridor for the parish and a designated hurricane evacuation route. Project construction will require raising and relocating the affected segment of LA 23 to a new bridge crossing the conveyance channel. The proposed construction footprint for the LA 23 Bridge is approximately 153 acres. The proposed bridge structure would have a length of 2,176 feet with at least 7 feet of clearance over the top of the conveyance channel floodwalls of 15.6 feet.

The LA 23 Bridge will be constructed using standard bridge construction techniques. The pilings supporting intermediate piers/bents within the conveyance channel may be installed prior to or after channel excavation, as determined by the contractor's preferred construction methods. Girders will be standard AASHTO-type precast and the deck will be cast in place. Pile-supported bridge approach segments will likely be precast concrete or steel.

The proposed sequence for preliminary construction sequence includes the following:

- Install the construction detour crossovers.
- Reduce and shift southbound traffic to shoulder; shift northbound traffic to southbound lanes.
- Place surcharge fill for ramps, levee road crossings, and relocated roadways.
- Construct flood walls on Louisiana Department of Transportation & Development (LADOTD) right-of-way.
- Construct LA 23 Bridge, 24-inch waterline relocation on bridge, and relocated highway with median barrier.
- Construct northbound ramps on both sides on the conveyance channel.
- Construct remaining segments of median barrier north and south of the conveyance channel.
- Shift LA 23 traffic to the bridge.
- Remove southbound LA 23 pavement.
- Construct remaining flood wall across LADOTD right-of-way.
- Complete southbound roadway tie-ins and southbound ramp connections and tie-ins to the haul roads.
- Place southbound roadway.

Highway and bridge construction will adhere to the BMPs for upland and inwater work described in Section 2.3.6. Preventative maintenance and inspection measures will follow typical intervals for similar highways and highway bridges as regulated by the Federal Highway Administration (FHA).

The Project will coordinate with LADOTD and FHA to ensure an appropriate emergency response plan is in place for any incidents along the portion of the highway crossing the conveyance channel to protect the Barataria Basin and Mississippi River from potential spills.

Mississippi River and New Orleans to Venice Levees

The MBSD Project will require tie-ins to the Mississippi River Levee (MRL). The U-frame intake structure is enclosed on both the north and south sides with inverted T-wall monoliths that will provide the tie-ins. Since the T-walls are within the open excavation for the U-frame and gated diversion structure, the nearest MRL T-walls will match their bottom elevations and step upward as they embed further into the levee. The design of this feature is currently being finalized.

The USACE is planning to move a segment of the New Orleans to Venice (NOV) levee landward, the existing back levee will remain on the current alignment.

The final configuration of the MBSD Project's conveyance channel levee will require closures of Timber Canal at approximate Station 113+50 and the NOV Back-Levee Canal at approximate Station 140+00.

Utilities

Several public and private facilities and utilities will be relocated as part of the Project. Currently linear power, communication, and water utilities run along the LA 23 corridor. These utilities will need to be modified to cross the MBSD. Water, fiber optic, and other utility improvements will be incorporated into the new LA 23 Bridge. In addition to utilities, several commercial pipelines cross the proposed conveyance channel corridor.

Specific utility improvements required for the Project are described in the following sections.

Power

Energy power transmission and distribution lines are currently located along the LA 23 corridor. The high-voltage transmission line is mounted on steel poles located on the west side of LA 23. The distribution lines are mounted on wooden poles along each side of the highway. Power transmission lines will be relocated to support the transmission tower improvements required to span the diversion channel. The distribution lines will be integrated into the new bridge structure.

The Project will coordinate with the power line owners to provide temporary service during construction.

Fiber Optic

AT&T Communications maintains fiber optic and copper telephone cables along the LA 23 right-of-way. CMA Communications maintains fiber optic and coaxial cables along the pipeline.

The Project will coordinate with the fiber optic line owners to provide temporary service during construction and restore service after Project is complete.

Water

Plaquemines Parish maintains 20-inch-diameter polyvinyl chloride (PVC) water line running along the west side of LA 23, a 16-inch water line running on the west side of LA 23, and an existing windmill/water well within the construction limits. Inframark Services owns a 16-inch-diameter line in the Project construction limits.

The Project will coordinate with the water owners to provide temporary service during construction and restore service after Project is complete.

Pipelines

Several pipelines cross the proposed conveyance channel. These include a 20-inch-diameter crude oil pipeline owned by Shell Pipeline Company, currently located on the flood side of the NOV Levee; a 12-inch-diameter natural gas line owned by High Point Gas Transmission; a 16-inch-diameter propylene line owned by Chalmette LA Liquids and Sulphur River Exploration; and a 12-inch-diameter gas pipeline owned by American Midstream Assets. The Shell pipeline will remain in its current alignment but will be lowered to a suitable depth to travel beneath the proposed conveyance channel. The remaining pipelines will be incorporated into the LA 23 bridge structure. The Project will coordinate with pipeline owners to create temporary bypasses to maintain service during construction.

Pipeline relocation activities will adhere to Project construction BMPs described in Section 2.3.6. The Project will also coordinate with US Department of Transportation (USDOT), LADOTD, and pipeline owners to ensure that an appropriate emergency response plan is in place for any incidents involving the conveyance channel in order to protect the Barataria Basin and Mississippi River from potential spills.

Drainage System

Project construction will bisect the existing drainage system; thus, to address interior drainage management needs in the area north of the diversion, construction of an inverted siphon/drop structure will occur. CPRA is considering using an inverted siphon or drop structure (located below the conveyance channel) to convey drainage from the northern drainage area to Wilkinson Pump Station. The 1,200-foot-long siphon will extend beyond the limits of the guide bank levees. The proposed construction limit for the inverted siphon/drop structure and other structural accommodations is about 215 acres and is within the existing construction footprint of the conveyance channel. The design and location of a siphon/drop structure is partially driven by the final location for the NOV levee and drainage design.

Beneficial Use Placement Areas

The proposed Project also includes beneficial use placement areas (BU areas) (Figure 2.3.2-1). These BU areas are intended to be used for placement of material excavated during Project construction on an as-needed basis. Material will be used at appropriate locations within 1 or both of the BU areas to create features that will allow excess sediments excavated during construction to be disposed of in a way to promote habitat improvements (such as wetland creation, wetland nourishment, shallow aquatic habitat, or other beneficial features such as ridges or terraces). The west BU area is approximately 442 acres and the east BU area is 1,729 acres.

These areas were chosen, in part, due to the general lack of existing oil and gas infrastructure in the vicinity and to minimize risk of interfering with the initial delta formation. Material excavated for construction of the conveyance channel and the OTF will, if suitable, first be used for construction of Project components. Any remaining dredged material would be used beneficially within a portion of 1 or both of the 2 identified proposed beneficial use areas. Because the exact type of material and quantities needed for construction are not yet known, the precise use of the material is unknown and cannot be quantified at this time.

2.4 Project Action Area

The "action area" for this Project is defined as: "all areas to be affected directly or indirectly by the Federal action and not merely the immediate area directly adjacent to the action [50 CFR §402.02]." The action area includes the proposed Project location and all surrounding areas where effects due to the sediment diversion may reasonably be expected to occur. The action area is contained within the Project Area described in Section 3.1.1 of the EIS.

The action area was developed by reviewing the direct and indirect impact mechanisms associated with the Project. These include construction activities associated with the Project as well as areas in Barataria Basin, Birdfoot Delta, and the Mississippi River Delta Basin potentially affected by Project operations. The extent of the action area in the Barataria Basin and Birdfoot Delta incorporates the limits of construction for the Project, areas (including portions of the Mississippi River) potentially affected by underwater or in-air noise associated with the Project, areas where dredging for site access may occur, and areas potentially affected by operations of the Project. The action area has been identified as the Barataria Basin, and the Birdfoot Delta, upland areas where construction activities will occur and the portions of the Mississippi River where construction activities are proposed in the immediate vicinity of Mississippi River Mile 60.7 (Figure 2.4-1).

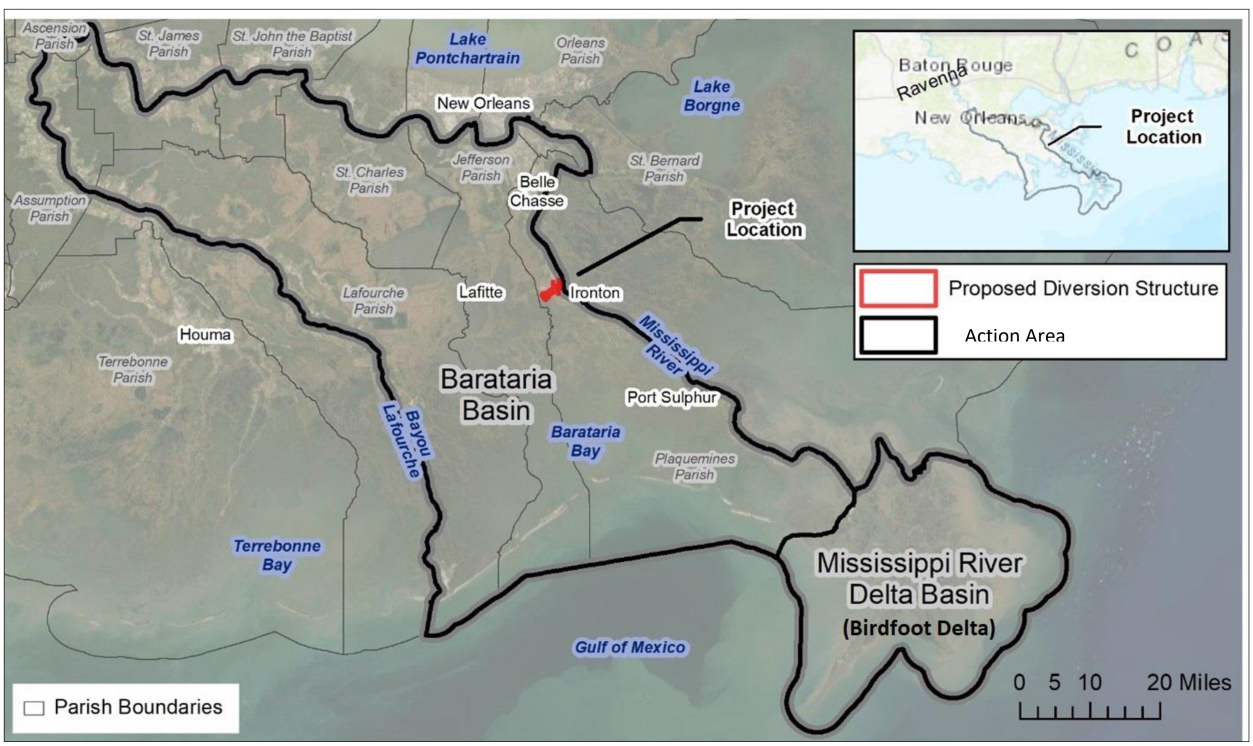


Figure 2.4-1. Project Action Area – Barataria Basin, Birdfoot Delta Basin and Proposed Diversion Structure.

3.0 STATUS OF THE SPECIES AND CRITICAL HABITAT

The following section includes information on ESA listed species that are potentially affected by the Project and a description of critical habitat if it is designated in the proposed action area for the Project.

Information in this section is consistent with the EIS, RP, and supporting information provided by CPRA. Where appropriate, this section will refer to sections of the EIS or RP for additional information.

3.1 Species List

Based on the compiled information from the Services (EIS Appendix A), the ESA listed species that may occur in the proposed action area are provided in Table 3.1-1 and are addressed in this BA. In cases when critical habitat has been designated or proposed for these species (EIS Appendix A), Table 3.1-1 identifies whether the critical habitat exists within the proposed action area for the Project. Effects to designated critical habitat physical or biological features (PBFs) are also analyzed in this document (Section 5.0).

Table 3.1-1. Special Status Species Potentially Affected by the Project

Listed Species	Federal Status	Listing Date	Critical Habitat
ESA Listed Fish			
Pallid sturgeon (*Scaphirhynchus albus*)	E	1990	None in action area
ESA Listed Birds			
Eastern black rail (*Laterallus jamaicensis jamaicensis*)	PT	2018	None designated
Piping plover (*Charadrius melodus*) - Atlantic Coast, Great Lakes, and Northern Great Plains population	T	1985	Coastal beaches and barrier islands*
Red knot (*Calidris canutus rufa*)	T	2015	None designated
ESA Listed Marine Mammals			
West Indian manatee (*Trichechus manatus*)	T	original listing 1967 ** downlisted 2017	None in action area
ESA Listed Turtles			
Green sea turtle (*Chelonia mydas*) - North Atlantic DPS - South Atlantic DPS	T	2016	None in action area
Hawksbill sea turtle (*Eretmochelys imbricata*)	E	1970**	None in action area
Kemp's ridley sea turtle (*Lepidochelys kempii*)	E	1970**	None designated
Leatherback sea turtle (*Dermochelys coriacea*)	E	1970**	None in action area
Loggerhead sea turtle (*Caretta caretta*), - Northwest Atlantic DPS	T	1978	Gulf of Mexico Sargassum***

Sources: NMFS 2018a, USFWS 2018a

* Critical habitat in the proposed action area occurs at West Belle Pass (Lafourche Parish), Elmer's Island, Grand Isle, and East Grand Terre (Jefferson Parish), and at South Pass (Plaquemines Parish)

** Listed under a law that preceded the Endangered Species Act.

*** Critical habitat in the proposed action area follows the 10-meter depth contour starting at the mouth of South Pass of the Mississippi River proceeding west and south to the boundary of the U.S. Exclusive Economic Zone.

Listed Species Identifiers: DPS = Distinct Population Segment

Status Identification: E = Endangered; T = Threatened; PT = Proposed Threatened

A number of ESA listed species that occur along the Gulf Coast are not known to regularly occur in Barataria Basin, the portion of the Mississippi River where the diversion is proposed, or the proposed action area. Therefore, the following species were not included in this analysis: Bryde's whale (*Balaenoptera edeni*), Fin whale (*Balaenoper physalus*), Sei whale (*Balaenoptera borealis*), Sperm whale (*Physeter macroocephal*), Oceanic whitetip shark (*Carcharhinus longimanus*), Giant manta ray (*Mobula birostris*), and Gulf sturgeon (*Acipsenser oxyrinchus desotoi*). Due to the lack of documented occurrence in the Project and action areas, the lack of suitable habitat in the proposed action area, and the lack of potential effects, the proposed action would have no effect on these species or their critical habitat, and they will not be discussed further.

3.2 Description of the Species

The following sections present general life history information and population status for the species listed in Table 3.1-1 above. The species life stages likely to occur in the action area, timing of occurrence, and habitat associations within the proposed Action are summarized in Table 4.6-1.

3.2.1 Pallid Sturgeon

The pallid sturgeon (*Scaphirhynchus albus*) is a bottom-dwelling freshwater fish found in the Missouri and Mississippi River drainages. They can weigh up to 80 pounds and reach lengths of 6 feet. This section summarizes best available data about the biology and current condition of pallid sturgeon throughout its range that are pertinent to evaluating the effects of the proposed action and its interrelated and interdependent actions.

General Life History

Habitats

Pallid sturgeon have a historical range that includes the Mississippi River downstream of the junction with the Missouri River, the Missouri River, the Yellowstone River, and its larger, turbid tributaries (e.g. the Tongue River and Powder River) (USFWS 2014). The present-day distribution is reduced and fragmented within this range, however the species has been documented in the lower Mississippi River in proximity to the Barataria Basin (LDWF 2014). The current known distribution of pallid sturgeon in the Mississippi River basin is shown in Figure 3.2.1-1 below.

Pallid sturgeon prefer large, free-flowing, turbid river habitats with moderate to swift currents, warm water, and diverse microhabitat conditions. They are commonly found at water depths ranging from 0.91 to 7.6 meters (3 feet to 25 feet) (LDWF 2014). They use a variety of main channel habitats in the lower Mississippi River, including natural and engineered features (Herrala et al, 2014). They appear to use submerged sand dunes for resting and/or feeding, as well as gravel dunes and flats (USFWS 2014a; Bramblett and White 200l; Hurley et al. 2004; Garvey et al. 2009; Koch et al. 2012).

Several studies have documented pallid sturgeon congregating near islands and dikes. These habitats are thought to provide a break in water velocity and an increased area of depositional substrates for foraging (Garvey et al. 2009, Koch et al. 2012). Increased use of protected areas around side channel and main channel islands has been noted in spring. Researchers have hypothesized that sturgeon are using these habitats as refugia during periods of increased flow (Garvey et al. 2009, Koch et al. 2012, Herrala et al. 2014). Recent telemetry monitoring of adult pallid sturgeon in the lower Mississippi River indicates use of most channel habitats, including dikes, revetment, islands, secondary channels, etc. (Kroboth et al. 2013, Herrala et al. 2014).

Islands and secondary channels are important for recruitment of larval sturgeon in the lower Mississippi River (Hartfield et al. 2013) and larval sturgeon are commonly associated with flooded sand bars in secondary channels and sand/gravel reefs in the main channel (Hartfield et al. 2013, Schramm et al. 2017). Pallid sturgeon are believed to spawn over gravel substrates like the closely related shovelnose sturgeon, but spawning has never been directly observed in this species (USFWS 1993, DeLonay et al. 2007, DeLonay et al. 2009).

Figure 3.2.1-1. Post-Development Map of Prominent Rivers in the Mississippi River Basin.

Bold line approximates current range of pallid sturgeon and includes both wild and hatchery-reared fish. Source: USFWS 2014; Data: National Pallid Sturgeon Database, U.S. Fish and Wildlife Service, Bismarck, North Dakota.

Hybridization

Recent studies have documented extensive hybridization between pallid sturgeon and shovelnose sturgeon in the lower Mississippi River (Coastal Plain Management Unit) (Heist et al. in litt. 2016, Kuhajda et al. in litt. 2016, Jordan et al. in prep. 2018). These studies also confirmed that small numbers of genetically pure pallid sturgeon continue to occupy the lower Mississippi River; however, genetic analysis is required for their accurate identification. There is currently no official Service policy for the protection of hybrids under the ESA, and the protection of hybrid progeny of endangered or threatened species is evaluated as necessary. The duration and significance of hybridization between pallid and shovelnose sturgeon is currently unknown, and it is not possible to visually distinguish pure pallid sturgeon from introgressed pallid sturgeon; therefore, for the purposes of management and consultation, we are considering all phenotypic pallid sturgeon as protected under the ESA.

Movement

As large river fish, pallid sturgeon are capable of moving long distances in search of favorable habitat or during spawning runs. Bramblett (1996) noted a maximum home range as large as 331 km (205 miles), with pallid sturgeon moving up to 21 km/day (13 miles/day). Pallid sturgeon, similar to other sturgeon, exhibit seasonal variation in movement patterns based upon increased water temperature and river discharge in the spring (Garvey et al. 2009, Blevins 2011). In the Mississippi River, the pallid sturgeon migrates from sandy substrates to gravel in May, possibly for spawning (Koch et al. 2012). Hoover et al. (2007) hypothesized that long-range movements during the spring may not just be associated with spawning but could also be associated with feeding. However, pallid sturgeon may remain sedentary, or remain in 1 area for much of the year, before migrating either upstream or downstream during spring (Garvey et al. 2009, Herrala et al. 2017). Pallid sturgeon have been found to have active movement patterns during both the day and night, but they move mostly during the day (Bramblett and White 2001). There have been no verified spawning areas located in the lower Mississippi River.

Much of the information about pallid sturgeon movement patterns comes from portions of the species range that may not be accurately representative of the lower Mississippi River population's behavior. However, general information about range and migratory behavior combined with regionally specific observations are useful for characterizing potential habitat use. Pallid sturgeon in the Atchafalaya River, part of the broader Coastal Plains Management Unit that includes the lower Mississippi River (USFWS 2014), begin displaying migratory behavior at water temperatures between 14 degrees Celsius (°C) and 21 °C (57.2 degrees Fahrenheit (°F) and 69.8 °F) and spring and early summer season. Movement patterns also varied between spawning versus non-spawning years. Migratory range varies between populations. Pallid sturgeon in the Yellowstone and Missouri rivers have an average home range of 78 km (48.8 miles), while sturgeon in the middle Mississippi River only have a home range of 34 km (21.2 miles). Most active periods of movement in the upper Missouri River were between March 20 and June 20 (Bramblett and White 2001). It has been speculated that because habitat in the Mississippi River is relatively uniform, large movements and home ranges may not be as beneficial, as fish are less likely to encounter new habitats.

Feeding

Benthic macroinvertebrates characteristic of river habitats are important dietary components for pallid sturgeons throughout their life history (Modde and Schmulbach 1977, Carlson et al. 1985). Invertebrates characteristic of lake and terrestrial habitats have also been sampled in pallid sturgeon stomachs, suggest that drifting invertebrates are likely also important forage organisms (Modde and Schmulbach 1977, Constant et al. 1997).

Data on earliest life stages are limited. In hatchery environments, exogenously feeding fry will consume brine shrimp, suggesting a likely diet of zooplankton and small invertebrates as their food base. In juvenile life stages, aquatic invertebrates dominate diet composition, with percent composition of fishes (mostly cyprinids) in their diet increasing in relation to their body size (Carlson and Pflieger 1981, Hoover et al. 2007, Gerrity et al. 2006, Grohs et al. 2009, Wanner 2006, French 2010). Between ages 4 and 5, pallid sturgeon have been observed to shift their diet from predominantly invertebrates to fishes (Kallemeyn 1983, Carlson et al. 1985, Hoover et al. 2007, Grohs et al. 2009). In a study of pallid sturgeon in the middle and lower Mississippi River, fish were a common dietary component and were represented primarily by Cyprinidae, Sciaenidae, and Clupeidae (Hoover et al. 2007). Other important dietary items for pallid sturgeon in the Mississippi River were larval Hydropsychidae (lnsecta: Trichoptera), Ephemeridae (lnsecta: Ephemeroptera), and Chironomidae (lnsecta: Diptera) (Hoover et al. 2007). Pallid sturgeon diet varies depending on season and location, and these differences probably are related to prey availability (Hoover et al. 2007). In a Mississippi River dietary study, Trichoptera and Ephemeroptera were consumed in greater quantities in winter months in the lower Mississippi River, while the opposite trend was observed in the middle Mississippi River (Hoover et al. 2007). Hoover et al. (2007) also found that in both the middle Mississippi River and the lower Mississippi River, dietary richness is greatest in winter months.

Species Tolerances to Selected Stressors

A description of pallid sturgeon occurrence in the action area by life stage is provided in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Disturbance and Habitat Exclusion: The Missouri River dams also are believed to have adversely affected pallid sturgeon by blocking migration routes and fragmenting habitats (USFWS 2014).

Turbidity and Silty Substrate: Early results in culturing pallid sturgeon indicate that sturgeon larvae will not survive in a silty substrate. In 1998, most of the larval sturgeon held in tanks at Gavins Point National Fish Hatchery, experienced high mortality when the water supply contained a large amount of silt which settled on the bottom of the tanks. Migration routes to spawning sites on the lower Yellowstone River have been fragmented by low-head dams used for water supply intakes. Such habitat fragmentation has forced pallid sturgeon to spawn closer to reservoir habitats and reduced the distance larval sturgeon can drift after hatching.

Entrainment: Another issue that is negatively impacting pallid sturgeon throughout its range is entrainment. The loss of pallid sturgeon associated with water intake structures has not been accurately quantified, though the USEPA published final regulations on Cooling Water Intake Structures for Existing Facilities per requirements of §316(b) of the Clean Water Act to limit the potential take of pallid sturgeon at these structures.

Population Status

Pallid sturgeon were listed as federally endangered in 1990. A total of 279 different pallid sturgeons were collected from the Mississippi River (below its confluence with the Missouri River) between 1990 and 2004 (USFWS 2013a). As few as 6,000 to as many as 21,000 pallid sturgeon may still exist throughout its range (Krentz et al. 2004). The lower Mississippi River population is poorly documented and likely low in abundance (Duffy et al. 1996). To date, more than 1,100 pallid sturgeon have been captured in the Coastal Plain Management Unit which includes the lower Mississippi River extending from the confluence of the Ohio River in Illinois, to the Gulf of Mexico, Louisiana (Kilgore et al. 2007). Pallid sturgeon and shovelnose sturgeon co-occur in the lower Mississippi River at abundance ratios ranging from 1:6 to 1:30 depending upon river reach, and 1:6 in the Atchafalaya River (Kilgore et al. 2007). There are only 2 captures of pallid sturgeon between river miles 33 and 85 where the Corps of Engineers collected 2 young-of-year *Scaphirhynchus* sturgeon with a trawl in the lower Mississippi River in November 2016 (USACE 2017).

3.2.2 Eastern Black Rail

The eastern black rail (*Laterallus jamaicensis jamaicensis*) is a small, secretive marsh bird that inhabits both freshwater and saltwater marshes. This section summarizes best available data about biology and condition of this subspecies of black rail.

General Life History

<u>Habitats</u>

The eastern black rail, 1 of 4 subspecies of black rail, is broadly distributed, living in salt- and freshwater marshes in portions of the United States, Central America, and South America. Partially migratory, the eastern subspecies winters in the southern part of its breeding range.

Eastern black rail habitat includes both tidally or non-tidally influenced areas, and ranges in salinity from salt to brackish to fresh. Tidal height and volume vary greatly between the Atlantic and Gulf coasts and contribute to differences in salt marsh cover plants in the bird's habitat (USFWS 2018). The black rail is exceedingly elusive, making accurate assessment of its range and habits difficult. Nesting and wintering habitats include high marsh areas (salt, brackish, and freshwater) with infrequent flooding, including pond borders, wet meadows, and grassy swamps (Eddleman et al. 1994). The subspecies' range extends from North America to South America, but populations are relatively small and highly localized.

Along portions of the Gulf Coast, eastern black rails can be found in higher elevation wetland zones with some shrubby vegetation. Impounded and unimpounded intermediate marshes (marshes closer to high elevation areas) also provide habitat for the subspecies. Inland coastal prairies and associated wetlands may also provide habitat for the bird but are largely uninvestigated (USFWS 2018). In Louisiana, black rails are known to winter in the marshes of Cameron and Vermilion parishes, outside of the proposed action area. However, given their elusive nature, the species is considered to be potentially present in all high marshes of coastal Louisiana.

Between 2010 and 2017, there were no credible records for black rail in Tennessee, Alabama, or Mississippi, and only a small number from Louisiana and Georgia. The 2016 population estimate for Louisiana was 0 to 10 breeding pairs compared to a Southeast Region population estimate of 400 to 1,200 breeding pairs (Watts 2016). Texas, Florida, South Carolina, and North Carolina contain 89% of all historical observations in the Southeast (Watts 2016). Other states are considered to be on the peripheries of known breeding areas. Of the historical stronghold states (which do not include Louisiana), North Carolina presently shows a severe decline in the number of occupied sites while South Carolina shows a limited distribution. This leaves Texas and Florida as present strongholds for the southeastern coastal US region. Region-wide, recent observations show poor presence inland and an overall widespread reduction in utilized sites across coastal habitats (USFWS 2018). Distributions of eastern black rail are shown in Figure 3.2.2-1 below.

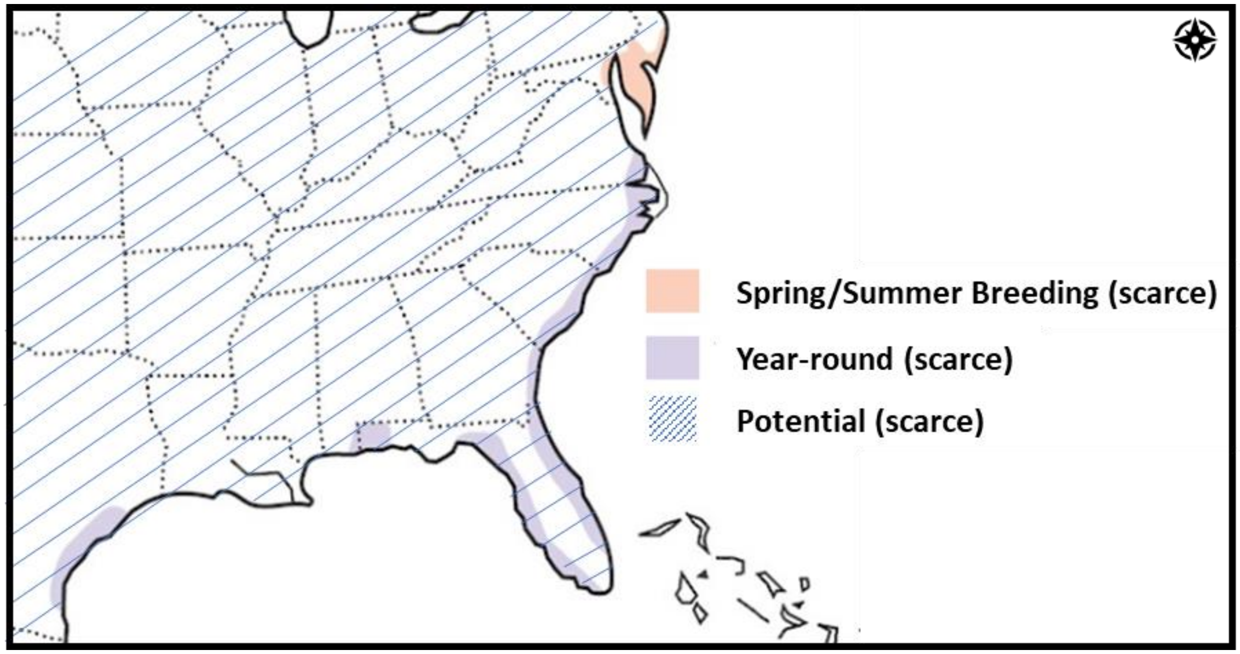


Figure 3.2.2-1. Current Range of the Eastern Black Rail in the SW US (2011 to present) (Sources: Eddleman et al. 1994, USFWS 2018)

Movement

Partially migratory, the eastern subspecies winters in the southern part of its breeding range. Along the Gulf Coast, however, eastern black rails can be found year-round, with a potential year-round distribution in the lower Mississippi River and the Mid-Barataria area (USFWS 2018). In Louisiana, black rails are known to winter in the marshes of Cameron and Vermilion parishes.

Feeding

Their bill shape suggests generalized feeding methods such as gleaning or pecking at individual items; thus they likely rely on sight for finding food. Their diet appears to consist of small aquatic and terrestrial invertebrates, as well as small seeds. Foraging most likely occurs on or near the edges of stands of emerging vegetation—both above and below the high-water line (USFWS 2018).

Species Tolerances to Selected Stressors

Eastern black rail occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to the species include loss and degradation of habitat, and invasion by non-native plant species (NatureServe 2017). Alterations to hydrology, sediment and nutrient transport, and salinity can affect the composition of wetland habitats used by the eastern black rail. For example, navigation channels and their management have had extensive impacts to tidal wetlands by modifying the vegetation community and increasing the frequency of extreme high tide or high flow events on tidal wetlands (USFWS 2018).

Population Status

Recent black rail surveys led by the Audubon Society have focused survey efforts on Cameron and Vermilion Parishes where most of the documented high-quality habitat occurs. Anecdotal reports suggest there may be black rails on Grand Isle and Elmer's Island; however, surveys have not documented black rails there since the Deepwater Horizon oil spill. The old Chenière Caminada Island may contain black rail habitat, but no surveys have occurred there (E. Johnson, Director of Bird Conservation for Audubon Louisiana, Pers. Com. 2019).

USFWS initiated the ESA status review of eastern black rail in 1994. The black rail is protected under the Migratory Bird Treaty Act, and is on the Louisiana Natural Heritage Program list of rare species in Louisiana (USFWS 2013b, USFWS 2018). After examining the eastern black rail's past, present and future conditions, the USFWS determined the subspecies meets the definition of threatened, and is proposing to list it as threatened under the ESA (USFWS 2018). Some populations of the eastern black rail along the Atlantic coast have dropped by as much as 90%, impairing the ability of the subspecies to respond to natural and anthropogenic threats and stressors in its environment (USFWS 2015).

3.2.3 Piping Plover

This section summarizes best available data about biology and condition of piping plover (*Charadrius melodus*). The piping plover is a small, stocky migratory shorebird that breeds in the northern United States and Canada and winters in the southern United States and some Caribbean Islands.

General Life History

<u>Habitats</u>

The USFWS lists 3 distinct breeding populations of piping plover: the Atlantic Coast subspecies (*C. m. melodus*) and the Northern Great Plains DPS and Great Lakes DPS populations of the Interior subspecies (*C. m. circumcinctus*; see Figure 3.2.3-1). Each population breeds in its distinct region in sparsely vegetated upper dunes, high sandy beaches and shorelines, and, in some regions, beaches with gravel or scattered cobble. In both breeding and wintering ranges, piping plover forage along shorelines, intertidal flats, mudflats, or sandflats where the birds glean various invertebrates (for example, worms, fly larvae, beetles, crustaceans, mollusks) from the surface, or occasionally probe for these items in sand or mud (NatureServe 2017). Birds observed in Louisiana are typically from either the Northern Great Plains or Great Lakes populations; however, individuals from all 3 breeding populations may be present in Louisiana (see Figure 3.2.3-1).

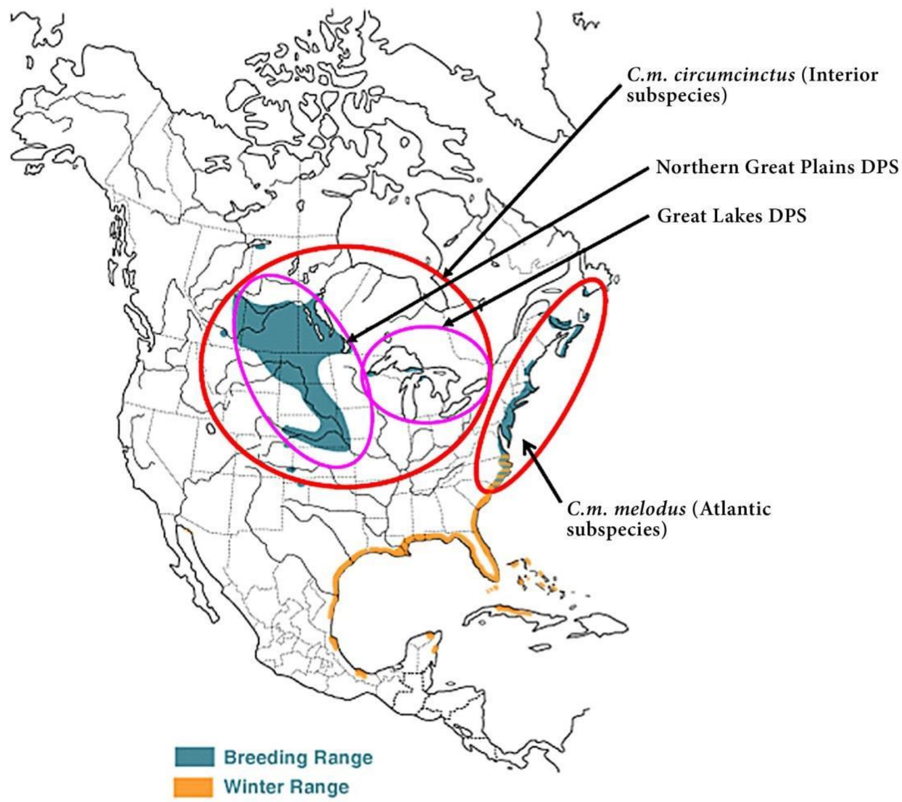


Figure 3.2.3-1. Distribution and Range* of Piping Plover—Great Lakes DPS and Northern Great Plains DPS as Delineated in the USFWS 2009 5-Year Review.

(Source: USFWS 2015). Base map from Elliott-Smith and Haig 2004, used by permission of Birds of North America Online). * Conceptual presentation of subspecies and DPS ranges are not intended to convey precise boundaries.

Wintering piping plovers utilize a mosaic of habitat patches and move among these patches in response to local weather and tidal conditions (Nicholls and Baldassarre 1990a, Nicholls and Baldassarre 1990b, Drake et al. 2001, Cohen et al. 2008). Preferred coastal habitats include sand spits, small islands, tidal flats, shoals (usually flood tidal deltas), and sandbars that are often associated with inlets (Nicholls and Baldassarre 1990b, Harrington 2008, Addison 2012). Sandy mud flats, ephemeral pools, seasonally emergent seagrass beds, mud/sand flats with scattered oysters, and over-wash fans are considered primary foraging habitats (Nicholls and Baldassarre 1990b, Cohen et al. 2008, USFWS 2015). Several studies identified wrack lines (organic material including seaweed, seashells, driftwood, and other materials deposited on beaches by tidal action) as an important component of roosting habitat for non-breeding piping plovers (USFWS 2015)

Movement

Piping plovers breed in the northern United States and Canada and winter in the southern United States and some Caribbean Islands (USFWS 2015). Piping plovers spend up to 10 months of their annual cycle on their migration and winter grounds, typically from July 15 through May 15 (Elliott-Smith and Haig 2004, Noel et al. 2007, Stucker et al. 2010). Southward migration from the breeding grounds primarily occurs from July to September, with the majority of birds initiating migration by the end of August (USFWS 1996). Piping plovers depart the wintering grounds as early as mid-February and as late as mid-May, with peak migration in March (Haig 1992). Potential exists for piping plovers to occur infrequently during migration within mudflats and estuarine habitat in the Barataria Basin, although it is not their preferred habitat. Wintering piping plovers may be present in the proposed action area for 8 to 10 months per year.

Piping plovers exhibit a high degree of fidelity to wintering areas, which often encompass several relatively nearby sites (Drake et al. 2001, Noel and Chandler 2008, Stucker et al. 2010). Gratto-Trevor et al. (2012) found little movement between or among regions, and reported that 97% of the birds they surveyed remained in the same region, often at the same beach. Only 6 of 259 banded piping plovers were observed more than once per winter moving across boundaries of 7 U.S. regions. Of 216 birds observed in multiple years, only 8 changed regions between years, and several of these shifts were associated with late summer or early spring migration periods (Gratto-Trevor et al. 2012). Although many sites on the northern Gulf Coast of Texas and in Louisiana were affected by hurricanes after the 2008 fall migration, all 17 birds known to have wintered in these areas before the hurricanes have been re-sighted near their original areas (Gratto-Trevor et al. 2012).

Feeding

Piping plovers primarily forage on macroinvertebrates, with the majority of their diet consisting of polychaete worms, insects, and other arthropods (USFWS 2015). Piping plovers are characterized as coastal beach gleaners that select insects and crustaceans from substrate during their non-breeding period (De Graaf et al. 1985).

Species Tolerances to Selected Stressors

Piping plover occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

The wide, flat, sparsely vegetated barrier beaches, spits, sandbars, and bayside flats preferred by piping plovers in the United States are formed and maintained by natural forces. In Louisiana, coastal shorelines used by wintering plovers are being lost due to natural processes as well as development. Dredging of inlets can affect spit formation adjacent to inlets, as well as ebb and flood tidal shoal formation. Jetties stabilize inlets and cause island widening and subsequent vegetation growth on the updrift inlet shores; they also cause island narrowing and/or erosion on the downdrift inlet shores. Seawalls and revetments restrict natural island movement and exacerbate erosion. Although dredge and fill projects that place sand on beaches and dunes may restore lost or degraded habitat in some areas, in other areas these projects may degrade habitat quality by altering the natural sediment composition, depressing the invertebrate prey base, hindering habitat migration with sea level rise, and replacing the natural habitats of the dune-beach-nearshore system with artificial geomorphology. CPRA has completed several barrier island projects with restoration of piping plover habitat as 1 of the goals. Construction of any of these projects during months when piping plovers are present also causes disturbance that disrupts the birds' foraging and roosting behaviors. Threats to piping plover habitat will likely be exacerbated by accelerating sea level rise. Anthropogenic responses to sea level rise and associated increases in erosion rates include shoreline hardening and stabilization, which prevents the natural migration of the beach and causes loss of piping plover habitat.

Plovers are also susceptible to predation by shoreline predators including domestic pets, coyotes, and raccoons. Many of the plover predators are associated with urbanization in and around plover habitat.

Population Status

Total numbers of piping plover have fluctuated over time, with some areas increasing while other areas showed declines. Regional and local fluctuations may reflect changes in the quantity and quality of suitable foraging and roosting habitat, which vary in response to natural coastal formation processes as well as anthropogenic habitat changes (for example, inlet relocation, dredging of shoals and spits) (USFWS 2015). Studies of wintering plovers suggest that there may be high site fidelity from winter to winter and that plovers may use relatively small winter home ranges (Noel and Chandler 2008).

Population viability analyses conducted for piping plovers (Ryan et al. 1993, Melvin and Gibbs 1996, Plissner and Haig 2000, Wemmer et al. 2001, Larson et al. 2002, Calvert et al. 2006, Brault 2007, McGowan and Ryan 2009) all demonstrate the sensitivity of extinction risk in response to small changes in adult and/or juvenile survival rates. These results further emphasize the importance of non-breeding habitat to species recovery (Roche et al. 2010). Poor overwintering and stop-over habitat quality has been shown to have a negative effect on survival of other shorebird species, which has contributed to breeding population declines (Gill et al. 2001, Baker et al. 2004, Morrison and Hobson 2004) and is likely also impacting piping plover.

In January 1986, the piping plover was listed under the provisions of the ESA as endangered in the Great Lakes watershed of both the United States and Canada, and as threatened in the remainder of its range (USFWS 1985). All piping plovers are classified as threatened on their shared migration and wintering range outside the watershed of the Great Lakes. However, USFW biological opinions prepared under section 7 of the ESA acknowledge that activities affecting wintering and migrating plovers differentially influence the survival and recovery of the 3 breeding populations.

3.2.4 Red Knot

This section summarizes best available data about biology and condition of the *rufa* subspecies of red knot (*Calidris canutus rufa*), hereafter referred to as red knot. The red knot is a highly migratory shorebird species; red knots breeding in the Canadian Arctic migrate from breeding grounds in the Canadian Arctic to wintering grounds that include the Gulf Coast, southeast United States, and South America.

General Life History

Habitats

Lowery (1974) indicated that red knots may be found in Louisiana year-round, but they are substantially less common from mid-June through July when the bulk of the population is breeding in the high arctic. Summer birds are likely non-breeders and may be mostly subadults. Outside of breeding season, the red knot is found primarily in intertidal, marine habitats, especially near coastal inlets, estuaries, and bays (Baker et al. 2013); within the proposed action area, this habitat may be present along beaches and barrier island habitat along the Gulf of Mexico (NatureServe 2017). The Audubon Society evaluated red knot surveys and observations in the Gulf and noted that substantially all observations are along barrier islands, with consistent observations on Grand Isle (Johnson 2013). The species is considered rare to uncommon along the Louisiana coast and barrier islands, although it has been a regular visitor to Grande Isle (Fontenot and DeMay 2014). Red knot wintering and migration stop-over areas are shown below in Figure 3.2.4-1.

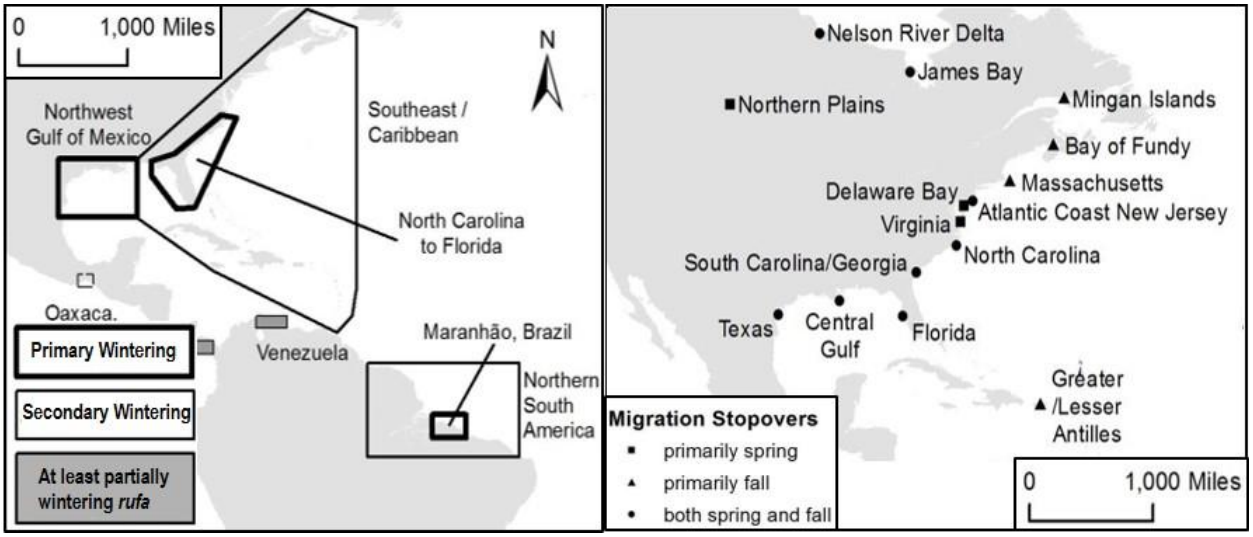


Figure 3.2.4-1. Red Knot Wintering Areas (left) and Migration Stop-Over Areas (right).
(Source: USFWS 2014).

Movement

The red knot migrates from breeding grounds in the Canadian Arctic to wintering grounds along the Gulf Coast, southeast United States, and farther south. Breeding season occurs from late May until early August, and most birds depart the northern breeding areas by mid-August. Departure from the breeding grounds begins in mid-July and continues through August. Red knots tend to migrate in single-species flocks with departures typically occurring in the few hours before twilight on sunny days. Based on the duration and distance of migratory flight segments estimated from geolocator results, red knots are inferred to migrate during both day and night (Normandeau Associates, Inc. 2011). Red knots also show some fidelity to particular migration staging areas between years (Harrington 2001, Duerr et al. 2011). Red knots are potentially present in the proposed action area is from August to May.

Feeding

The red knot is a specialized molluscivore, eating hard-shelled mollusks, sometimes supplemented with easily accessed softer invertebrate prey, such as shrimp- and crab-like organisms, marine worms, and horseshoe crab (*Limulus polyphemus*) eggs (Harrington 2001, Piersma and van Gils 2011, USFWS 2014b). From studies of other subspecies, Zwarts and Blomert (1992) concluded that the red knot cannot ingest prey with a circumference greater than 30 mm (1.2 inches). Foraging activity is largely dictated by tidal conditions, as the red knot rarely wades in water more than 2 cm to 3 cm (0.8 inch to 1.2 inches) deep (Harrington 2001). Due to bill morphology, the red knot is limited to foraging on only shallow-buried prey, within the top 2 cm to 3 cm (0.8 inch to 1.2 inches) of sediment (Zwarts and Blomert 1992, Gerasimov 2009).

In non-breeding habitats, the primary prey of the red knot include blue mussel (*Mytilus edulis*) spat (juveniles), *Donax* and *Darina* clams, snails (*Littorina* spp.), and other mollusks, with polycheate worms, insect larvae, and crustaceans also eaten in some locations (USFWS 2015). A prominent departure from typical prey items occurs each spring when red knots feed on the eggs of horseshoe crabs, particularly during the key migration stop-over within the Delaware Bay, which serves as the principal spring migration staging area for the red knot because of the availability of horseshoe crab eggs (Morrison and Harrington 1992, Harrington 1996, Harrington 2001, Clark et al. 2009, USFWS 2014b). Horseshoe crab eggs provide a superabundant source of easily digestible food for migrating shorebirds. Red knots and other shorebirds that are long-distance migrants must take advantage of seasonally abundant food resources at intermediate stop-overs to build up fat reserves for the next nonstop, long-distance flight (Clark et al. 1993). Although foraging red knots can be found widely distributed in small numbers within suitable habitats during the migration period, birds tend to concentrate in those areas where abundant food resources are consistently available from year to year (USFWS 2015).

Species Tolerances to Selected Stressors

Red knot occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

After assessing the best scientific and commercial data available regarding past, present, and future threats to the red knot, USFWS identified that the primary threats to the red knot are habitat loss and degradation due to sea level rise, shoreline stabilization, and Arctic warming as well as reduced food availability and asynchronies in the annual cycle. Other threats are moderate in comparison to the primary threats; however, cumulatively, they could become significant when working in concert with the primary threats if they further reduce the species' resiliency. Such secondary threats include hunting, predation, human disturbance, harmful algal blooms, oil spills, and wind energy development, all of which affect red knots across their range. Although conservation efforts (for example, management of the horseshoe crab population and regulatory mechanisms for the species and its habitat) are being implemented in many areas of the red knot's range to reduce some threats, significant risks to the subspecies remain (USFWS 2015).

The comprehensive list of threats to red knots includes the following: climate change, reduced food availability, asynchronies ("mismatches") in the red knot's annual cycle (particularly with horseshoe crab breeding), shoreline stabilization and coastal development, hard structures, mechanical sediment transport, wrack removal and beach cleaning, invasive vegetation, aquaculture and agriculture, hunting, scientific study, disease, predation, human disturbance, harmful algal blooms, environmental contaminants, oil spills, and wind energy development.

Population Status

Two recent winter estimates are available for the central Gulf of Mexico. During the International Piping Plover Census in 2006 and 2011, 250 to 500 red knots were counted from Alabama to Louisiana. Christmas Bird Count data suggest that the Gulf Coast population is declining at 2.3% per year, representing a 60.6% decline over the 40-year period of record (Johnson 2013). During work related to the DWH oil spill, an estimated 900 red knots were reported from the Florida Panhandle to Mississippi. Except for localized areas, there have been no long-term systematic surveys of red knots in Texas or Louisiana, and no information is available about the number of red knots that winter in northeastern Mexico. From survey work in the 1970s, Morrison and Harrington (1992) reported peak winter counts of 120 red knots in Louisiana and 1,440 in Texas, although numbers in Texas between December and February were typically in the range of 100 to 300 birds. Records compiled by Skagen et al. (1999) report a single peak count of 2,500 red knots along the coast of Louisiana (on Grand Isle, specifically), from between January and June over the period 1980 to 1996, but this figure could include spring migrants (Johnson 2013). There are no current estimates for the size of the Northwest Gulf of Mexico wintering group as a whole (Mexico to Louisiana). The best available current estimates for portions of this wintering region are about 2,000 in Texas (Niles 2012) or approximately 3,000 in Texas and Louisiana, with about half in each state and movement between them.

The USFWS listed the red knot as threatened in January 2015, primarily due to its dependence on horseshoe crab populations of the Delaware Bay region, which have been declining (USFWS 2014b).

3.2.5 West Indian Manatee

The West Indian manatee is a large gray or brown marine mammal in the order Sirenia. Adults average approximately 3 meters (10 feet) in length and weigh up to 2,200 pounds. They have no hind limbs and their forelimbs are flippers. This section summarizes best available data about biology and condition of West Indian manatee.

General Life History

Habitats

The West Indian manatee is primarily tropical, and is found along the Atlantic basin, utilizing inland freshwater habitats as well as coastal estuarine habitats such as tidal rivers and streams, springs, salt marshes, lagoons, and canals (UNEP 2010). The West Indian manatee may occur in coastal and inland waters from Massachusetts to Brazil, although sightings are rare north of the Carolinas (UNEP 2010), including along the entire Gulf Coast (USFWS 2015). Throughout their range, they utilize fresh, brackish, and marine environments. Manatees are typically found in water depths between 1.5 meters to 6.1 meters (5 feet and 20 feet). Manatees are tolerant of brackish and marine environments only if they have access to fresh water regularly (Fertl et al. 2005). Manatees may use the ocean for transits between thermal refugia and feeding areas and have been found more than 4.8 km (3 miles) off the Florida Gulf Coast (Powell and Rathbun 1984). Preferred habitats include areas near the shore featuring underwater vegetation like seagrass and eelgrass (USFWS 2008). Temperature is the dominant factor determining their range, and they respond to cold weather (less than 68 °F) by moving to warmer waters, which may be associated with natural springs and/or industrial areas such as power plants (USFWS 2008). Manatee observations in Louisiana tend to be reported during the summer months and these may reflect manatees that winter in Mexico (Powell and Rathbun 1984) and/or be strays from the Florida or Mexico populations (Fertl et al. 2005). Hurricanes and major storms may affect manatee distribution, as many sightings west of Florida have occurred shortly after hurricanes or tropical storms entered the Gulf of Mexico (Fertl et al. 2005). Distribution of West Indian manatees in the U.S. Southeast is shown in Figure 3.2.5-1.

Figure 3.2.5-1. Distribution of the West Indian Manatee in United States Based on Aerial Surveys, Boat Surveys, Interviews and Documented Sightings. The dark shading indicates year-round distribution, while the light shading indicates seasonal or occasional occurrence. (Source: UNEP 2010)

Movement

The West Indian manatee is generally restricted during winter to inland and coastal waters of the Florida panhandle (Laist and Reynolds 2005, Laist et al. 2013, USFWS 2014), but exhibits seasonal migration and greater dispersal during summer months and are periodically observed. Manatees may migrate during periods of mild weather or mild temperatures and may use warm-water refuges along their migratory routes during both the early spring and late fall (Reid et al. 1991).

Feeding

Sirenians are unique among marine mammals in that they are aquatic herbivores. In addition, they are hindgut fermenters (or digesters), which means that they spend most of the day foraging (Reynolds and Rommel 1996, Reynolds and Marshall in press). As the West Indian manatees move among riverine, estuarine, and marine environments, they consume many plant species present in each of those habitats, including non-native water hyacinths (*Eichhornia crassipes*) and hydrilla, along with native aquatic plants such as eelgrass (*Vallisneria* spp.). They prefer submergent aquatic vegetation (SAV) such as turtle grass (*Thalassia testudinum*) and manatee grass (*Syringodium filiforme*), but will feed on floating and emergent plants as well (Reynolds 1977, Jiménez 1999, Riquelme et al. 2006). Manatees also require fresh water for drinking (UNEP 2010).

Although primarily herbivorous, manatees will occasionally feed on fish and consume a variety of invertebrates, including bivalves, snails, amphipods, isopods, shrimp, crabs, and tunicates, found in the roots and foliage of macrophytes (Hartman 1979, Reynolds 1977, Best 1981). In addition to consuming vascular plants, manatees feed on freshwater algae including Enteromorpha, Oscillatoria, and Navicula and the marine algae *Ulva lactuta* and *Caulerpa prolifera* (Mignucci-Giannoni and Beck 1998, UNEP 2010).

Species Tolerances to Selected Stressors

A description of West Indian manatee occurrence in the action area by life stage is provided in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to the species include vessel strikes (direct impact and/or propeller), entrapment and/or crushing in water control structures, entanglement in fishing and crab pot lines, pollution, human disturbance, habitat degradation and loss, hunting, exposure to cold, loss of warm-water refuge, storm events, and exposure to red tide (USFWS 2008, UNEP 2010). Direct human causes (hunting, disturbance, vessel strikes, etc.) are estimated to result in about 99 manatee mortalities per year (USFWS 2014a).

Although hunting of manatees is illegal, they are hunted in some areas (mostly outside of the United States) for meat, oil, amulets, and other products and, on a more restricted basis, as a socio-cultural activity. Although threats due to hunting are diminishing in some areas, all other threats appear to be increasing in most areas. Pollution from agriculture and mining is consistently noted in reports on threats to manatees in Central American countries and may be affecting them in other areas. Manatee deaths as a result of boat strikes have been documented in places such as Florida, Belize, Colombia, Costa Rica, and Venezuela (UNEP 2010).

Population Status

The West Indian manatee (*Trichechus manatus*) was listed as endangered throughout its range for both the Florida and Antillean subspecies on March 11, 1967, and received federal protection with the passage of the ESA in 1973. The West Indian manatee was downlisted from endangered to threatened in 2017 due to increases in manatee populations and improvements in habitat (42 FR 47840) (USFWS 2017c). Critical habitat was designated in 1976, 1994, 1998, 2002, and 2003 for the Florida subspecies.

There are no robust estimates of total population size for this species (USFWS 2014a); studies have reported an abundance ranging from 5,076 (based on a single survey of warm-water refuges) to 6,350 manatees (based on models) (Laist et al. 2013, Martin et al. 2015). Within Louisiana waters, there were only 121 reported sightings of the West Indian manatee over the course of 14 years (between 1990 and 2004), and this total did not account for potential repeat sightings of individuals (Fertl et al. 2005). Louisiana accounts for 39% of the records west of Florida (Fertl et al. 2005). These limited data suggest that this species could be present within the proposed action area, but likely only as a transient visitor (particularly during the warmer months), and not a resident species. Observations tend to be individuals or a cow/calf pairing. Furthermore, some reports may be of the same individual detected multiple times. The most likely origins of manatees occurring along the northern Gulf Coast are the wintering populations from southwest Florida or Mexico (Fertl et al. 2005).

3.2.6 Green Sea Turtle

Green sea turtles (*Chelonia mydas*) are the largest of the hard-shelled turtles, with only leatherback sea turtles surpassing them in size (Witherington et al. 2006b, Prichard 2010). This section summarizes best available data about biology and condition of green sea turtle.

General Life History

<u>Habitats</u>

The North Atlantic DPS of green sea turtles, which is listed as threatened, is distributed throughout inshore and nearshore waters from Texas to Massachusetts, although most nesting occurs on Florida's southeast coast (NOAA 2018a). With the exception of post-hatchlings, green sea turtles live in nearshore tropical and subtropical waters as well as bays and lagoons. They have specific foraging grounds and may make large migrations between these forage sites and natal beaches for nesting (Hays et al. 2001). After emergence, hatchlings swim to offshore areas where they remain pelagic for several years. Once the juveniles reach a certain age/size range, they leave the open ocean habitat and travel to nearshore foraging grounds (NOAA 2018a).

Green sea turtles nest on sandy beaches of mainland shores, barrier islands, coral islands, and volcanic islands in more than 80 countries worldwide (Hirth 1997). The complete nesting range of North Atlantic DPS green sea turtles within the southeastern United States includes sandy beaches between Texas and North Carolina, as well as Puerto Rico (Dow et al. 2007, NMFS and USFWS 1991). The vast majority of green sea turtle nesting within the southeastern United States occurs in Florida, outside of the proposed action area (Johnson and Ehrhart 1994, Meylan et al. 1995). Principal U.S. nesting areas for green sea turtles are in eastern Florida, predominantly Brevard County south through Broward County.

The 2 largest nesting populations are found at Tortuguero, on the Caribbean coast of Costa Rica (part of the North Atlantic DPS), and Raine Island, on the Pacific coast of Australia along the Great Barrier Reef. There are no known nesting sites within the proposed action area.

In U.S. Atlantic and Gulf of Mexico waters, green sea turtles are distributed throughout inshore and nearshore waters from Texas to Massachusetts. Within U.S. waters individuals from both the North and South Atlantic DPSs can be found on foraging grounds.

Movement

Green turtles migrate from foraging areas to natal nesting beaches and may travel hundreds or thousands of kilometers each way (Hays et al. 2001). Available information on green turtle migratory behavior indicates that long distance dispersal is seen only in juvenile turtles, suggesting that larger adult-sized turtles return to forage and stay within the region of their natal rookeries (Monzón-Argüello et al. 2010).

Feeding

Early-stage juveniles forage on plant and animal life found in pelagic drift communities (such as pelagic *Sargassum* communities). After their 5- to 7-year pelagic developmental phase, they settle into coastal habitats and shift to being primarily herbivores. At this stage, their diet mainly consists of algae and seagrasses, depending on what habitat they reside in. They may also forage on sponges and other invertebrates (NOAA 2018a).

In the southeastern United States, green sea turtles' principal benthic foraging areas include Aransas Bay, Matagorda Bay, Laguna Madre, and the Gulf inlets of Texas (Doughty 1984, Hildebrand 1982, Shaver 1994), the Gulf of Mexico off Florida from Yankeetown to Tarpon Springs (Caldwell and Carr 1957), Florida Bay and the Florida Keys (Schroeder and Foley 1995), the Indian River Lagoon system in Florida (Ehrhart 1983), and the Atlantic Ocean off Florida from Brevard through Broward counties (Guseman and Ehrhart 1992, Wershoven and Wershoven 1992). The summer developmental habitat for green sea turtles also encompasses estuarine and coastal waters from North Carolina to as far north as Long Island Sound (Musick and Limpus 1997). Additional important foraging areas in the western Atlantic include coastal areas of Puerto Rico, Cuba, Mexico and Central and South America (Hirth 1971).

Species Tolerances to Selected Stressors

Green sea turtle occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to sea turtles include interactions with fishing gear, military operations, and dredging operations; habitat alterations (including channel construction); artificial lighting; vessel operations; marine debris and pollution; poaching; global climate change; cold-stunning; and predation (NMFS 2016).

The principal cause of past declines and extirpations of green sea turtle assemblages has been the overexploitation of the species for food and other products. Although intentional take of green sea turtles and their eggs is not extensive within the southeastern United States, green sea turtles that nest and forage in the region may spend large portions of their life history outside the region and outside U.S. jurisdiction, where exploitation is still a threat.

Green sea turtles, specifically, face many of the same threats as other sea turtle species, but with their primary aquatic threats being bycatch, poaching, natural predation, pollution, marine debris, and disease. Their primary terrestrial threats come from poaching of eggs and the loss and degradation of nesting habitat (NMFS 2015).

In addition to general threats, green sea turtles are susceptible to natural mortality from Fibropapillomatosis (FP) disease. In turtles, FP results in the growth of tumors on soft external tissues (flippers, neck, tail, etc.), the carapace, the eyes, the mouth, and internal organs (gastrointestinal tract, heart, lungs, etc.). These tumors range from 0.1 cm (0.04 inch) to greater than 30 cm (11.81 inches) in diameter and may affect swimming, vision, feeding, and organ function (Aguirre et al. 2002, Herbst 1994, Jacobson et al. 1989). Presently, scientists are unsure of the exact mechanism causing this disease, though it is believed to be related to both an infectious agent, such as a virus (Herbst et al. 1995), and environmental conditions (for example, habitat degradation, pollution, low wave energy, and shallow water) (Foley et al. 2005). FP is cosmopolitan, but it has been found to affect large numbers of animals in specific areas, including Hawaii and Florida (Herbst 1994, Jacobson 1990, Jacobson et al. 1991).

Cold-stunning is another natural threat to green sea turtles. Although it is not considered a major source of mortality in most cases, it affects their behavior. As temperatures fall below 8 °C to 10 °C (46.4 °F to 50 °F) turtles may lose their ability to swim and dive, often floating to the surface. The rate of cooling that precipitates cold-stunning appears to be the primary threat, rather than the water temperature itself (Milton and Lutz 2003). Sea turtles that overwinter in inshore waters are most susceptible to cold-stunning because temperature changes are most rapid in shallow water (Witherington and Ehrhart 1989a). During January 2010, an unusually large cold-stunning event in the southeastern United States resulted in around 4,600 sea turtles, mostly greens, found cold-stunned, and hundreds found dead or dying. Another large cold-stunning event occurred in the western Gulf of Mexico in February 2011, resulting in about 1,650 green sea turtles found cold-stunned in Texas. Of these, about 620 were found dead or died after stranding, while the remaining 1,030 turtles were rehabilitated and released. During this same time frame, about 340 green sea turtles were found cold-stunned in Mexico, though about 300 of those were subsequently rehabilitated and released.

Turtles are susceptible to impacts from oil spills. The DWH oil spill is estimated to have exposed a total of 154,000 small juvenile green sea turtles to oil (these juvenile greens make up 36.6% of all small juvenile sea turtles exposed to oil from the spill). About 57,300 of these juvenile greens were estimated to have died from the exposure. Four nests (580 eggs) were also translocated during response efforts, with 455 hatchlings released (the fate of which is unknown) (DWH Trustees 2015). Additional unquantified effects may have included inhalation of volatile compounds, disruption of foraging or migratory movements due to surface or subsurface oil, ingestion of prey species contaminated with oil and/or dispersants, and loss of foraging resources, which could lead to compromised growth and/or reproductive potential. There is no information currently available to determine the extent of those impacts, if they occurred (DWH Trustees 2015).

Population Status

The green sea turtle was originally listed as threatened under the ESA on July 28, 1978, except for the Florida and Pacific Coast of Mexico breeding populations, which were listed as endangered. On April 6, 2016, the original listing was replaced with the listing of 11 DPSs (81 FR 20057 2016). The Mediterranean, Central West Pacific, and Central South Pacific DPSs were listed as endangered. The North Atlantic, South Atlantic, Southwest Indian, North Indian, East Indian-West Pacific, Southwest Pacific, Central North Pacific, and East Pacific were listed as threatened. For the purposes of this consultation, only the South Atlantic DPS and North Atlantic DPS will be considered, as they are the only 2 DPSs with individuals occurring in the Atlantic and Gulf of Mexico waters of the United States.

Accurate population estimates for marine turtles do not exist because of the difficulty in sampling turtles over their geographic ranges and within their marine environments. Nonetheless, researchers have used nesting data to study trends in reproducing sea turtles over time. The North Atlantic DPS of green sea turtles, which is listed as threatened, is distributed throughout inshore and nearshore waters from Texas to Massachusetts, although most nesting occurs on Florida's southeast coast (NOAA 2020a).

The North Atlantic DPS is the largest of the 11 green sea turtle DPSs, with an estimated nester abundance of over 167,000 adult females from 73 nesting sites. Overall, this DPS is also the most data rich. Eight of the sites have high levels of abundance (i.e., <1,000 nesters), located in Costa Rica, Cuba, Mexico, and Florida. All major nesting populations demonstrate long-term increases in abundance (Seminoff et al. 2015).

The South Atlantic DPS is less than half the size of the North Atlantic DPS, with their total nester abundance estimated at over 63,000 adult females from 51 nesting sites. The South Atlantic DPS boundary adjoins the North Atlantic DPS boundary near the north coast of South America; however, green sea turtles from the South Atlantic DPS can and do travel into the Northern Gulf of Mexico (Foley et al. 2007) and could occasionally be present within the Gulf of Mexico portion of the action area. Long-term monitoring data for this DPS is relatively scarce, but existing data suggest an overall trend of increasing abundance at primary nesting sites (Seminoff et al. 2015).In the continental United States, green sea turtle nesting occurs along the Atlantic coast, primarily along the central and southeast coast of Florida where an estimated 200 to 1,100 females nest each year (Meylan et al. 1994, Weishampel et al. 2003). Occasional nesting has also been documented along the Gulf Coast of Florida (Meylan et al. 1995). Green sea turtle nesting is documented annually on beaches of North Carolina, South Carolina, and Georgia, though nesting is found in low quantities (nesting databases maintained on www.seaturtle.org).

While green turtles regularly use the northern Gulf of Mexico, the proportion of the population using the northern Gulf of Mexico at any given time is relatively low. Although the DWH oil spill resulted in adverse impacts and reduction in numbers of animals in the Gulf of Mexico, the impact on the overall population of green sea turtles is reduced by the relatively small proportion of the population that is expected to have been exposed to and directly impacted by the event; also, the impacts were primarily to smaller juveniles, which have lower reproductive value than adults and large juveniles. It is unclear what impact these losses may have caused on a population level, but it is not expected to have had a large impact on the population trajectory moving forward. However, it will likely take decades of sustained efforts to reduce the existing threats and enhance survivorship of multiple life stages of green sea turtles in order for the local population to recover equivalent to what was lost in the DWH oil spill (DWH Trustees 2015).

3.2.7 Hawksbill Sea Turtle

The hawksbill sea turtle (*Eretmochelys imbricata*) is a small to medium-sized marine turtle with an elongated oval shell with overlapping scutes on its carapace, and a relatively small head with a distinctive hawk-like beak. This section summarizes best available data about biology and condition of hawksbill sea turtle.

General Life History

Habitats

The endangered hawksbill sea turtle occurs globally, including in the Gulf of Mexico, and nests can be found from Texas to Florida (NOAA 2017b). Adults are most commonly associated with healthy coral reefs, but also occur in shallow coastal areas, lagoons or oceanic islands, narrow creeks and passes, typically in depths of less than 19.8 meters (65 feet) (NOAA 2017b, USFWS 2018). Hatchlings are often found floating in masses of sea plants, and nesting may occur on almost any undisturbed deep-sand beach in the tropics. Adult females are able to climb over reefs and rocks to nest in beach vegetation (USFWS 2018). Critical habitat (nesting beaches) has been established in the coastal waters of Mona Island, Puerto Rico, outside of the proposed action area.

Movement

After emergence, hatchlings swim offshore to mature among floating algal mats and drift lines before returning to coastal foraging grounds. Adult hawksbill turtles migrate from foraging areas to natal nesting beaches and may travel long distances each way. The nesting season varies with locality, but in most locations, nesting occurs sometime between April and November (NOAA 2017b, USFWS 2018).

Feeding

Hawksbill sea turtles feed primarily on invertebrates such as sponges, sea urchins, and barnacles, as well as seagrasses and algae. During the oceanic phase, hawksbills are thought to ingest a combination of plant and animal material associated with surface zones (Bjorndal 1997). Newborn and juvenile hawksbills have been found associated with Sargassum (Witherington et al. 2012). Hawksbill turtles are oceanic until 7-10 years of age (Bell and Pike 2012) at which point they move into neritic habitats (habitats associated with shallow areas near the continental shelf) and transition from pelagic to benthic diets (NMFS and USFWS 2013a).

Species Tolerances to Selected Stressors

Hawksbill sea turtle occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to sea turtles include interactions with fishing gear, military operations, and dredging operations; habitat alterations (including channel construction); vessel operations; marine debris and pollution; poaching; global climate change; cold-stunning; and predation (NMFS 2016). The decline of the hawksbill is primarily due to human exploitation for tortoise shell. Other terrestrial threats include loss or degradation of nesting habitat from coastal development and beach armoring, disorientation of hatchlings by beachfront lighting, and nest predation by native and non-native predators. Other aquatic threats include degradation of foraging habitat, marine pollution and debris, watercraft strikes, and bycatch from commercial fishing.

Population Status

The hawksbill sea turtle was listed as endangered throughout its entire range in 1970 under the Endangered Species Conservation Act of 1969, a precursor to the ESA. Globally, hawksbills are known to nest in 88 nesting assemblages across 10 ocean regions (NMFS and USFWS 2013a). The Atlantic population is comprised of 33 nesting sites across 4 regions that contain between 3,626 and 6,108 nesting females per season (NMFS and USFWS 2013a). While some of these sites show an upward population trajectory within the past 20 years, all of the sites show either a downward or unknown trajectory over the past 20 to 100 years.

3.2.8 Kemp's Ridley Sea Turtle

The Kemp's ridley sea turtle is the smallest of all sea turtles. Adults generally weigh less than 100 lb (45 kg) and have a carapace length of around 65 cm (2.1 feet). Adult Kemp's ridley shells are almost as wide as they are long. This section summarizes best available data about biology and condition of Kemp's ridley sea turtle.

General Life History

Habitats

The range of the Kemp's ridley includes the Gulf coasts of Mexico and the United States, and the Atlantic coast of North America, with juveniles recorded as far north as Nova Scotia and Newfoundland. The primary range of Kemp's ridley sea turtles is within the Gulf of Mexico basin. Nesting is essentially limited to the beaches of the western Gulf of Mexico, primarily in Tamaulipas and Veracruz, Mexico, with a few historical records in Campeche, Mexico. Nesting also occurs regularly in Texas and infrequently in Florida, Georgia, the Carolinas, and Virginia (USFWS 2017).

Hatchlings emerge and swim to offshore environments, where they spend around 2 years before returning to nearshore coastal habitats. Juvenile Kemp's ridley sea turtles use these nearshore coastal habitats in the warmer months; in winter they move towards deeper offshore waters. Adult Kemp's ridley habitat largely consists of sandy and muddy areas in shallow, nearshore waters less than 37 meters (120 feet) deep, although they can also be found in deeper offshore waters in areas that support their primary prey species (USFWS 2017).

Northern Gulf of Mexico waters—including in and around the proposed action area, including portions of Jefferson, Laforche and Plaquemines parishes—are important foraging and migratory pathway areas for sea turtles, especially juvenile and post-nesting Kemp's ridley sea turtles.

Movement

Juvenile Kemp's ridley sea turtles use nearshore coastal habitats from April through November, and overwinter in deeper offshore waters (or more southern waters along the Atlantic coast). Nesting occurs from April into July, during which time the turtles appear off the Tamaulipas and Veracruz coasts of Mexico. Possibly precipitated by strong winds and changes in barometric pressure, the females often nest in synchronized emergences, known as arribadas or arribazones, primarily during daylight hours.

Feeding

Juvenile and adult Kemp's ridley sea turtles primarily eat swimming crabs, but may also consume fish, jellyfish, and an array of mollusks.

Species Tolerances to Selected Stressors

Kemp's ridley sea turtle occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Kemp's ridley sea turtles face many of the same threats as other sea turtle species, including destruction of nesting habitat from storm events, oceanic events such as cold-stunning, pollution (plastics, petroleum products, petrochemicals, etc.), ecosystem alterations (nesting beach development, beach nourishment and shoreline stabilization, vegetation changes, etc.), poaching, global climate change, fisheries interactions, natural predation, and disease.

Stranding rates for Kemp's ridleys in the northern Gulf of Mexico have spiked in recent years for unknown reasons. Necropsy results indicate that a number of the turtles have likely perished due to forced submergence, a cause of death commonly associated with fishery interactions (B. Stacy, NMFS pers. comm. to M. Barnette, NMFS Protected Resources Division, March 2012). However, available information indicates that fishing effort was limited during the periods when strandings occurred. Furthermore, 80% or more of all Louisiana, Mississippi, and Alabama stranded sea turtles in the past 5 years were Kemp's ridleys. This could be a function of the species' preference for shallow, inshore waters coupled with increased population abundance.

Population Status

The Kemp's ridley sea turtle was listed as endangered on December 2, 1970, under the Endangered Species Conservation Act of 1969, a precursor to the ESA. The Kemp's ridley has historically been recognized as the most endangered of the sea turtles (Groombridge 1982, TEWG 2000, Zwinenberg 1977). Its numbers precipitously declined after 1947 (in 1947, over

40,000 nesting females were estimated in a single arrival event to core nesting areas; in 1985 the nesting population produced a low of 702 nests).

The implementation of nesting protection efforts and regulatory requirements for the use of turtle excluder devices in commercial fisheries has placed the species on a trajectory towards recovery. The number of nests observed in core nesting areas increased exponentially from the mid-1980s through 2009, leading to predictions that the species could be downlisted to threatened by 2011 (Caillouet et al. 2018). The subsequent Deepwater Horizon oil spill posed a major setback to species recovery (DHNRDAT 2016). A large number of juvenile and adult turtles were killed directly by oil exposure and subsequent indirect effects from damage to foraging habitat, substantially reducing both the size of the population and the percentage of mature females (Caillouet et al. 2016, Caillouet et al. 2018, DHNRDAT 2016, Putman et al. 2015). Nest abundance on core monitoring beaches dropped by more than one-third between 2009 and 2015 (Caillouet et al. 2016, Caillouet et al. 2018, DHNRDAT 2016). In 2011, a total of 20,570 nests were documented in Mexico (81% of these nests were documented along the 30 km [18.6 miles] of coastline patrolled at Rancho Nuevo) and 199 nests were recorded in the U.S., primarily in Texas (NMFS 2011). Population modeling suggests that the population could resume rebuilding as long as existing regulatory protections continue, and habitat impacts are avoided (Kocmoud et al. 2019).

3.2.9 Leatherback Sea Turtle

The leatherback sea turtle (*Dermochelys coriacea*) is the largest, deepest diving, and most migratory and wide ranging of all sea turtles. The adult leatherback can reach 1.2 meters to 2.4 meters (4 feet to 8 feet) in length and 500 to 2,000 pounds in weight. Its shell is composed of a mosaic of small bones covered by firm, rubbery skin with 7 longitudinal ridges or keels. This section summarizes best available data about biology and condition of leatherback sea turtles.

General Life History

Habitats

The endangered leatherback sea turtle has the widest global distribution of all reptile species and circumnavigates the Atlantic Ocean (NOAA 2020b). Leatherbacks inhabit a wide range of temperatures and a broad north-to-south geographic range (NMFS and USFWS 1995). Leatherbacks can migrate more than 10,000 km (6,000 miles) in a single year (Benson et al. 2007a, Benson et al. 2011, Eckert 2006, Eckert et al. 2006). The northwest Atlantic population of leatherback sea turtles nests primarily on sandy, tropical beaches from southern Virginia to Alabama, with additional nesting beaches along the northern and western Gulf of Mexico.

While leatherbacks also forage in shallower coastal waters, they appear to prefer the open ocean at all life stages (Heppell et al. 2003). Non-nesting, adult female leatherbacks are reported throughout the U.S. Atlantic, Gulf of Mexico, and Caribbean Sea. Historic nesting in the Barataria Basin was limited to barrier island beaches.

Movement

Migratory routes of leatherbacks are not entirely known; however, recent information from satellite tags have documented long travels between nesting beaches and foraging areas in the Atlantic and Pacific Ocean basins (Benson et al. 2007a, Benson et al. 2011, Eckert 2006, Eckert et al. 2006, Ferraroli et al. 2004, Hays et al. 2004, James et al. 2005). Leatherbacks nesting in Central America and Mexico travel thousands of miles through tropical and temperate waters of the South Pacific (Eckert and Sarti 1997, Shillinger et al. 2008). Data from satellite tagged leatherbacks suggest that they may be traveling in search of seasonal aggregations of jellyfish (Benson et al. 2007b, Bowlby et al. 1994, Graham 2009, Shenker 1984, Starbird et al. 1993, Suchman and Brodeur 2005).

Unlike other sea turtle species, female leatherbacks do not always nest at the same beach year after year; some females may even nest at different beaches during the same year (Dutton et al. 2005, Eckert 1989, Keinath and Musick 1993, Spotila et al. 1996).

Feeding

Leatherback sea turtles search for food between latitudes 71 °N and 47 °S, in all oceans, and travel extensively to and from their tropical nesting beaches. Leatherbacks forage for soft-bodied prey such as jellyfish and sea squirts (Heppell et al. 2003).

Species Tolerances to Selected Stressors

Leatherback sea turtle occurrence in the action area by life stage is described in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to sea turtles include interactions with fishing gear, military operations, and dredging operations; habitat alterations (including channel construction); vessel operations; marine debris and pollution; poaching; global climate change; cold-stunning; and predation (NMFS 2016). Specifically, the top threats to leatherbacks are bycatch in fishing gear, harvesting of eggs, intentional killing, vessel strikes, nesting beach habitat loss and alteration, ocean pollution, and marine debris (NMFS 2015)

Population Status

The leatherback sea turtle was listed as endangered throughout its range in 1970 and subsequently protected under the ESA in 1973.

The equatorial waters appear to be a barrier between breeding populations; the northwestern Atlantic Ocean stock appears to be restricted to areas north of the equator (NMFS and USFWS 2013b). The most recent population estimate suggests a range of 34,000-94,000 adult leatherbacks worldwide, with 470 nesting sites in the Atlantic Ocean (NMFS and USFWS 2013b). No nesting is known to occur in the proposed action area. The leatherback sea turtle was listed as endangered throughout its range in 1970, under the Endangered Species Conservation Act of 1969.

3.2.10 Loggerhead Sea Turtle

Loggerheads sea turtles (*Caretta caretta*) are named for their relatively large heads. They are the most abundant species of sea turtle found in U.S. coastal waters. This section summarizes best available data about biology and condition of the loggerhead sea turtle.

General Life History

Habitats

Loggerhead sea turtles have a global distribution and inhabit the continental shelf and estuarine habitats in tropical and temperate regions. The Northwest Atlantic (NWA) DPS nests along the U.S. East and Gulf coasts, but most nesting occurs from southern Virginia to Alabama (NOAA

2017d). After decades without any nesting within Barataria Basin, 2 adults were documented as successfully nesting on Grande Isle in 2015. Important habitat for loggerhead sea turtles includes nearshore reproductive habitat, winter areas, breeding areas, migratory corridors, and/or *Sargassum* habitat.

Movement

After emerging from nests, hatchlings migrate offshore and become associated with *Sargassum* habitats, drift lines, and other convergence zones. Oceanic juveniles normally return to coastal habitats after 7 to 12 years. Loggerhead adults nest on ocean beaches and occasionally on estuarine shorelines. Non-nesting, adult female loggerheads are reported throughout the U.S. Atlantic, Gulf of Mexico, and Caribbean Sea. Little is known about the distribution of adult males who are seasonally abundant near nesting beaches. Aerial surveys suggest that loggerheads as a whole are distributed in U.S. waters as follows: 54% off the southeast U.S. coast, 29% off the northeast U.S. coast, 12% in the eastern Gulf of Mexico, and 5% in the western Gulf of Mexico (TEWG 1998).

Feeding

Juveniles are omnivorous and forage on crabs, mollusks, jellyfish, and vegetation at or near the surface (Dodd Jr. 1988). Subadult and adult loggerheads are primarily found in coastal waters and eat benthic invertebrates such as mollusks and decapod crustaceans in hard bottom habitats. Their strong jaws allow them to predate upon hard-shelled prey, such as whelks and conch. In neritic zones, loggerheads are primarily carnivorous, although they do consume some plant matter as well (see Bjorndal 1997 and Dodd Jr. 1988 for reviews). Loggerheads feed on a wide variety of food items with ontogenetic or developmental stage, regional, and even individual differences in diet. Loggerhead diets have been described from just a few coastal regions, and little information is available about differences or similarities in diet at various life stages. In general, loggerheads in neritic habitats within the NWA DPS prey on benthic invertebrates, primarily mollusks and benthic crabs (NMFS and USFWS 2008).

Species Tolerances to Selected Stressors

A description of loggerhead sea turtle occurrence in the action area by life stage is provided in Table 4.6-1. Species responses to stressors anticipated to result from the Project will be discussed in the Analysis of Effects (Section 5.0).

Threats to sea turtles include interactions with fishing gear, military operations, and dredging operations; habitat alterations (including channel construction); vessel operations; marine debris and pollution; poaching; global climate change; cold-stunning; and predation (NMFS 2016). Loggerheads may be particularly affected by organochlorine contaminants; they have the highest organochlorine concentrations (Storelli et al. 2008) and metal loads (D'Ilio et al. 2011) in sampled tissues among the sea turtle species. Climate change may also have an impact on loggerhead sea turtles, as modeling suggests an increase of 2 °C (2.6 °F) above current temperatures would significantly skew sex ratios so existing nesting beaches in North Carolina would result in 80% female offspring, while beaches in southern Florida would result in almost 100% female offspring (Hawkes et al. 2007). Further increases beyond 3 °C (5.4 °F) are predicted to result in egg mortality in nests (Hawkes et al. 2007). Warmer sea surface temperatures have also been correlated with an earlier onset of loggerhead nesting in the spring (Hawkes et al. 2007, Weishampel et al. 2004), short inter-nesting intervals (Hays et al. 2002), and shorter nesting seasons (Pike et al. 2006).

Population Status

The loggerhead sea turtle (*Caretta caretta*) was listed as a threatened species throughout its global range on July 28, 1978. NMFS and USFWS published a Final Rule that designated 9 DPSs for loggerhead sea turtles (76 FR 58868, September 22, 2011, and effective October 24, 2011). This rule listed 4 DPSs as threatened and 5 as endangered, as follows:

- **Threatened**: Northwest Atlantic Ocean DPS, South Atlantic Ocean DPS, South Indo-Pacific Ocean DPS, and Southwest Indian Ocean DPS
- **Endangered**: Northeast Atlantic Ocean DPS, Mediterranean Sea DPS, North Pacific Ocean DPS, South Pacific Ocean DPS, and North Indian Ocean DPS

The Northwest Atlantic (NWA) is the only loggerhead DPS that occurs within the proposed action area. Richards et al. (2011) estimated the NWA DPS adult female loggerhead population to be between 30,096 and 51,211 turtles based on nesting data between 2001 and 2010. A preliminary regional abundance survey of loggerheads within the NWA DPS estimated about 801,000 individuals (NMFS 2016). The NWA DPS was negatively affected by the Deepwater Horizon oil spill. Based on modeled estimates and extrapolation from observed mortalities, anywhere from 21,000 to 31,000 juvenile loggerhead turtles were likely exposed to the spill, and over 10,000 were likely directly killed (Putman et al. 2015, Wallace et al. 2015). About 30,000 large juvenile and adult loggerheads were likely exposed and as many as 3,600 directly killed (Wallace et al. 2015). The estimated adult female population size in the western North Atlantic was between 32,000 and 45,000 in 2010 (Richards et al. 2011). These findings suggest that a significant percentage of the breeding female population in this DPS could have been exposed to oiling effects. This had significant implications for smaller breeding groups like the northern Gulf of Mexico subpopulation, supported by between 350 and 550 adult females in 2010 (Richards et al. 2011). Loggerhead nest densities declined by over 40% (relative to expected nesting rates) on beaches affected by the spill and subsequent cleanup activities (Lauritsen et al. 2017).

3.3 Critical Habitat

As indicated in table 4.1-1, critical habitat has been designated for pallid sturgeon, piping plover, West Indian manatee, green sea turtle, hawksbill sea turtle, leatherback sea turtle, and loggerhead sea turtle. However, the only species that have designated critical habitat within or adjacent to the action area are piping plover and loggerhead sea turtles (Figure 3.3-1).

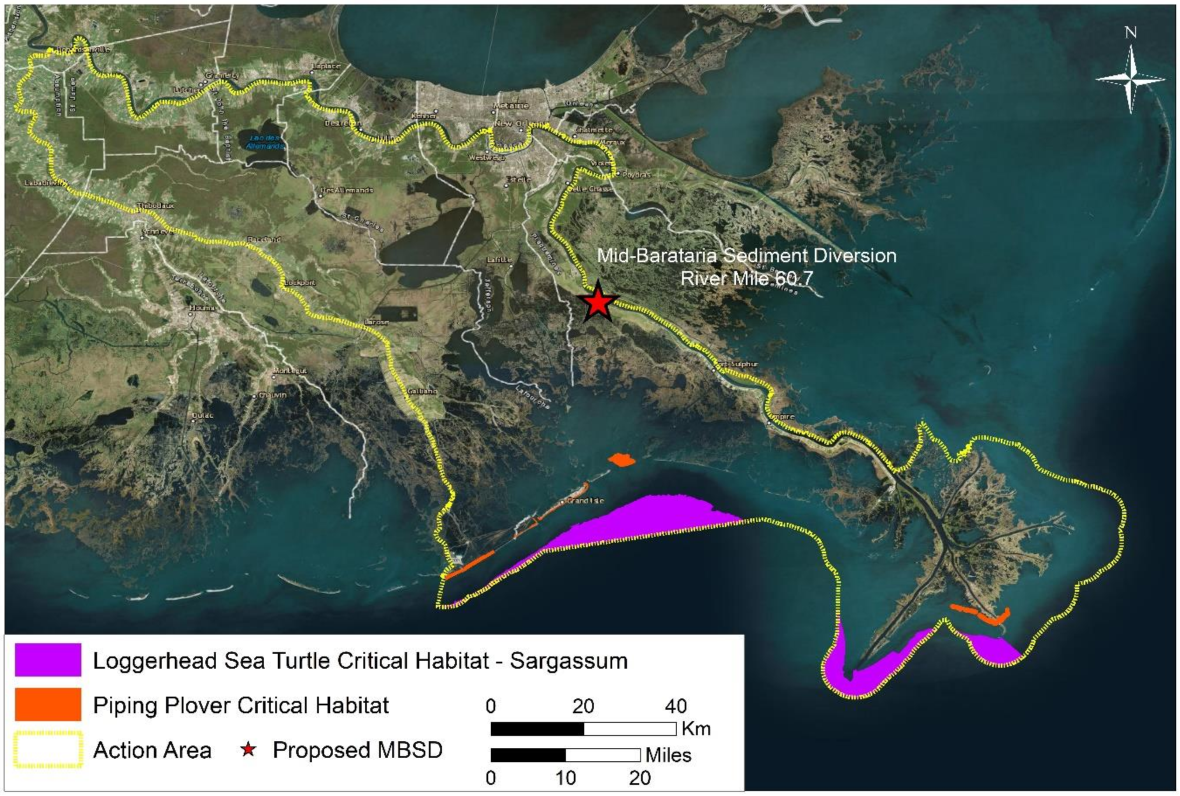


Figure 3.3-1. Critical Habitat for Federally Listed Species in the Action Area, adjacent to the Project Action Area.

3.3.1 *Piping Plover*

The USFWS has designated critical habitat for the piping plover throughout its breeding range and nonbreeding wintering areas, including coastal beaches and barrier islands of the northern Gulf of Mexico (USFWS 2017e). Critical habitat for piping plovers has been designated along the barrier islands, which are located along the southern edge of the proposed action area of Barataria Basin (Figure 3.3-1). On July 10, 2001, USFWS published a Final Rule designating 142 areas along the coasts of North Carolina, South Carolina, Georgia, Florida, Alabama, Mississippi, Louisiana, and Texas as critical habitat for the wintering population of the piping plover (66 FR 36037). Approximately 66,881 hectares (165,211 acres) of area have been designated as critical habitat.

Primary constituent elements (PCEs) for the piping plover wintering habitat are those habitat components that are essential for the plover's primary biological needs of foraging, sheltering, and roosting. The PCEs are found in coastal areas that support intertidal beaches and flats and associated dune systems and flats above high tide.

3.3.2 Loggerhead Sea Turtle

NOAA has designated critical habitat for the loggerhead sea turtle that includes one or a combination of habitat types: nearshore reproductive habitat, winter area, breeding areas, constricted migratory corridors and/or *Sargassum* habitat (79 FR 39855). Critical *Sargassum* habitat that has been designated for loggerhead sea turtles occurs just outside of the southern barrier islands, within the action area, but outside of any Delft3D modelled project impacts (Figure 3.3-1). The unit of critical habitat within and adjacent to the action area is identified as "Gulf of Mexico *Sargassum*". The unit follows the 10-meter depth contour starting at the mouth of South Pass of the Mississippi River proceeding west and south to the outer boundary of the U.S. Exclusive Economic Zone. The physical or biological features essential for conservation in critical habitat within and adjacent to the action area are focused on *Sargassum* habitat; accumulations of floating material such as *Sargassum* are important for development and foraging for young loggerhead turtles. Primary Constituent Elements (PCEs) that support this habitat also focus on *Sargassum*, and include the following:

- Convergence zones, surface-water downwelling areas, the margins of major boundary currents (Gulf Stream), and other locations where there are concentrated components of the *Sargassum* community in water temperatures suitable for the optimal growth of *Sargassum* and inhabitance of loggerheads;
- *Sargassum* in concentrations that support adequate prey abundance and cover;
- Available prey and other material associated with *Sargassum* habitat including, but not limited to, plants and cyanobacteria and animals native to the Sargassum community such as hydroids and copepods;
- Sufficient water depth and proximity to available currents to ensure offshore transport (out of the surf zone), and foraging and cover requirements by *Sargassum* for post-hatchling loggerheads (i.e., > 10 m depth).

4.0 ENVIRONMENTAL BASELINE

The environmental baseline is defined as "the past and present impacts of all Federal, State, or private actions and other human activities in the proposed action area, the anticipated impacts of all proposed Federal projects in the proposed action area that have already undergone formal or early section 7 consultation, and the impact of State or private actions which are contemporaneous with the consultation process" (50 CFR 402.02). The following information discusses the environmental setting of the Mississippi River and Barataria Basin, focusing on conditions related to aquatic habitat and other biological resources, trends in regional landforms, and how ESA listed species use the Mississippi River and Barataria Basin. Section 4.7 provides a summary of the environmental baseline that would be used for this Project.

Information in this section is consistent with the EIS, RP, and supporting information provided by CPRA. Where appropriate, this section will refer to sections of the EIS or RP for additional information.

4.1 Current Habitats

The proposed action area is located within the southern portion of the Mississippi Alluvial Plain, a sub-province of the Atlantic Coastal Plain (Vigil et al. 2000, Hunt 1967), which follows the Mississippi River south from Illinois through Missouri, Arkansas, Tennessee, Mississippi, and Louisiana, ending at the Gulf of Mexico (Omernik 1987). This sub-province is dominated by the Mississippi River. The Mississippi-Missouri River system drains water and the associated sediment load from the entire central portion of the United States. The northern portion of the Mississippi Alluvial Plain sub-province is known as the Mississippi Embayment, a low-lying geologic basin filled with fluvial sediments deposited by the river between the Cretaceous period and present day. The river has occupied its current channel for the last 1,320 years (McFarlan 1961, Saucier 1963, Saucier 1994, Weinstein and Gagliano 1985, Tornqvist et al. 1996).

The southern portion of the Mississippi Alluvial Plain sub-province is known as the Mississippi River Delta. The delta, as we know it today, is geologically modern and most surficial sediments were deposited by the Mississippi and Atchafalaya Rivers during the Holocene epoch, beginning about 7,000 years ago (Turner et al. 2018). The main channel of the Mississippi River is dynamic, with delta lobes forming from sediment deposition in the Gulf of Mexico and delta switching occurring approximately every 1,000 to 1,500 years over the last 7,000 years (Roberts 1997, Day et al. 2007, Blum and Roberts 2012). The Mississippi River's modern active delta, known as the Plaquemines-Balize delta, or Birdfoot Delta, extends farthest into the Gulf of Mexico in a large middle lobe. The action area is located primarily within the deltaic coastal marshes including, and to the west of, this currently active lobe (Daigle et al. 2006). Portions of the action area also overlap the more inland swamps and Holocene meander belts that form the margins of these marshes. In this document, references to the Mississippi

River Delta describe the area encompassed by the current Mississippi River, its historic inactive lobes, and the currently active Birdfoot Delta. The action area comprises only the central portion of the broader Mississippi River Delta.

In developing the Louisiana Comprehensive Wildlife Conservation Strategy (Louisiana Department of Wildlife and Fisheries [LDWF] 2015), the LDWF and the Nature Conservancy developed a system of ecoregions specific to the state of Louisiana, based on similarities in physiography. The Barataria Basin comprises parts of 2 ecoregions: the Mississippi River Alluvial Plain ecoregion and the Gulf Coast Prairies and Marshes ecoregion (LDWF 2005). The Mississippi River Alluvial Plain includes all or parts of Assumption, St. James, Ascension, St. John the Baptist, St. Charles, Jefferson, Orleans, and Plaquemines parishes in the basin as well as St. Bernard Parish. Terrestrial (upland) habitats in the Barataria Basin associated with this ecoregion include primarily agriculture/cropland/grassland, some hardwood mixed forest, and live oak natural levee forest. The Mississippi River Alluvial Plain is, as its name implies, rich in alluvial sediments and is associated with primarily bottomland hardwood forests, as well as freshwater swamps and other forested wetlands.

The Gulf Coast Prairies and Marshes ecoregion in Louisiana includes the coastal portion of the Barataria Basin. This ecoregion includes all or portions of St. Charles, St. John the Baptist, Jefferson, Plaquemines, and Orleans parishes in the basin. Barrier islands, live oak natural levee forest, coastal dune grasslands, and agriculture/cropland/grassland habitats are typical of this ecoregion (LDWF 2005). The coastal marsh areas are composed of salt, brackish, intermediate, and fresh marshes. Other plant communities associated with the Gulf Coast Prairies and Marshes ecoregion are the cypress and cypress-tupelo swamps, coastal live oak-hackberry forests (cheniers) of the southwest coast, live oak natural levee forests of the southeast coast, and some bottomland hardwood forests.

Physical features characterizing the Barataria Basin include natural and artificial levees, bays, lakes, bayous, coastal beaches, barrier islands, forested wetlands, and marshes, which occur across gradients of both elevation and salinity. The upper-most extent of the Barataria Basin is at Donaldsonville, Louisiana (Conner and Day 1987). Water flows through a system of lakes and bayous, from Lac des Allemands in the upper basin, to Lake Salvador via Bayou des Allemands, south into Little Lake via Bayou Perot, and then into Barataria Bay (see Figure 4.1-1). The lower portion of the basin is a bar-built estuary with shallow water, sand bars, and a low-tide, low-energy coast (Conner and Day 1987). The barrier islands between the bay and the Gulf of Mexico moderate the effects of marine influences and storms in the basin. In addition to the natural waterways, the Gulf Intracoastal Waterway (GIWW), which bisects the basin from northeast to southeast below Lake Salvador, and the Barataria Bay Waterway, which extends from below Lake Salvador to Barataria Bay, are the primary federal navigation channels that cross through the basin.

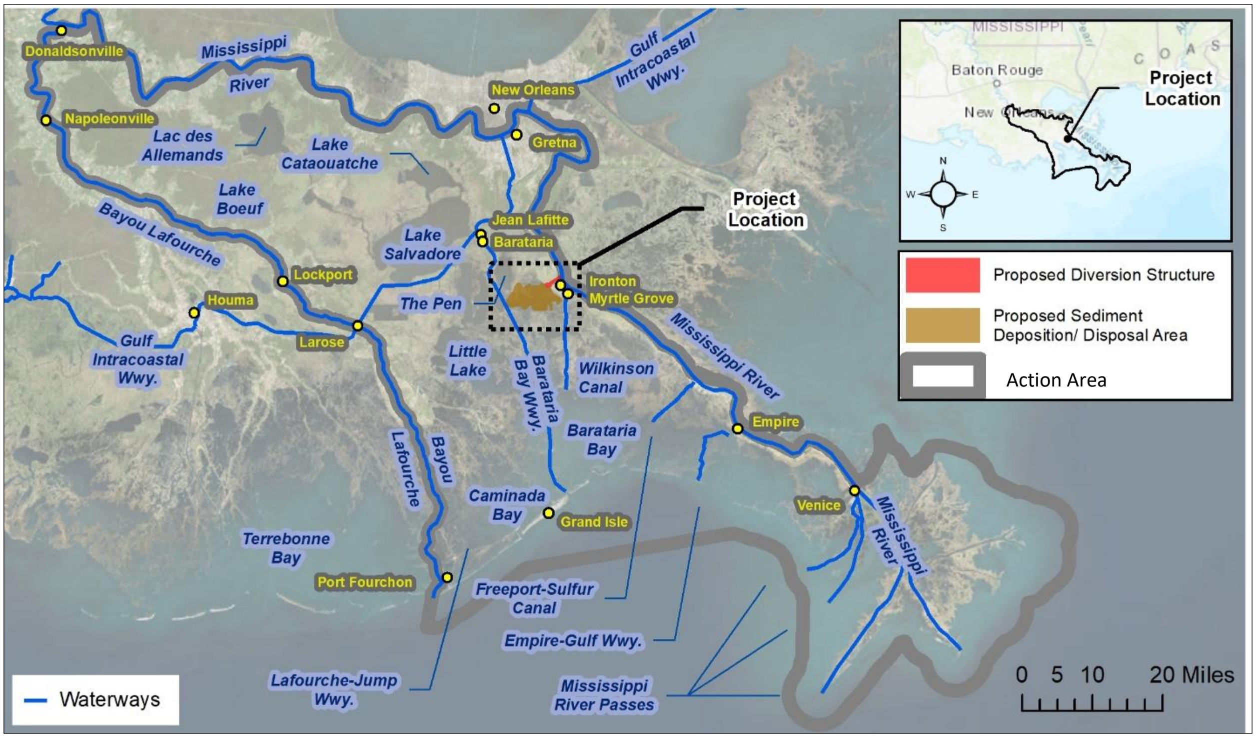


Figure 4.1-1. Major Waterbodies in the Action Area, with Key Towns and Landmarks

4.2 Ambient Water Quality (Freshwater to Marine)

This section describes selected parameters of the ambient water quality in the proposed action area, based on available data from the USGS National Water Quality Monitoring Council (USGS 2018), the Louisiana Department of Environmental Quality (LDEQ) Ambient Water Quality Data Portal (LDEQ 2018), and CPRA's Coastwide Reference Monitoring System (CRMS) (CPRA 2018). These data were gathered for the water quality stations shown in Figure 4.2-1. Mississippi River water quality is also discussed by river segment, shown in Figure 4.2-2.

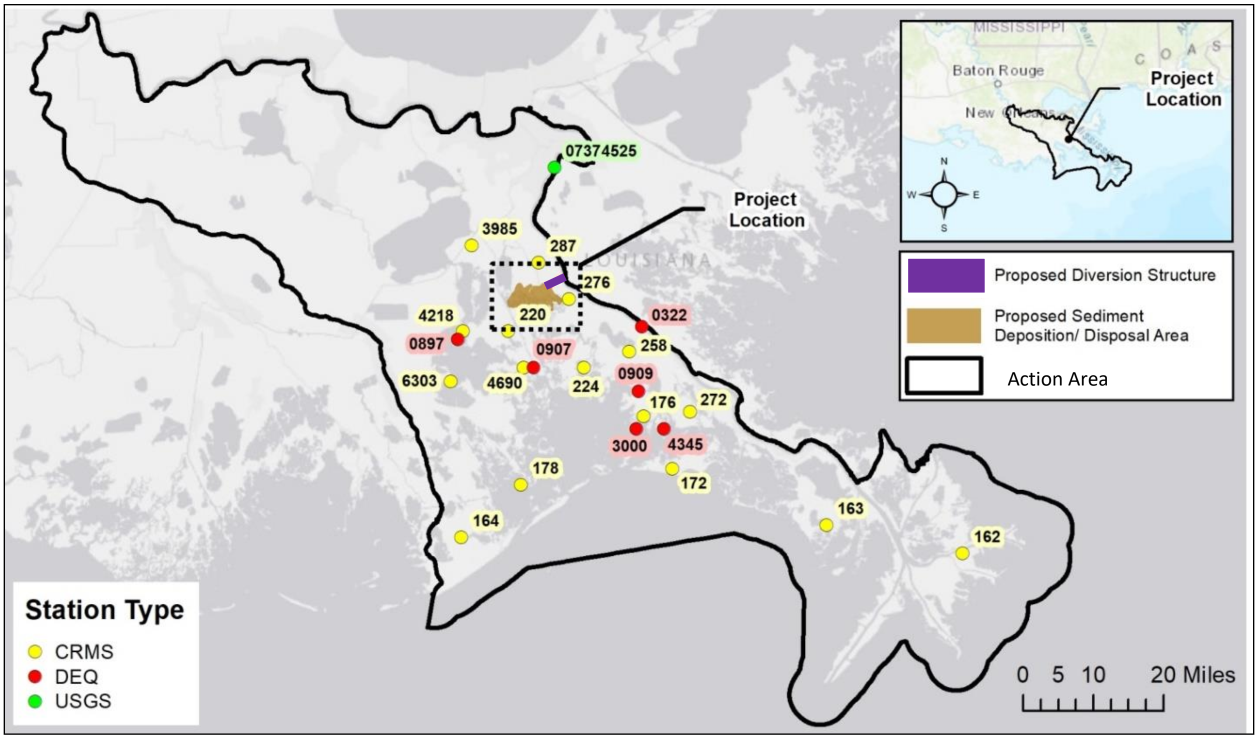


Figure 4.2-1. USGS, LDEQ, and CRMS Ambient Water Quality Stations Used for This Section
(Sources: USGS 2018, CPRA 2018, LDEQ 2018)

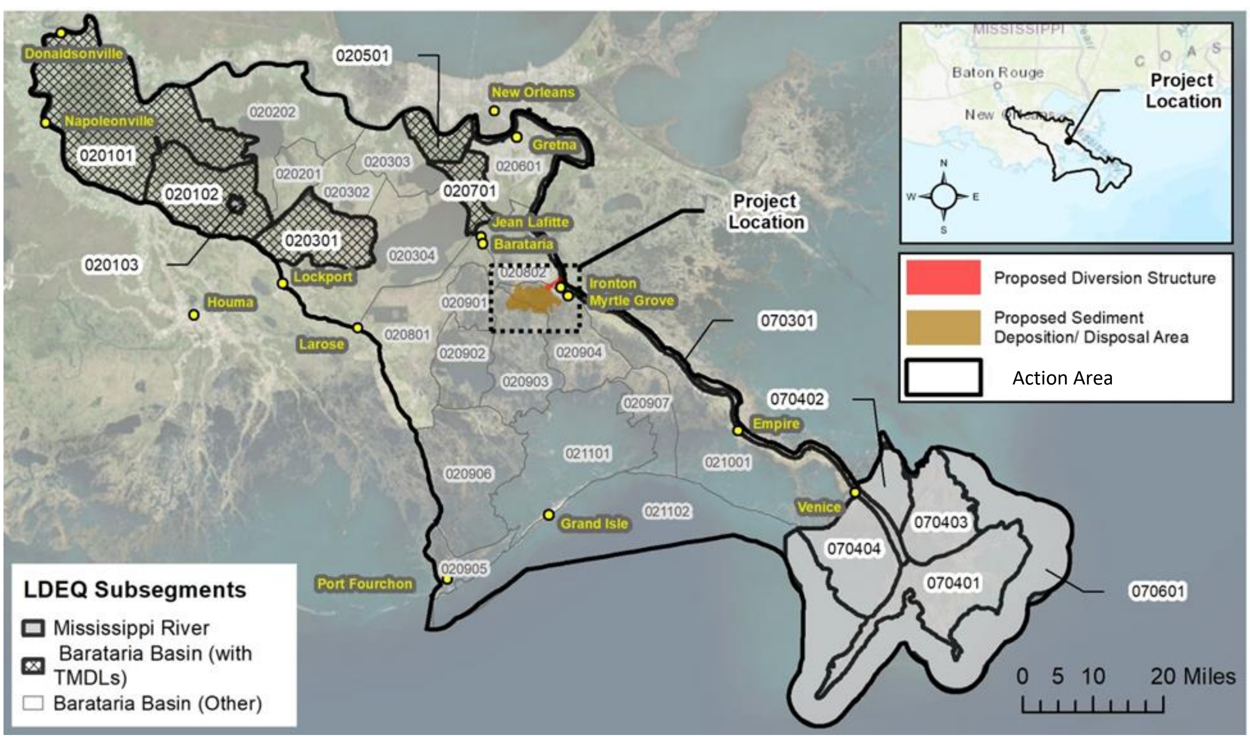


Figure 4.2-2. Mississippi River and Barataria Basin Water Quality Subsegments in the Action Area

4.2.1 Specific Conductance

Specific conductance is a measure of the ability of a water mass to conduct electricity. Because the ability to conduct electricity varies with the concentration of ionized compounds, it is an indirect measurement of the concentration of ions in solution. It is one of the most frequently measured and useful water quality parameters, and it can be an indicator of salinity intrusion into freshwater or brackish water systems. It is also useful to quantify stress to aquatic communities, as many aquatic plants and organisms have an optimal salinity range. Significant fluctuations (magnitude and/or duration) above or below the optimal range can result in stress, mortality, or habitat shifts. The LDEQ has not adopted water quality standards for specific conductance.

The conversion of specific conductance to salinity incorporates both water temperature and pressure. The atmospheric pressure of surface water is 1 pound per square inch (psi); therefore, pressure is a constant for calculations of salinity. Table 4.2.1-1 provides a range of salinity values at 25 ˚C (77 ˚F) for corresponding specific conductance values.

Table 4.2.1-1. Specific Conductance versus Salinity at 25˚C (77˚F)

Marsh Type	Specific Conductance Range (µS/cm)	Salinity Range (ppt[a])
Fresh	0-2,200	0-1
Intermediate	2,200-9,300	1-5
Brackish	9,300-29,300	5-18
Saline	29,300-46,200	18-30

Source: FGDC 2013
[a] parts per thousand

Specific conductance values in the Mississippi River at Belle Chasse upstream of the proposed Project's gated control structure ranged from 259 to 709 µS/cm between 1977 and 2017, consistent with expected values for a freshwater system. The data indicate an overall pattern of lower values associated with higher average discharge (see Table 4.2.1-2). Downstream of the proposed Project diversion structure, the most recent long-term data available indicate that specific conductance in the Mississippi River at West Pointe a la Hache ranged from 225 µS/cm to 640 µS/cm between 1971 and 1998. Based on Table 4.2.1-1, this would correspond with a salinity value of 0 to 1 parts per thousand (ppt) (fresh water).

Table 4.2.1-2 Comparison of Mississippi River and Barataria Basin Temperature and Specific Conductance

Month	Mississippi River Average Flow [a] (cfs)	Monthly Average Temperature °C (°F)		Specific Conductance (µS/cm)	
		Mississippi River [a]	Barataria Basin [b]	Mississippi River [a]	Barataria Basin [b]
January	639,506	6.6 (43)	13 (55)	367	14,281
February	596,742	7.1 (45)	15 (59)	369	12,784
March	683,182	10 (50)	19 (66)	364	12,400
April	786,672	16 (61)	23 (73)	356	10,522
May	769,218	20 (68)	26 (79)	368	10,216
June	647,750	26 (79)	29 (84)	410	9,421
July	533,649	29 (84)	30 (86)	427	9,694
August	389,346	30 (86)	30 (86)	490	10,613
September	272,003	29 (84)	28 (82)	495	13,034
October	297,083	23 (73)	24 (75)	488	15,873
November	320,673	17 (63)	19 (66)	488	18,376
December	518,222	11 (52)	15 (59)	431	17,416

[a] - USGS National Water Information System (NWIS) Belle Chasse Station, 1977-2017
[b] - Selected CRMS stations monthly average data in Barataria Basin, 2006-2018

In the Barataria Basin, specific conductance concentrations were evaluated using data from the CRMS stations shown on Figure 4.2-1 and described in Table 4.2.1-2. All available data collected between 2006 and early 2018 were reviewed. In aggregate, the Barataria Basin exhibits significantly higher specific conductance concentrations than the Mississippi River, consistent with expected values for a brackish to saline system. The data show a correlation between seasonally increasing specific conductance concentrations and decreasing temperature in the Barataria Basin (see Table 4.2.1-2).

A spatial gradient is also present in the basin, with fresher water in the upper reaches transitioning to more saline conditions in the southern area of the basin near the Gulf (see Figure 4.2.1-1). The exceptions are the 2 stations (163 and 162) located within the Mississippi River Birdfoot Delta, which are influenced by the river and exhibit much lower specific conductance concentrations. A monthly comparison of specific conductance between the Mississippi River and the Barataria Basin depicts consistently fresher (less saline) conditions in the Mississippi River (see Table 4.2.1-2).

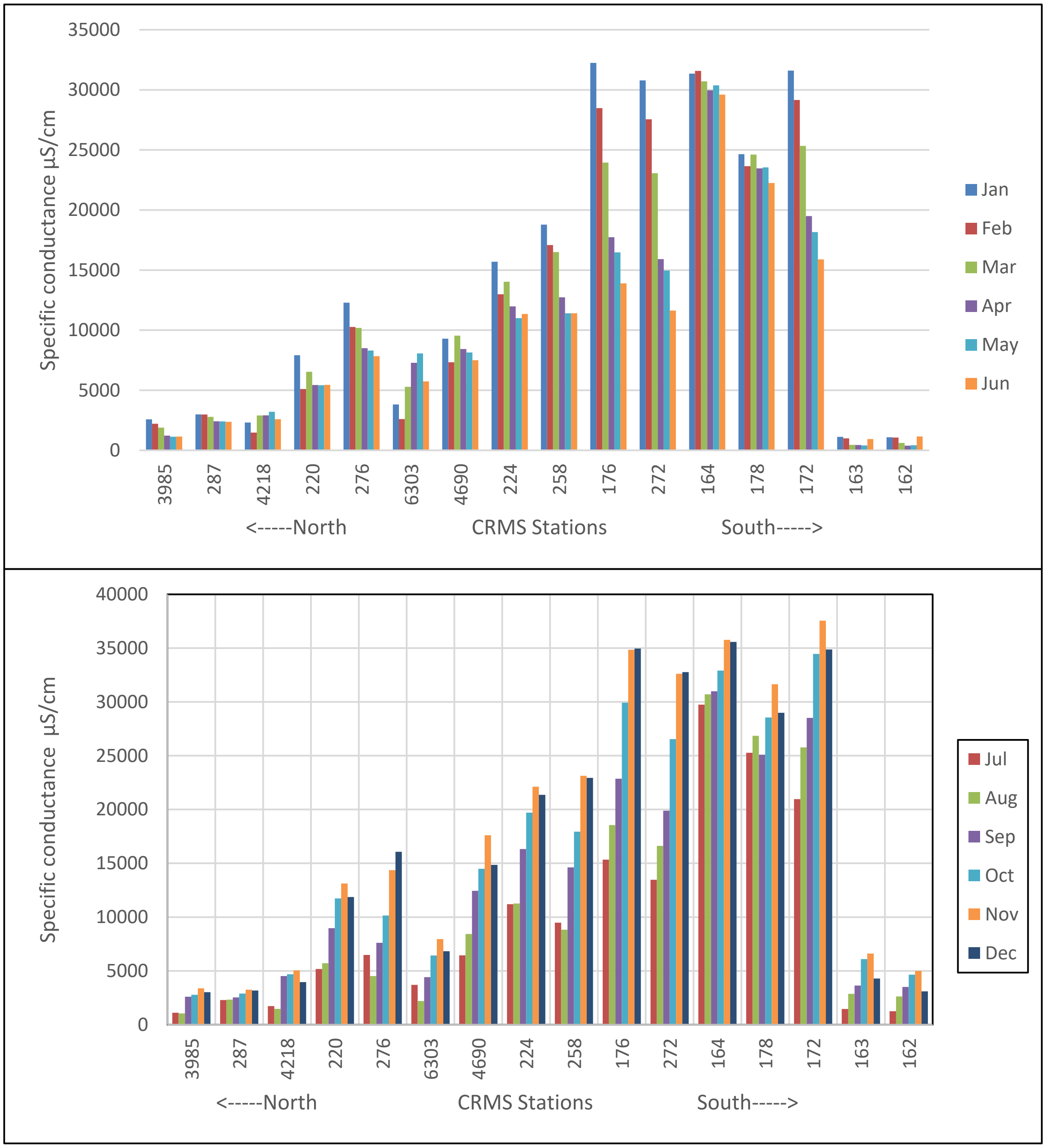


Figure 4.2.1-1. Monthly Specific Conductance Average (2006-2018) at Select Barataria Basin CRMS Sites.

4.2.2 Salinity

Salinity is a measure of dissolved salt in the water column, which can be calculated from specific conductance and water temperature. For stations in the Mississippi River, salinity values are not readily available; measurements are more frequently provided as specific conductivity. Salinity concentrations correlate in a positive manner with both specific conductivity (see Figure 4.2.2-1) and temperature. Consequently, salinity concentrations within the Barataria Basin follow the same general trends as the specific conductivity data described in Section 4.2.1 above. The LDEQ has not adopted water quality standards for salinity.

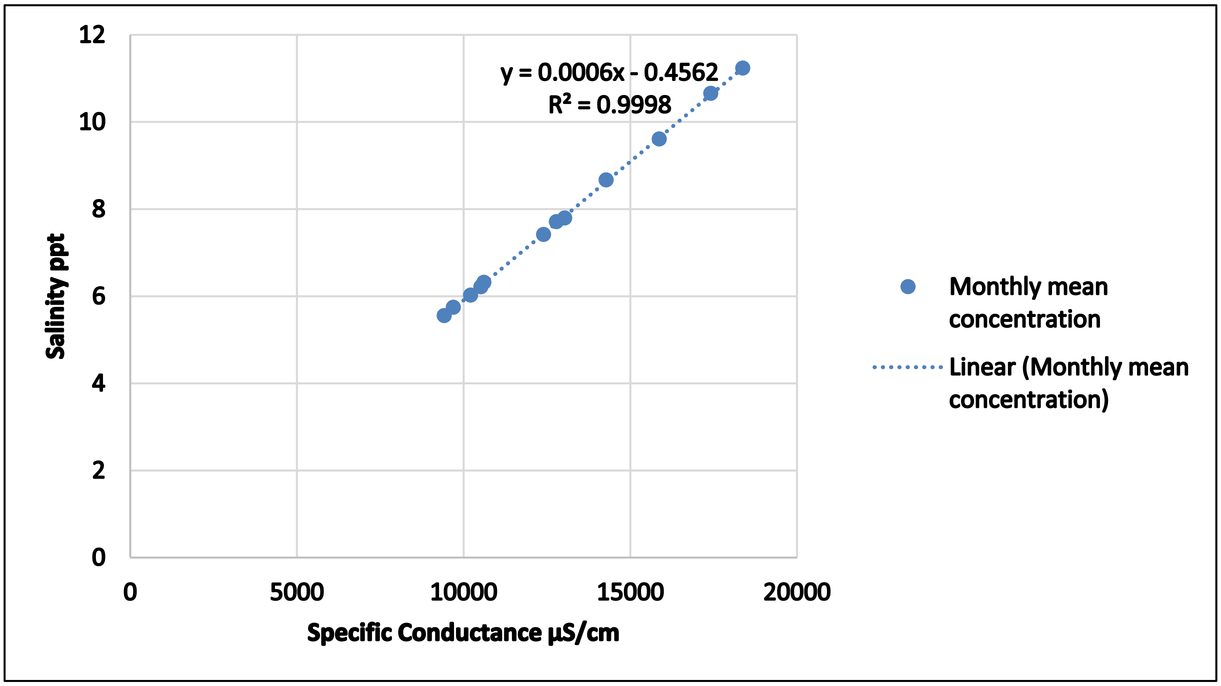


Figure 4.2.2-1. Correlation between Monthly Average Specific Conductance and Salinity (2006-2018) at Select CRMS Sites.

Annual average salinity at select CRMS stations in the Barataria Basin ranged from 7.7 to 11 ppt between 2006-2018, with lower values in the upper portions of the proposed action area. The exceptions are the 2 stations located within the Mississippi River Birdfoot Delta, (Stations 162 and 163) which are influenced by the river and exhibit much lower salinity concentrations. There is a substantial range in salinity concentrations at individual stations, indicating a highly dynamic system. Salinity concentrations are influenced by numerous factors, including seasonal rain events, Mississippi River discharge, synoptic and seasonal timescale wind-forcing, and lunar tides. Figure 4.2.2-2 displays the variability in seasonal average salinity across the proposed action area. Salinity in the proposed action area is variable and generally ranges from fresh in the spring and summer to brackish in the fall and winter.

Turner et al. (2017) conducted monthly water quality sampling at 37 stations in the Barataria Basin for nutrients, salinity, and solids between 1994 and 2016. The sampled transect extended from Grand Isle northward to Bayou Chevreuil. In the study, salinity concentrations ranged from 0 to 21 practical salinity units (psu), which are roughly equivalent to ppt. The study found that annual average salinity in the Barataria Basin declined over the 22 years of sampling, and that the salinity in the basin is strongly correlated with the average annual discharge of the Mississippi River at Tarbert Landing, Louisiana.

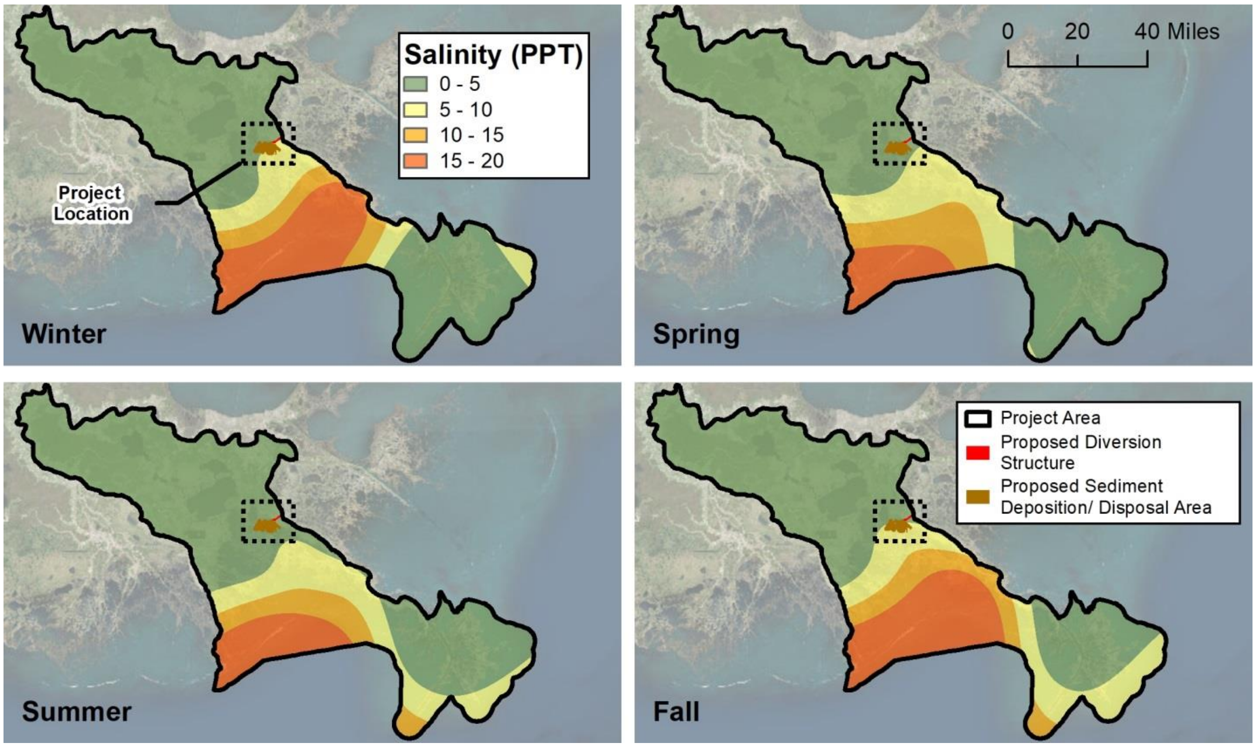


Figure 4.2.2-2. Generalized Seasonal Salinity Averages (2006-2018) at Select Barataria Basin CRMS Sites. (Source: Generated by GEC based on CPRA CRMS data)

4.2.3 Temperature

Water temperature can directly impact biological activity and growth in aquatic plants and animals. Aquatic plants and other organisms often have a preferred temperature range in which they thrive. Temperatures that fluctuate above or below the optimal range and the optimal magnitude and/or duration may lead to stress or mortality in these organisms. Numeric temperature water quality standards are defined in LAC 33:IX.1113.C.4 and 1123.Table 3 (LDEQ 2016). In general, the maximum temperature criterion in freshwater systems is 32.2 °C (89.9 °F) and the maximum criterion in estuarine and coastal waters is 35 °C (95 °F).

In Mississippi River subsegments 070301 and 070601, the maximum temperature criterion is 32 °C (about 90 °F); in subsegment 070401, the maximum criterion is 35 °C (95 °F) (see Figure 4.2-2 for locations of these subsegments). Seasonal fluctuations in water temperature are evident with warmer temperatures during the summer months and cooler temperatures during the winter period. For example, the monthly average Mississippi River water temperature at Belle Chasse (subsegment 070301) ranged from 6.6 °C (43.9 °F) in January to 30 °C (86 °F) in August between 1977 and 2017 (see Table 4.2.1-2). LDEQ's 2016 Water Quality Integrated Report indicated that all 3 Mississippi River subsegments within the proposed action area meet the temperature standards criteria.

The maximum LDEQ water quality standards for temperature in all Barataria Basin subsegments within the proposed action area are either 32 °C or 35 °C (90 °F or 95 °F). Aggregate average water temperatures in the Barataria Basin between 2006 and 2018 ranged from 13 °C (55 °F) in January to 30 °C (86 °F) in July and August and do not exceed the criteria. Cooler temperatures were evident during winter (December to February) compared to summer months (June to September). There is a substantial range in temperature at all CRMS sites, demonstrating the influence of regional weather patterns on water temperature in the basin. A monthly comparison of water temperatures demonstrates consistently warmer temperatures in the Barataria Basin when compared with the Mississippi River. Turner et al. (2017) found the annual average temperature in the Barataria Basin at any station in his study to be 21 to 22 °C (about 70 to 72 °F).

4.2.4 Dissolved Oxygen (DO)

DO is a measure of the amount of oxygen that is dissolved within the water column; DO is a requirement for most forms of aquatic life. Water temperature and specific conductance directly impact the DO capacity within a system. In the absence of effects from biological communities, lower DO values are observed when water temperatures are higher and are often higher when water temperatures are lower. Similarly, a more saline environment can result in lower DO values, as salinity influences the solubility of oxygen in water. In addition to these physical factors, biological processes (animal and plant respiration and organic material decomposition) utilize DO, which can in turn reduce the DO available to sustain aquatic life. Excessive nutrient (nitrogen and phosphorus) loads create algal blooms which in turn deplete the bottom water DO levels due to photosynthetic processes and the decomposition of the organic material. This creates hypoxic conditions, or "dead zones" that persist for a long time and can be detrimental for immobile organisms, such as oysters, which are unable to retreat to areas with higher concentrations of DO. These hypoxic events occur when DO concentrations are extremely low (less than 2 milligrams per liter [mg/L]) (see Figure 4.2.4-1; Rabalais et al. 1995, 2002; Turner and Rabalais 2017).

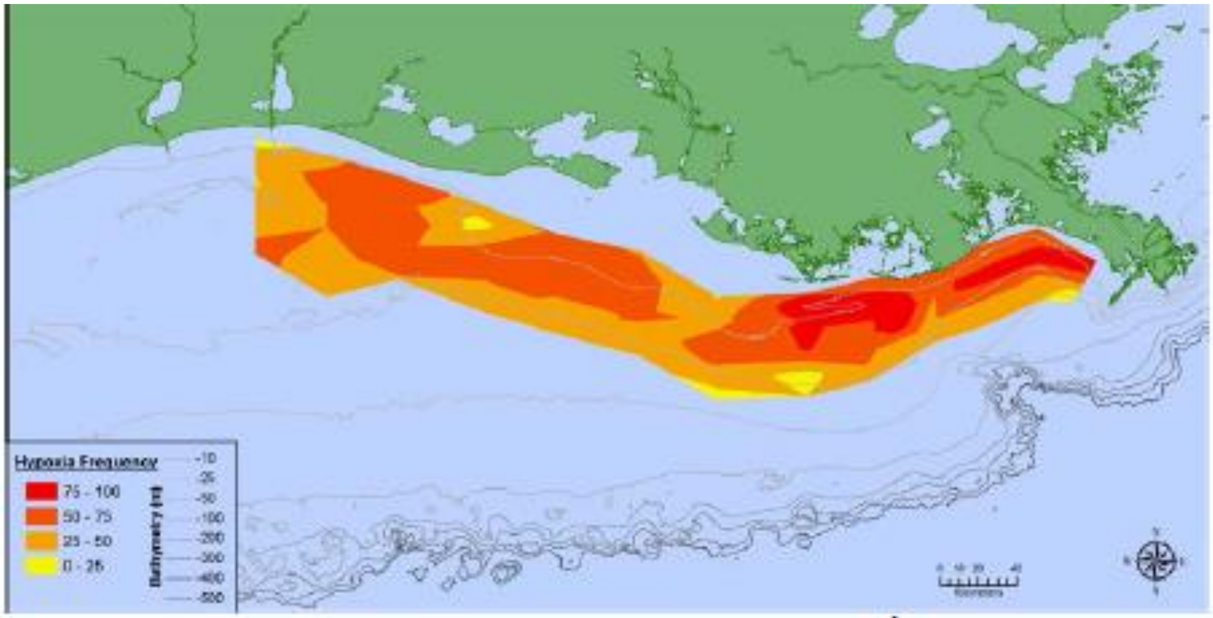


Figure 4.2.4-1. Frequency of Mid-Summer Hypoxia (oxygen ≤ 2 mg/L) (1985-2014) over the 70 to 90 Station Grid on the Louisiana and Texas Shelf during the summer from 1985 to 2014.
(Source: Turner and Rabalais 2017).

In the Mississippi River, DO concentrations fluctuate with temperature, with higher concentrations when water temperatures are cooler (see Figure 4.2.4-2). An analysis of LDEQ data showed that DO concentrations do not correlate with river flow. Average monthly DO concentrations ranged from 5.9 mg/L (July) to 12 mg/L (January) in the Mississippi River at Belle Chasse between 1977 and 2017. Individual sample concentrations fell below the water quality standard of 5.0 mg/L in the summer months of July, August, and September. At the monitoring station at West Pointe a la Hache, average monthly DO concentrations ranged from 5.9 mg/L (August) to 11 mg/L (February) between 1977 and 2017, with individual concentrations falling below the 5.0 mg/L standard in July only.

Many of the subsegments in the Barataria Basin have site-specific seasonal standards for DO, ranging from 2.5 mg/L to 5.0 mg/L in selected months (LAC:IX.1123.Table 3). An analysis of the LDEQ data in the basin showed that DO average monthly concentrations ranged from 6.1 mg/L (August) to 10 mg/L (January) between 2000 and 2017. Individual concentrations fell below 5.0 mg/L in May, June, and August (see Figure 4.2.4-2).

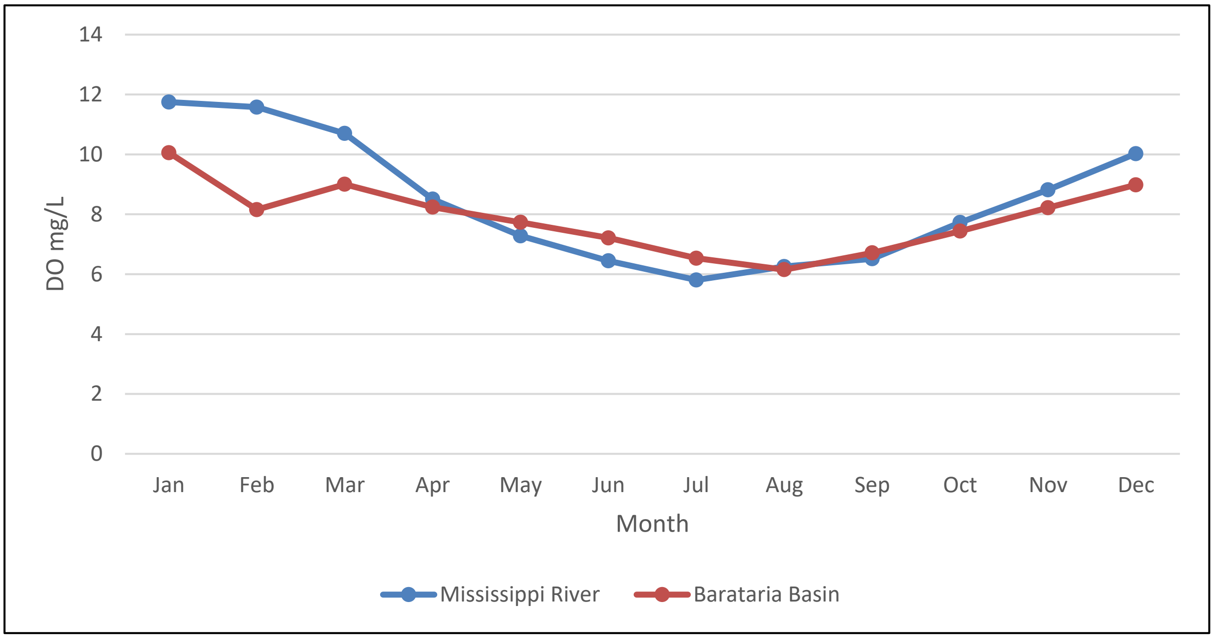


Figure 4.2.4-2. Monthly DO Average Concentrations in the Mississippi River at Belle Chasse (1977-2017) and at Select Barataria Basin Sites (2006-2018).

Starting in December 2014, LDEQ collected DO profiles within the Barataria Basin Coastal Bays and Gulf Waters subsegment (021102_00) in order to assess the impairment status of this subsegment, evaluate temporal data trends, and compare the Barataria Basin to neighboring systems such as the coastal bays and Gulf waters of the Mississippi River (070601_00) and Terrebonne Basin (120806_00, LDEQ 2016). Both the Barataria and Mississippi River subsegments failed to meet the DO criterion of 5.0 mg/L for subsegment 021102_00, with 36.7% and 42.7% of the values below the state standard, respectively (LDEQ 2016). LDEQ's vertical profile data provided evidence that depressed DO values co-occurred with rapid increases in salinity indicative of a halocline (vertical gradient in salinity). It is likely that the influence of the Mississippi River resulted in abrupt salinity stratification and a subsequent DO decline within the deeper (more saline) waters. Excessive nutrient loading from the Mississippi River is also suspected as a cause of such DO declines.

4.2.5 Turbidity

Turbidity is an optical measure of the amount of suspended particles within the water column, which can affect water clarity. A decline in water clarity due to increased turbidity reduces light penetration within the water column, which can adversely impact primary productivity (for example, phytoplankton production). Turbidity is primarily influenced by total suspended solids (TSS) and colored dissolved organic material. Louisiana's turbidity criterion (LAC 33:IX.1113.B.9) states that turbidity other than that of natural origin shall not cause substantial visual contrast with the natural appearance of the waters of the state or impair any designated water use, and that turbidity shall not significantly exceed background. The established turbidity standard for the Mississippi River is 150 nephelometric turbidity units (NTU). The established turbidity standard for estuarine waterbodies is 50 NTU. In other state waters, turbidity in NTU caused by any discharges shall be restricted to the appropriate background value plus 10%. LDEQ has identified a number of waters in the Barataria Basin as being impaired due to excessive turbidity.

Average monthly turbidity concentrations at the Belle Chasse station in the Mississippi River ranged from 22 NTU in September to 84 NTU in March between 1978 and 2017. Turbidity concentrations at Belle Chasse exhibit a positive linear correlation with flow. At West Pointe a la Hache, average turbidity concentrations ranged from 12 NTU in September to 70 NTU in February between 1971 and 1998.

In the Barataria Basin, the average monthly turbidity concentrations over the period of 2000 to 2017 ranged from 10 NTU in August to 40 NTU in January. Average turbidity concentrations are lower in the summer and fall (July - October) than at other times of the year. Comparatively, the Mississippi River typically has higher turbidity concentrations than the Barataria Basin.

4.3 Sediment Quality

4.3.1 Mississippi River

The Mississippi River carries dissolved and suspended contaminants and bacteria that originate from a variety of municipal, agricultural, and industrial sources. The distribution of contaminants along the Mississippi River depends on the nature and location of their sources and the degree of wastewater treatment and organic contaminants such as polychlorinated biphenyls (PCBs) and inorganic contaminants such as lead, which are more likely to adhere to sediment particles than to remain in the dissolved phase (Meade 1995). The USGS summary of contaminant levels in the Mississippi River for the period 1987 to 1992 (Meade 1995) found that contaminant concentrations in suspended and bed sediments decreased from the northern to the southern regions of the drainage basin as a result of dilution with uncontaminated materials, evaporative losses, losses due to dissolution in water, chemical and microbial breakdown, and the geographic distribution of chemical discharges. Metals naturally occur in sediments; the highest concentrations of contaminant metals are mostly found in coastal areas close to human activities that release such metals (Kenicutt 2017).

In support of federal dredging projects performed for navigation channel maintenance, Mississippi River sediment quality is periodically assessed in various locations from Baton Rouge to Head of Passes (RM 0.0) to determine the presence of contaminants in river sediment and the potential for contaminant release at dredged material disposal areas (which are often in offshore locations). Periodic maintenance dredging, as frequent as once a year in some locations, is performed with hopper and cutterhead dredges. The CEMVN is responsible for evaluating the proposed discharge of dredged material, and the testing procedures are performed according to the Regional Implementation Agreement (RIA) for Evaluating Dredged

Material Proposed for Ocean Disposal Off the Louisiana Coast (1992) as well as current national guidance jointly developed by USEPA and USACE.

The RIA provides a list of potential contaminants of concern (COCs) to be included in the chemical analyses, which include USEPA Priority Pollutants. COCs typically analyzed include metals, polycyclic aromatic hydrocarbons (PAHs), pesticides, organonitrogen compounds, chlorinated hydrocarbons including but not limited to PCBs, total organic carbon (TOC), and ammonia. Tests for physical parameters include percent solids/total solids and grain size analysis. The chemical analyses of the channel sediment and elutriate samples indicate any expected release of potential toxins from the sediment into the water column. The suspended particulate phase bioassays are designed to determine the potential impact to sensitive water column organisms from dredging and ocean placement. The solid phase bioassays are designed to determine the potential impact of the placement of the dredged material on designated sensitive marine organisms living on the bottom of the Gulf of Mexico. The bioaccumulation studies are designed to indicate any uptake of potential toxins by sensitive benthic, or bottom crawling organisms. Physical analysis of the dredged material provides general information on the physical characteristics of the dredged material and can assist in assessing the impact of disposal on the benthic environment and the water column at the disposal site.

The following sediment quality evaluations were performed in support of federal navigation channel maintenance projects; they provide information on the general conditions of sediment quality in the Mississippi River in proximity to the proposed Project intake structure:

- Mississippi River-Southwest Pass Louisiana Contaminant Assessment (CEMVN 2007);
- Contaminant Assessment Mississippi River, Baton Rouge to the Gulf of Mexico, Louisiana Southwest Pass (CEMVN 2009);
- Contaminant Assessment Mississippi River-Southwest Pass Louisiana (CEMVN 2011);
- Mississippi River, Baton Rouge to the Gulf of Mexico, Louisiana Navigation Project Southwest Pass Ocean Dumping Evaluation (CEMVN 2016); and
- Evaluation of Dredged Material Collected from the Deep-Draft Crossings of the Mississippi River (CEMVN 2017).

Individual COCs analyzed for each of the assessments are provided in Table 4.3.1-1. The assessments performed in 2007, 2011, and 2016 evaluated chemical, physical, and biological test data for the following media: water, sediment, elutriate, and tissue (bioaccumulation testing). The 2009 and 2017 assessments included chemical and physical analyses only.

Table 4.3.1-1 Parameters for Dredge Sediment Quality Evaluations: USEPA Priority Pollutants, Contaminants of Concern (COC), and Conventional Parameters

Metals and Cyanide	**LPAH and HPAH Compounds**	**Pesticides**
Antimony (Total)[c]	Acenaphthene [b]	Aldrin [c]
Arsenic (Total) [c]	Acenaphthylene [b]	Alpha-BHC [c]
Beryllium (Total) [c]	Anthracene [b]	Beta-BHC [c]
Cadmium (Total) [c]	Fluorene [b]	Gamma-BHC (Lindane) [c]
Chromium (Total) [c]	Naphthalene [b]	Delta-BHC [c]
Chromium (+3) [c]	Phenanthrene [b]	Chlordane [c]
Chromium (+6) [c]	Benzo(a)anthracene [b]	4,4'-DDD [c]
Copper (Total) [c]	Benzo(a)pyrene [b]	4,4'-DDE [c]
Cyanide (Total) [c]	Benzo(ghi)perylene [b]	4,4'-DDT [c]
Lead (Total) [b]	Benzo(b & k)fluoranthene [b]	Dieldrin [c]
Mercury (Total) [b]	Chrysene [b]	Alpha-endosulfan [c]
Nickel (Total) [b]	Dibenzo (a,h) anthracene [b]	Beta-endosulfan [c]
Selenium (Total) [c]	Fluoranthene [b]	Endosulfan sulfate [c]
Silver (Total) [c]	Indeno (1,2,3-cd) pyrene [b]	Endrin [c]
Thallium (Total) [c]	Pyrene [b]	Endrin aldehyde [c]
Zinc (Total) [c]	2-Methylnaphthalene [d]	Heptachlor [c]
Vanadium (Total)[d]		Heptachlor epoxide [c]
	Chlorinated Hydrocarbons	Toxaphene [c]
Conventional Parameters	1,2-Dichlorobenzene [c]	
Total Organic Carbon [c]	1,3-Dichlorobenzene [c]	**PCBs**
Total Petroleum Hydrocarbons [b]	1,4-Dichlorobenzene [c]	Total PCBs [b]
Ammonia[nt]	2-Chloronapthalene [c]	PCB-1242 [b]
Percent Solids/Total Solids[c]	Hexachlorobenzene [c]	PCB-1254 [b]
Oil&Grease[d]	Hexachlorobutadiene [c]	PCB-1221 [b]
Organic Compounds	Hexachlorocyclopentadiene [c]	PCB-1232 [b]
Phenols/Substituted Phenols [c]	Hexachloroethane [c]	PCB-1248 [b]
2-Chlorophenol [c]	1,2,4-Trichlorobenzene [c]	PCB-1260 [b]
2,4-Dichlorophenol [c]		PCB-1016 [b]
2,4-Dimethylphenol [c]	**Phthalate Esters**	
4,6-Dinitro-o-Cresol [c]	Bis(2-ethylhexyl) phthalate [c]	**Organonitrogen Compounds**
2,4-Dinitrophenol [c]	Butyl benzyl phthalate [c]	Benzidine [c]
2-Nitrophenol [c]	Diethyl Phthalate [c]	3,3'-Dichlorobenzidine [c]
4-Nitrophenol [c]	Dimethyl Phthalate [c]	2,4-Dinitrotoluene [c]
p-Chloro-m-Cresol [c]	Di-n-Butyl Phthalate [c]	2,6-Dinitrotoluene [c]
Pentachlorophenol [c]	Di-n-octyl Phthalate [c]	1,2-Diphenylhydrazine [c]
Phenol [c]		Nitrobenzene [c]
2,4,6-Trichlorophenol [c]	**Halogenated Ethers**	N-nitrosodimethylamine [c]
	Bis(2-chloroethoxy) methane [c]	N-nitrosodi-n-propylamine [c]
Miscellaneous	Bis(2-chloroethyl) ether [c]	N-nitrosodiphenylamine [c]
Isophorone [c]	Bis(2-chloroisopropyl) ether [c]	
	4-Bromophenyl phenyl ether [c]	
	4-Chlorophenyl phenyl ether [c]	

a - Evaluations included: Mississippi River-Southwest Pass Louisiana Contaminant Assessment – 2007, Contaminant Assessment Mississippi River, Baton Rouge to the Gulf of Mexico, Louisiana Southwest Pass – 2009, Contaminant Assessment Mississippi River-Southwest Pass Louisiana – 2011, and Mississippi River, Baton Rouge to the Gulf of Mexico, Louisiana Navigation Project Southwest Pass Ocean Dumping Evaluation – 2016

b - Parameters analyzed in all 5 assessments; c - Parameters analyzed only in 2007, 2011, 2016, and 2017;

d - Parameters analyzed only in 2009 assessment

In the 2007 assessment, organic compounds, pesticides, and PCBs were not detected, and all metals detected in water and elutriate samples were less than the USEPA Water Quality Criteria (WQC) and state Water Quality Standards (WQS). All metals detected in sediment samples were less than the NOAA Effects Range Low (ERL) standards. The ERL standard is the concentration of a chemical in sediments that resulted in biological effects approximately 10% of the time based on the literature (Kenicutt 2017). Although there was potential for bioaccumulation shown for 1 metal analyte by 1 organism at 1 sampling station, no definitive ecological effects were determined. Thallium was detected in 1 water sample and in most sediment samples, but this contaminant does not have a WQC or WQS for water samples and does not have a NOAA ERL.

In the 2009 assessment, lead, nickel, and vanadium were detected in water, elutriate, and sediment samples. All lead and nickel concentrations were less than the respective WQC and WQS, but vanadium does not have WQC or WQS. Organic COCs were not detected in water and elutriate samples, but fluoranthene, pyrene, and oil and grease were detected in sediment samples. Fluoranthene and pyrene were only detected in 2 sediment samples, and concentrations were less than the NOAA ERL. Oil and grease does not have a NOAA ERL for sediments.

In the 2011 assessment, copper concentrations exceeded WQS values in 4 out of 9 channel samples and exceeded WQC values in 2 out of 9 channel samples. One elutriate sample exceeded the WQC value for ammonia. All metal concentrations in sediment samples were less than their respective NOAA ERL values. Thallium and selenium do not have NOAA ERL values, but were detected in sediment samples in very low concentrations. Except for acenaphthene and fluorine in 1 sediment sample, all organic compounds detected in sediment samples were less than the NOAA ERL values; acenaphthene and fluorene were not detected in the other 9 channel sediment samples. Benzo(b)fluoranthene and benzo(k)fluoranthene were also detected in several sediment samples; however, a NOAA ERL is not provided for the fore-mentioned PAHs. Benzo(b)fluoranthene was detected in 5 out of the 9 channel sediment samples, and benzo(k)fluoranthene was detected 2 of the 9 channel sediment samples.

In the 2016 assessment, besides copper and silver, all metals detected in water and elutriate samples were less than the WQC. Organic COCs were not detected in water or elutriate channel samples. The concentration of silver detected in dredging elutriates exceeded the regulatory WQC but was less than concentrations observed in ambient Ocean Dredged Material Disposal Site (ODMDS) waters. The CEMVN recommended a follow-up analysis for copper and silver to determine if elevated concentrations of copper were anomalous to this evaluation or related to high river stage. Ammonia was detected in dredging elutriates at concentrations greater than WQC. All detected metals, PAHs, and pesticides (DDT only) were less than NOAA ERL values except for 4,4′-DDT. All concentrations for 4,4′-DDT were less than the NOAA Effects Range-Median (ERM) standard.

In the 2017 assessment, antimony, arsenic, chromium, copper, nickel, selenium, and zinc were detected in all liquid samples. Lead, silver, and mercury were detected but less frequently than the fore-mentioned contaminants, and at concentrations near analytical detection limits. Nearly all metals were detected at concentrations below their respective WQC or WQS, with the exception of zinc. The pesticides Aldrin, Endrin ketone, and alpha-BHC were detected in liquid samples collected at 2 sites. All pesticide detects were at parts per trillion concentrations, and the fore-mentioned Aldrin detect was several orders of magnitude below the WQC. No other organic pollutants were detected in the liquid fraction of the dredged material.

The metals arsenic, beryllium, cadmium, chromium, copper, lead, nickel, selenium, thallium, and zinc were detected in sediment samples from all sites. Mercury and silver were observed less frequently in sediments, and at concentrations at or near analytical detection limits. All detected metals in sediments were at concentrations below NOAA's Threshold Effect Level (TEL) screening values for freshwater sediments. Note that NOAA TEL values are less than NOAA ERL values and that freshwater TELs are less than marine TELs. The PAHs naphthalene, acenaphthylene, benzo(a)pyrene, and PCB-1248 were detected in sediments collected at individual sites, but the concentration of all PAHs and PCBs detected in sediment samples were less than available TELs. Low concentrations of chlordane pesticides were detected in sediments collected at several sites. The pesticides 4,4′-DDD and 4,4′-DDE were detected at low concentration at several sites, but were present in concentrations less than available TELs.

In addition to the federal studies, CPRA conducted sediment sampling in the Mississippi River in 2009 in support of the Bayou Dupont marsh restoration project. River bottom sediments were sampled from the borrow area north of Myrle Grove and from reference area near New Orleans, as well as from the placement area which is located in the Project area. The Sediment Testing of Dredging Material Proposed for the Mississippi River Sediment Delivery System-Bayou Dupont (BA-39) Project report (CPRA 2009) was reviewed. Sediment samples were analyzed for grain size, polycyclic aromatic hydrocarbons (PAHs), PCBs, metals (lead, nickel, mercury and vanadium), total petroleum hydrocarbons (TPH), total organic carbon, and oil and grease. Solid phase bioassay/benthic toxicity tests were also conducted on sediments from the borrow, reference and placement areas. The study concluded that fluorene, dibenzo(a)anthracene (a PAH), and total PAHs exceeded Screening Quick Reference Tables (SQuiRTs) concentrations protective of marine life; however, the bioassay results determined that there was no significant difference between mortality to organisms exposed to the borrow and fill area sediments and those exposed to the reference sediment. Therefore, the dredged material is predicted not to be acutely toxic to benthic organisms.

Although the above sediment assessments do not provide sediment quality data for sediments that would necessarily be transported to the Barataria Basin via the proposed Project diversion structure, the reports document general conditions of sediment quality in the Mississippi River close to the proposed Project intake structure, both north and south. The above reports concluded that the Mississippi River sediments evaluated are free from COCs at concentrations that would result in detrimental impacts from placement of dredged sediments in either the Mississippi River, Barataria Basin, or associated ODMDS. The consistency in these findings provide some indication of the capacity of the Mississippi River to dilute both dissolved contamination and contamination bound to sediments. With the exception of the Bayou Dupont study, interpretation of the conclusions of the above reports is limited, however, because the reports draw conclusions for the specific disposal of sediments into either the Mississippi River or an ODMDS where currents (including littoral currents at ODMDS), waves, and tides can rework and/or transport disposed sediments and potentially aid in contaminant dilution. Additionally, conclusions of the above reports consider dilution models that would likely require modification to be applicable to the Project outfall area. The Project is designed to deliver sediments to an area for deposition which has lower water energy conditions than the Mississippi River or an ODMDS and likely a significantly lower dilution potential. The Bayou Dupont study evaluated the placement of Mississippi River sediment dredged immediately upriver of the Project and placed into the eastern Barataria Basin in the Project area. Although some COCs were detected in the River sediments, the study concluded that the sediments would not be acutely toxic to benthic organisms. Mississippi River sediment quality is dependent upon occurrence and conditions of point source and nonpoint source pollution, and is subject to significant change over time. Nonetheless, these assessments provide a snapshot of the types and concentrations of COCs known to be present in Mississippi River sediments.

4.3.2 Barataria Basin

As part of a larger review of sediment quality data in the Northern Gulf of Mexico and adjacent national estuaries, Kennicutt (2017) reviewed sediment contamination data collected from 2000 to 2001 in order to rate Gulf and estuarine sediments using a sediment quality index. The index was a composite indicator based on sediment toxicity, contaminants, and TOC content. Index ratings of sediment contaminants were defined as good if no sampled contaminants at any sample sites exceeded NOAA effects range median (ERM) values and fewer than 5 NOAA ERL values were exceeded; fair if 5 or more ERL values were exceeded; and poor (red) if 1 or more ERM values were exceeded. ERM value is the concentration of a chemical in sediments that resulted in biological effects approximately 50% of the time based on the literature. As stated earlier, the ERL value is the concentration of a chemical in sediments that resulted in biological effects approximately 10% of the time based on the literature (Kenicutt 2017).

Using this index, the Barataria-Terrebonne Estuarine Complex, of which the proposed action area is a part, was rated as good, with 8% of the estuarine area rated as poor. Sediment contaminant content in the estuarine complex was rated good overall, with 4% of the area rated poor (Figure 4.3.2-1). Kenicutt (2017) rated 2 locations in the complex as poor mostly because of localized, elevated TOC concentrations, and rated all sediment ratings within the action area as good.

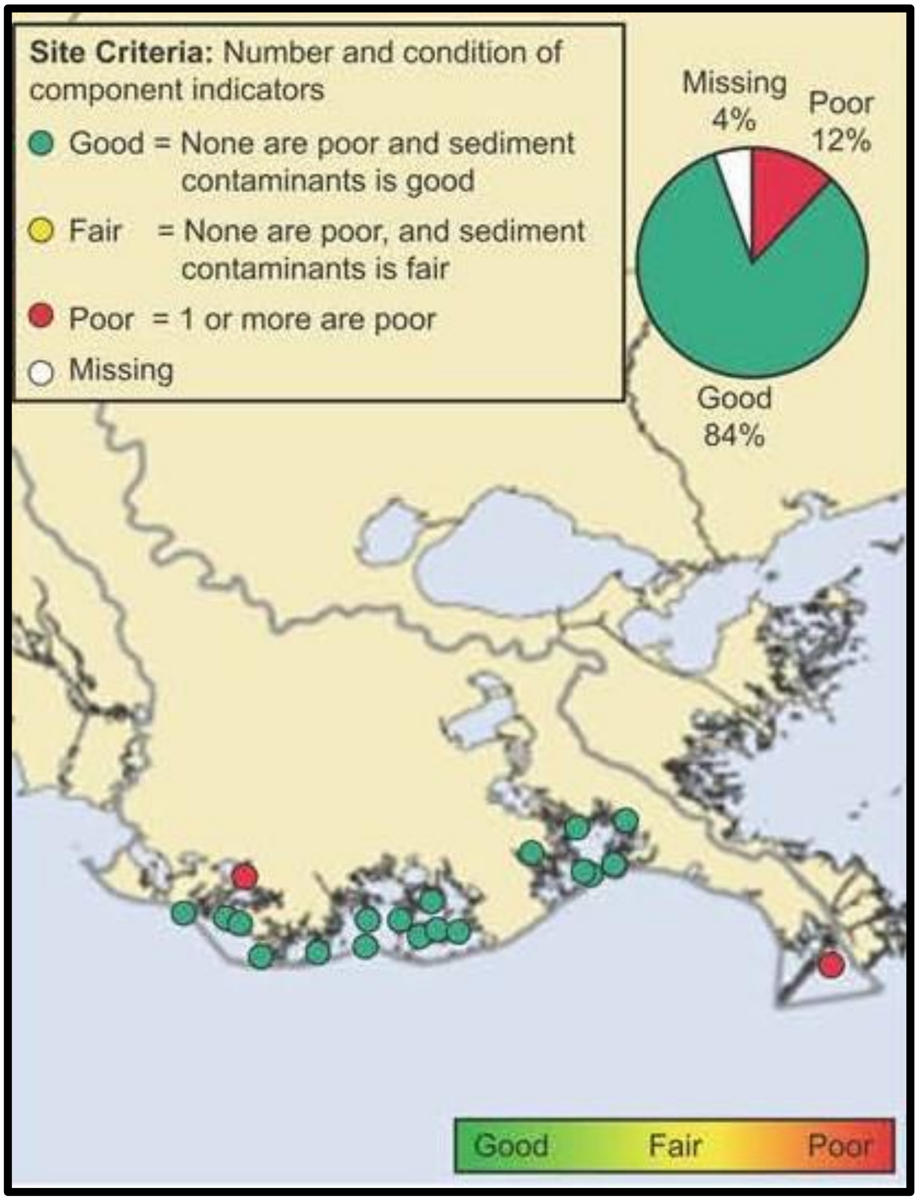


Figure 4.3.2-1. Sediment Quality Index Ratings for Barataria-Terrebone Estuarine Complex (Source: Kenicutt 2017)

Federal navigation maintenance/dredging projects performed on the Barataria Bay Waterway provide additional sediment quality data within the action area outside of the Mississippi River. The Barataria Bay Waterway runs from Bayou Villars, near Jean Lafitte, to Grand Isle, entering Barataria Bay approximately 11.2 km (7 miles) south of the proposed Project outfall area.

Historically, sediments generated in the construction and maintenance of the waterway have been disposed of in open water areas adjacent to the channel, wetland development disposal areas, upland confined disposal areas or beneficial use sites along east and west banks of the waterway, and sites such as the Barataria Bay Waterway ODMDS (bar channel) (CEMVN 2017c). Additional sediment quality data within the proposed action area has been generated in support of the Fifi Island beneficial use/wetlands creation project near Grand Isle (Russo et al. 2014) and through evaluation of impacts from the DWH oil spill to the Barataria Bay Waterway (CEMVN 2010).

The Barataria Bay Waterway ODMDS Site Management Plan (CEMVN 1998) discusses historic sediment quality trends for the bar channel (mile 0 to mile -3.8). The plan states that sediments sampled in 1991 and 1994 were of sufficient quality for disposal at the ODMDS. Sediment sampling was performed in 2002 in support of the Fifi wetlands creation/maintenance dredging project; the sampling on the Bayou Rigaud (north of Grand Isle) portion of the Barataria Bay Waterway revealed that only ammonia was present at levels requiring action; the beneficial use/wetlands creation project was installed to use the dredge sediments in a beneficial way that would also result in mitigation of ammonia.

The bar channel reach of the Barataria Bay Waterway was evaluated for impacts from the DWH oil spill in 2010. Analytes indicative of oil contamination were present in shoal material only in trace amounts, and at concentrations that are not expected to adversely impact benthic organisms. The CEMVN concluded that additional biological effects-based testing was not warranted and special management of dredged material was not required for channel maintenance. The majority of the length of the bar channel contains a high percentage of clay and silt; ODMDS surface sediments consist of sand (CEMVN 1980). Interpretation of this data for documentation of sediment quality within the Project outfall area is subject to limitations. The Barataria Bay Waterway bar channel and ODMDS are about 38.6 km (24 miles) south/southeast from the Project's outfall area, and Barataria Bay Waterway sediment quality is documented for in-channel sediments. Navigation channel sediment sources and depositional environment(s) vary from those existing in the vicinity of Project features.

The Sediment Testing of Dredging Material Proposed for the Mississippi River Sediment Delivery System-Bayou Dupont (BA-39) Project evaluated sediment in the marsh creation placement area in the eastern Barataira Basin near the Project oufall. Sediments sampled from the placement area contained naphthalene in excess of SQuiRTs concentrations protective of marine life, as well as detectable concentrations of PAHs, lead, nickel, vanadium, and TPH. As previously noted, there was no significant difference in mortality to benthic organisms exposed to the fill area sediments and those exposed to the reference and borrow sediments.

4.4 Historical and Existing Wetland Habitat and Deltaic Processes in Barataria Basin

4.4.1 Coastal Zone

The Coastal Zone Management Act (CZMA) calls for the effective management, beneficial use, protection, and development of the nation's coastal zone and promotes active state involvement in achieving those goals. To reach those goals, the CZMA requires participating states to develop management programs that demonstrate how those states will meet their obligations and responsibilities in managing their coastal areas. In Louisiana, the Louisiana Department of Natural Resources (LDNR) Office of Coastal Management (OCM) administers the Coastal Zone Management Program (LDNR 2017a). The inland boundary of the Louisiana Coastal Zone was most recently delineated in the 2012 Regular Session of the Louisiana Legislature with the passage of House Bill 656 (Act 588) and consists of all or part of 20 coastal parishes. The proposed action area is located entirely within the 2012 Louisiana Coastal Zone (LDNR 2017b).

4.4.2 Watershed Characterization

The action area is defined by the boundaries of the Barataria Basin and the Lower Mississippi River watersheds identified by USGS as the East Central Louisiana and Lower Mississippi River Hydrologic Units (HUCs; HUC 08090301 and 08090100, respectively) (USGS 2017) (see Figure 4.4.2-1). The majority of the Barataria Basin consists of low-relief coastal bays, lakes, and deltaic marshes between Bayou Lafourche and the Mississippi River within the East Central Louisiana watershed. Surface waters in the East Central Louisiana watershed are largely influenced by estuarine and oceanic waters of the Gulf of Mexico. The flow of fresh surface water into the Barataria Basin has been reduced due to the construction and maintenance of flood control levees along the Mississippi River and other modifications explained in Section 4.4.1 above. At present, diversion projects introduce a small amount of water into the basin from the Mississippi River.

The lower Mississippi River watershed above Venice consists only of the river channel and the adjacent levees. Below Venice, the watershed widens to include the coastal bays, passes, levees, and deltaic marshes of the river delta, which still receives the flow of fresh surface water from the Mississippi River. Some portions of the flow of the Mississippi River within the Birdfoot Delta have been diverted for marsh creation and restoration projects. Surface water flow in the lower Mississippi River watershed is generally dominated by the Mississippi River itself except during very low river flows, during which time it is influenced more by estuarine and oceanic waters of the Gulf (Wells 1980). Each of these HUCs is further subdivided by the USGS into finer sub-basins, as depicted in Figure 4.4.2-1.

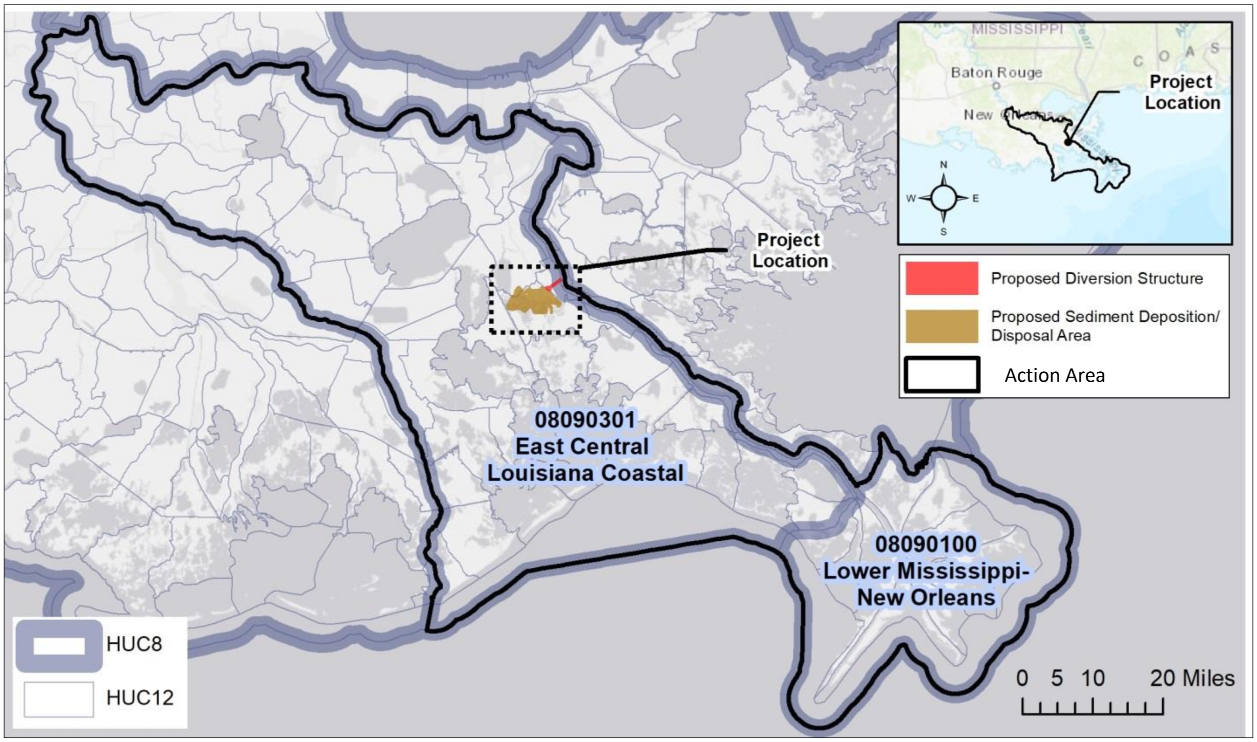


Figure 4.4.2-1. Map of HUC8- and HUC12-level basins and Sub-Basins within the Action Area

4.4.3 Waterbodies in the Action Area

The Barataria Basin is delineated by the natural levees that were formed by Bayou Lafourche and the Mississippi River. A chain of barrier islands separates the basin from the Gulf of Mexico. In the northern half of the basin several large lakes occupy the lower lying areas approximately halfway between the ridges. The southern half of the basin consists of tidally influenced marshes connected to a large bay system behind the barrier islands.

Waterbodies within the Barataria Basin include numerous lakes (Lac des Allemands; Lakes Boeuf, Cataouatche, Salvadore, and Little Lake), Caminada Bay, and Barataria Bay (see Figure 4.1-1). In addition, the USACE maintains major navigation channels in the proposed action area. These include the Mississippi River, the GIWW, the Barataria Bay Waterway, and Bayou.

4.4.4 Hydrology and Hydrodynamics

Historical Context

The Mississippi is a massive river system, draining over 768 million acres covering parts of 31 states and 2 Canadian Provinces (Alexander et al. 2012). The Barataria Basin was an active sublobe of the St. Bernard delta complex and lies between the natural levees that were formed by Bayou Lafourche and Bayou des Familles (Frazier 1967, LDWF 2015). The basin was supplied with fresh water, sediment, and nutrients from the Mississippi River, through both direct connection to the river and seasonal overbank flooding. The primary connection between the Barataria Basin and the Mississippi River—Bayou Lafourche—was closed off in 1904 when a dam was built across the head of Bayou Lafourche in Donaldsonville, cutting off all flow from the Mississippi River (van Heerden et al. 1996). Continued channelization of the main channel of the Mississippi River and increasing levee heights during the 1930s and 1940s further isolated the Barataria Basin from fresh water and sediment carried by floodwaters that historically overflowed into the wetlands (Alexander et al. 2012, Conner and Day 1987).

Bathymetry

Elevation data for dry land is termed topography and for land below the water surface is termed bathymetry. Figure 4.4.4-1 shows the bathymetry of Barataria Basin developed for the current hydrodynamic model known as the Delft model version 2 (Liang et al. 2016). Elevations in the area are highest along the Mississippi River levee and lowest in navigation channels.

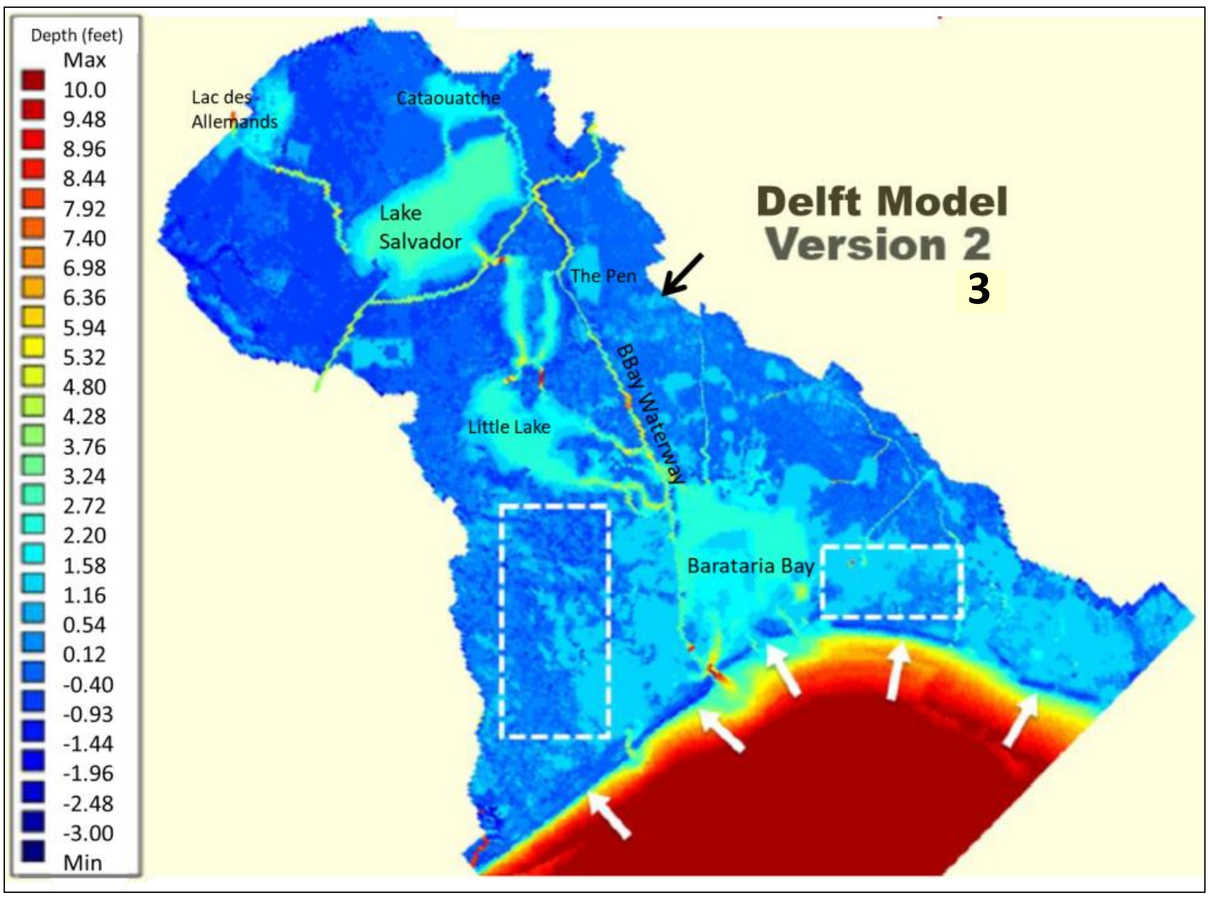


Figure 4.4.4-1. Barataria Basin Model Grid Bathymetry from the Delft Model Version 3.

Figure is adapted from Liang et al. 2016 which used white boxes to demonstrate improved bathymetry in regions. The black arrow marks the approximate location of the proposed Project diversion structure. The color red indicates the deepest areas and blue indicates the highest elevations. Negative numbers in dark blue indicate land above mean water level.

Elevations in Lac des Allemands, which covers about 4,856 hectares (12,000 acres), range from -1.8 meters to 3.0 meters MHHW (-6 feet to -10 feet) (Figure 4.4.4-1, Meselhe et al. 2015). The 2 primary waterbodies in the center of the basin are the 6,070 hectare (15,000-acre) Lake Salvador and 3,237 hectares (8,000-acre) Lake Cataouatche (Figure 4.4.4-1). The latter is the receiving body for the Davis Pond Freshwater Diversion Project outfall. Bed elevations for both lakes also range from approximately -1.8 meters to -3.0 meters (-6 to -10 feet) (Meselhe et al. 2015).

Extending south from the GIWW to the Gulf of Mexico, the Barataria Basin contains numerous bayous and open water. The largest areas of open water are Little Lake and Barataria Bay. From Lake Salvador, water flows through Bayous Perot and Rigolettes into Little Lake and then into Barataria Bay. Elevations of Little Lake and Barataria Bay are approximately -0.9 meters to -1.8 meters (-3 to -6 feet), with Bayou St. Denis and Grand Bayou with areas at -6.4 meters (-21 feet) (OCS 2017). The Barataria Bay Waterway runs between The Pen and Bayou Rigolettes, past Little Lake, and through Barataria Bay (Figure 4.4.4-1). It is a major conveyance channel and acts as a conduit for saltwater intrusion. Survey cross-sections conducted in 2011 showed that most of the land elevations in this region were about 0.3 meters to 0.6 meters (1 foot to 2 feet) (Baker Smith 2011). The deepest portions of the Barataria Basin are at the passes between the barrier islands separating Barataria Bay from the Gulf of Mexico. Barataria Pass, between Grand Isle and Grand Terre Island as shown in the NOAA chart 11358, has depths over 24.4 meters (80 feet) (OCS 2017). Other passes, like Caminada Pass and Quatre Bayou Pass, have depths near 6.1 meters (20 feet).

4.4.5 Water Levels

Water levels in the Barataria Basin are influenced by tides from the Gulf of Mexico, wind, and rainfall. A high wind event at Grand Isle on June 21, 2017 increased water surface elevation by almost 0.6 meters (2 feet), as shown in Figure 4.4.5-1. This wind effect has been documented throughout the Louisiana coast (for example, Moeller 1993, Walker 2001, and Li et al. 2010), and occurs primarily when winds are blowing from the south or southeast as they “stack up” water in the bay. Northerly or westerly winds have the opposite effect and lower water levels in the Barataria Basin by effectively pushing water out of the basin towards the Gulf.

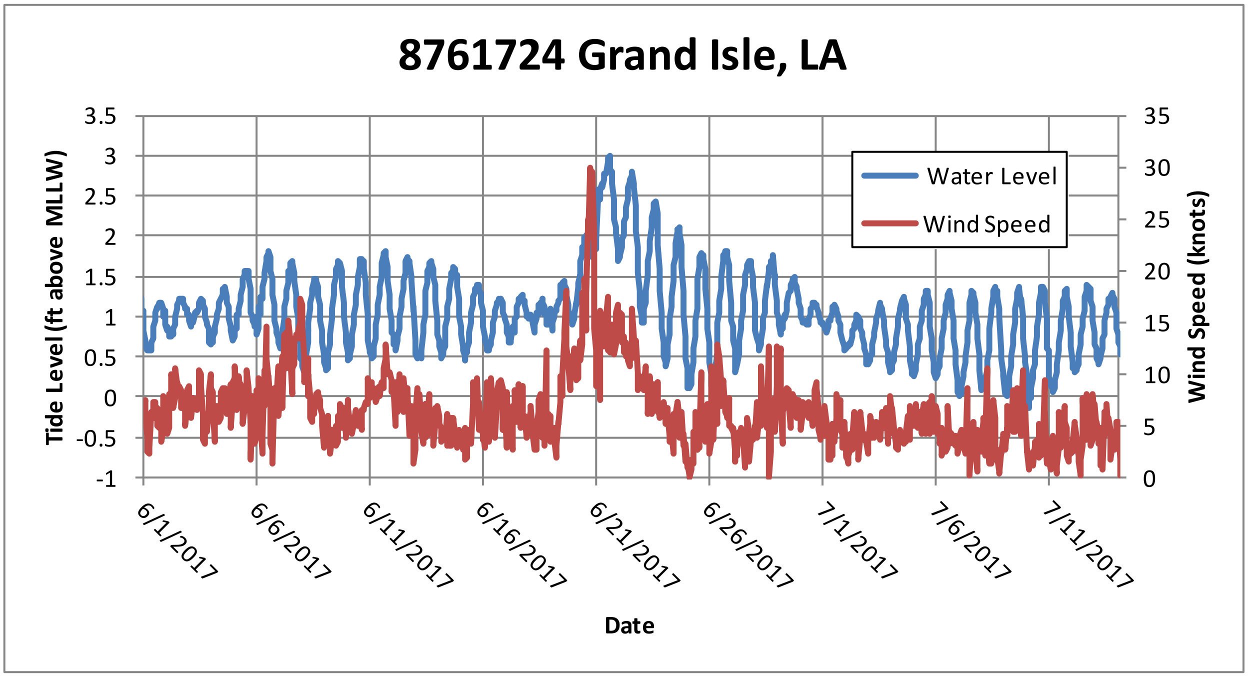


Figure 4.4.5-1. Water Level and Wind Speed at the Grand Isle, LA, Station.

USGS gages and CRMS stations throughout the Barataria Basin report daily or hourly water levels, most of which are referenced to the NAVD88 datum (USGS 2017, CPRA 2017). As shown in Table 4.4.6-1, average water levels within Barataria Basin are generally about 1 foot.

4.4.6 Tides, Currents, and Flow

Tides

Tides are the cyclical rising and falling of water levels driven primarily by gravitational forces from the sun and moon. The tide is diurnal in the Barataria Basin. The tidal signal in the Barataria Basin is most pronounced near the Gulf of Mexico and less pronounced farther north. The tidal signal also propagates up the Mississippi River, where the tidal range is often around 1 foot or more at Belle Chasse. A comparison of the mean tidal ranges at the NOAA Grand Isle station (8761724), the Hackberry Bay station near the center of the basin (8761819), and the Laffite station near the GIWW (8761899) demonstrates the decrease in tidal ranges farther into the basin (see Table 4.4.6-1).

Table 4.4.6-1 Typical Water Levels within Barataria Basin

Agency	Station Number	Station Name	Datum	Avg Water Level	Max Water Level	Min Water Level	Mean Tide Range
NOAA	8761724	Grand Isle LA	Local	6.61 ft	--	--	1.04 ft
NOAA	8761819	Texaco Dock, Hackberry Bay	Local	3.44 ft	--	--	0.89 ft
NOAA	8761899	Lafitte, Barataria Waterway	Local	3.21 ft	--	--	0.32 ft
USGS	73802516	Barataria Pass at Grand Isle	NAVD88	0.79 ft	1.47 ft[a]	0.06 ft[a]	--
USGS	7380330	Bayou Perot at Point Legard	NAVD88	1.24 ft	1.54 ft[a]	0.95 ft[a]	--
USGS	2.951E+12	L. Cataouatche at Whiskey Canal	NAVD88	1.17 ft	1.37 ft[a]	0.99 ft[a]	--
CRMS	176	--	NAVD88	0.36 ft	2.94 ft	-2.40 ft	--
CRMS	276	--	NAVD88	0.68 ft	3.39 ft	-0.11 ft	--
CRMS	3617	--	NAVD88	0.55 ft	3.08 ft	-1.25 ft	--
CRMS	181	--	NAVD88	0.31 ft	2.66 ft	-1.57 ft	--
CRMS	3136	--	NAVD88	0.64 ft	2.30 ft	-0.93 ft	--

Source: CPRA 2017

[a] - Average, minimum, and maximum water levels estimated for CRMS and USGS stations for all available data during the period of record. Note that the period of record varies by station, with start years ranging from 2000 to 2012.

NAVD88 - North American Vertical Datum 1988

Currents

Currents within the Barataria Basin are generally characterized by fresh water flowing from Lac des Allemands and the Davis Pond Freshwater Diversion Project south towards the Gulf of Mexico, and saltwater driven northward by tides from the Gulf into Barataria Bay. The Atchafalaya River flow also strongly influences the region via the GIWW, which intersects Lake Salvador and Bayou Perot. The tidal signal in the Gulf generally acts as a wave sweeping counterclockwise (Guillon et al. 2010) and can be observed from data from NOAA stations about 64.4 km (40 miles) apart: the Grand Isle station and the Pilots Station East, Southwest Pass (CO-OPS 2017) station. Figure 4.4.6-1 shows the tidal signal at both stations in June 2017. The high tide reaches Southwest Pass 1 hour to 2 hours before it reaches Grand Isle. This phasing difference combined with the narrow openings between the barrier islands can induce local variations in circulation as the tide propagates through the passes. Throughout the rest of the basin, currents are more complicated and influenced by a variety of local factors. Wind-forced fluctuations in the currents also commonly recur on 3- to 10-day timescales from about October through April in Barataria Basin.

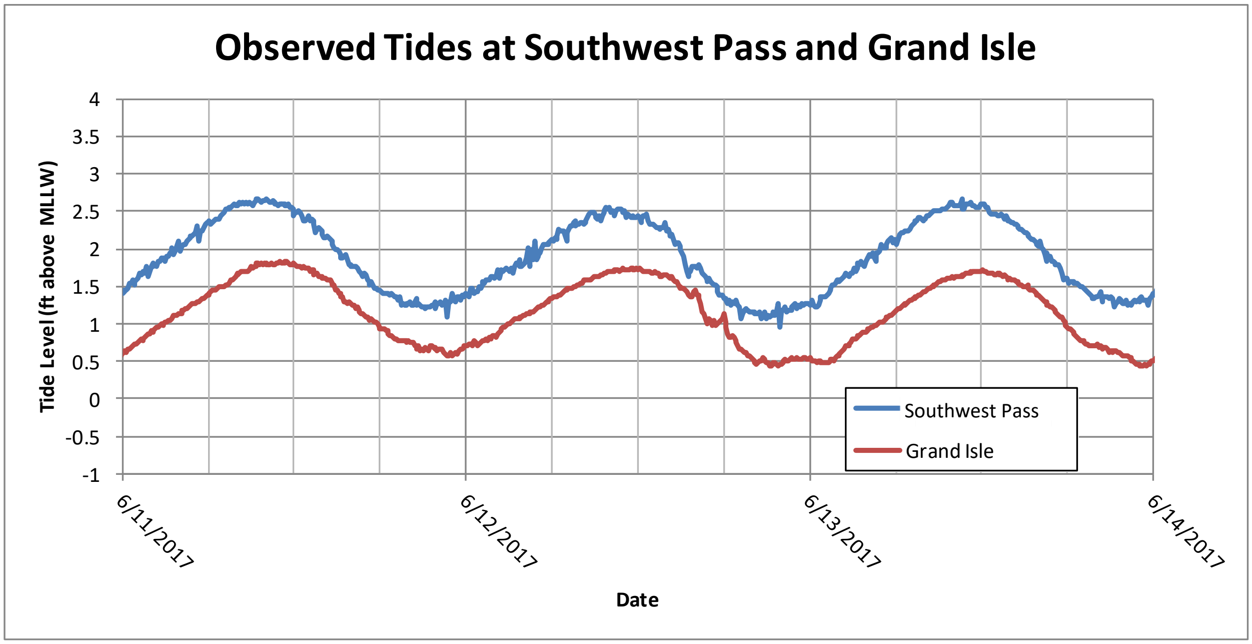


Figure 4.4.6-1. Observed Water Levels at NOAA Stations at Southwest Pass (8760922) and Grand Isle (8761724). (Source: CO-OPS 2017).

Flow

The present-day Barataria Basin receives fresh water mainly through rainfall and the Davis Pond Freshwater Diversion Project (LDWF 2015). Due to the hydrologic modifications in and adjacent to the Mississippi River, most of the Mississippi River fresh water, nutrient, and suspended sediment loads are discharged into the Gulf of Mexico and off the continental shelf in a plume. There is currently very little freshwater influence from the Mississippi River plume to the Barataria Basin, except when river stages are high, winds are blowing from the southwest, and the long shore current cycles the western part of the plume around to the barrier islands (Schiller et al. 2011).

The Mississippi River plume is the largest source of fine sediment and nutrients, as well as freshwater and saltwater mixing in the northern Gulf of Mexico. Satellite data have shown that the size of the plume ranges from 450 square km to 7,700 square km (174 square miles to 2,973 square miles), with the size depending on the magnitude of river discharge (Walker and Rouse 1993). While the nutrient-rich waters of the plume fuel food web and fishery production in the northern Gulf, they also lead to over-eutrophication and hypoxic bottom waters west of the Mississippi River Birdfoot Delta (Rabalais et al. 2007).

In the drier, upper reaches of the basin, rainfall flows as sheetflow (shallow overland flow) to small streams and bayous, then to Lac des Allemands, and eventually to Lake Salvador and Barataria Bay. Storms and associated rainfall and wind events impact circulation within the basin. Increased rainfall at the upper basin can raise local water levels and produce faster-moving streams with greater flows. Increased water levels in the upper basin set up a north–

south flow that pushes fresh water out towards the Gulf. An onshore wind can “pile up” water in the basin, increasing water levels and flooding the marshes. An offshore wind can push water out of the basin, draining the marshes. Local wind effects can produce local cells of circulation based on water level differences and flows induced by wind drag (Reed 1995).

Water flow in the Mississippi River is subject to similar atmospheric factors. The Mississippi River extends over 3,700 km (2,300 miles) and includes more than 20 locks and dams. The southern 1,770 km (1,100 miles) of the river are free flowing, with no locks or dams. Several major tributaries, such as the Ohio and Tennessee rivers, add to the river flow. Farther downstream at the Old River Control Structure in Vidalia, Louisiana, flow from the Red and Mississippi rivers is diverted down the Atchafalaya River. During periods of extremely high flow, water may also be released through the Morganza and Bonnet Carre spillways.

The Davis Pond Freshwater Diversion Project, opened in 2002, operates intermittently to divert up to 10,000 cfs from the Mississippi River into Lake Cataouatche at the head of the Barataria Basin (CPRA 2016). The Project is operated to maintain seasonal average salinities at established gages in the basin. A small portion of Mississippi River water is diverted within the Birdfoot Delta by uncontrolled river diversion projects for marsh creation and restoration, such as the West Bay Sediment Diversion and the Delta Wide Crevasses Project.

During low-flow periods in the Mississippi River, the tidal signal from the Gulf is evident in the river up to New Orleans at RM 102.8 above Head of Passes (AHP) and as far north as the Bonnet Carre Spillway at RM 126.9 AHP. During low-flow periods, the tidal range at the Belle Chasse station at RM 76 AHP is 1.0 foot or more.

The USACE Tarbert Landing gage, immediately downriver from the Old River Control Structure and at RM 306 AHP, has a flow record dating back to 1930. Here, the Mississippi River flows exhibit an annual cyclical pattern, with an average peak flow of nearly 800,000 cfs in April and a minimum of 200,000 cfs in September. The maximum and minimum observed flows are over 1.6 million cfs and 100,000 cfs, respectively (see Figure 4.4.6-2). Local weather patterns, such as high winds, also affect water stages in the Mississippi River.

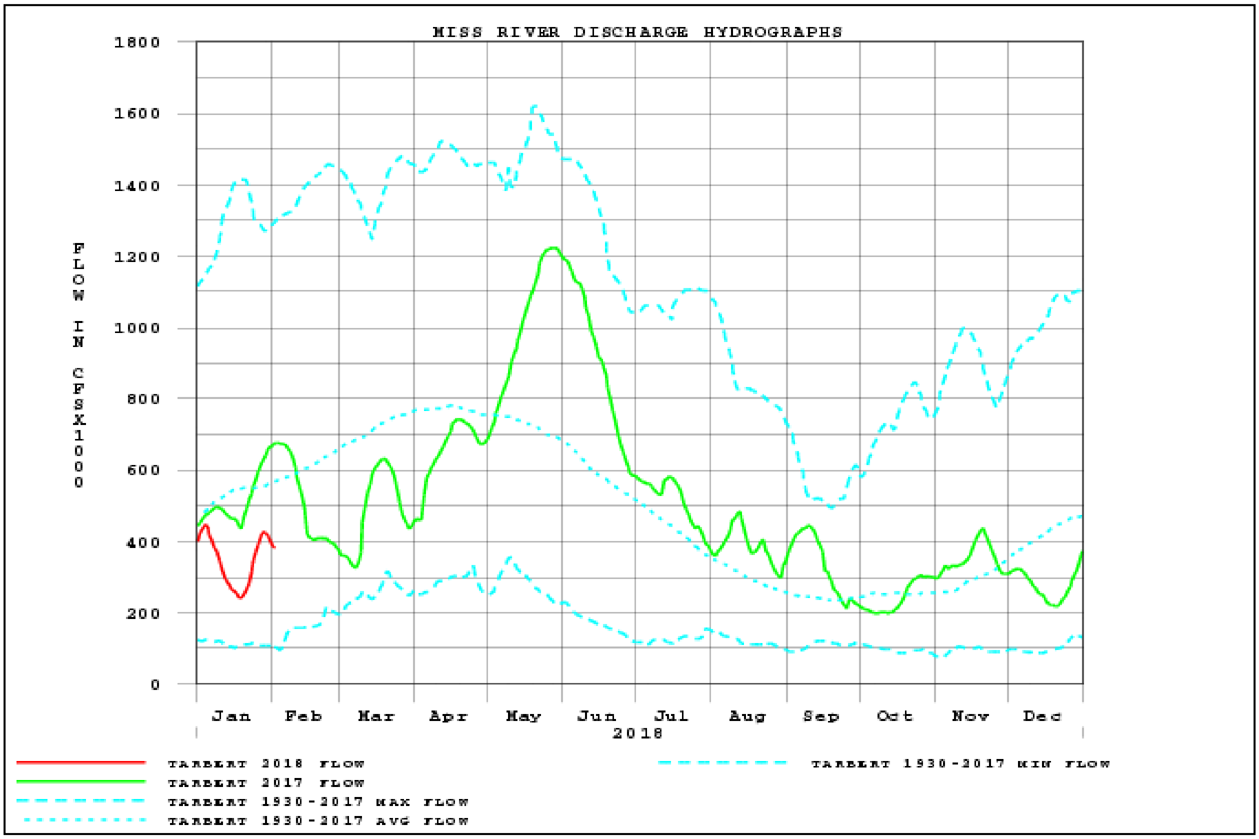


Figure 4.4.6-2. Mississippi River Flow at the Tarbert Landing Gage

The salt water in the Gulf of Mexico is denser than the fresh water flowing in the Mississippi River. During low-flow periods, the Gulf's salt water migrates upstream along the bottom of the river underneath less dense fresh water. This poses risks for municipal water intakes along the lower Mississippi River. As a mitigation measure for deepening the river channel to 13.7 meters (45 feet), during extreme low water conditions, the USACE constructs a temporary sand sill (called a saltwater sill) at RM 65 AHP to block the wedge from migrating upriver. Since deepening the channel to 13.7 meters (45 feet), a sand sill has been constructed 3 times (1988, 1999, and 2012) in order to mitigate for the increased duration and extent of saltwater intrusion above RM 64 AHP (USACE 2018a).

4.4.7 Sediment Transport

Historical Context

The amount of sediment carried down the Mississippi River has decreased significantly in the past 100 years due to a variety of factors including sediment capture at upstream dams, river bank revetments to control erosion, the construction of the levees following the 1927 flood, and soil-conservation programs (Thorne et al. 2008). The river historically carried over 400 million tons of sediment annually, but the annual sediment load has decreased by more than 50% since the early 1900s (Keown et al. 1986, Milliman and Syvitski 1992, Alexander et al. 2012). The present Mississippi-Atchafalaya combined sediment load is approximately 190 million tons per year. Currently, the sediment load is either trapped in the river basin by existing dams, settles out in the navigation channel, or is discharged into the Gulf of Mexico. Navigation channels, such as jetties, are maintained within the Birdfoot Delta to move the remaining river sediment into the Gulf of Mexico. Below the Atchafalaya diversion, the amount of sediment transported by the main channel of the Mississippi River is presently estimated as 124 million tons per year (Horowitz et al. 2001, Horowitz 2006). With the virtual elimination of overbank floodplain deposition, coastal wetlands are not receiving enough sediment to offset erosion and subsidence.

Existing Conditions

The total sediment load of the Mississippi River is composed of finer-grained silt and clay particles (suspended load) higher in the water column, and heavier coarse-grained sand nearer the bottom of the water column (bed load). Figure 4.4.7-1 shows the annual concentration of fine-grained, coarse-grained, and TSS from 1959 to 2005 at the Tarbert Landing gage. Fine-grained sediments are defined as those with grain sizes of 63 microns or smaller, and coarse-grained sediments are those with grain sizes larger than 63 microns (Thorne et al. 2008). Trend lines shown in Figure 4.4.7-1 show that the concentration of total suspended solids has decreased over this time period. A long-term decline in sediment transport in the river during the 19th and late-20th centuries was identified by Thorne et al. (2008), who also suggested some caution in using this estimate given large gaps in the available data, uncertainties associated with early measurements of sediment load, and other factors.

Seasonal variations in flow discharge are evident at the Tarbert Landing gage (see Table 4.4.7-2 for seasonal flow exceedance curves at the landing for 1963 to 2005). The highest flows occur during winter and spring. As noted by Thorne et al. (2008) and shown in Figure 4.4.7-2, as a general rule, discharge is about twice as high during periods of peak flow during the spring than low flow, which typically occurs in the fall. Discharge is, however, highly variable in any season. Flow discharge is expected to influence sediment transport and delivery rates.

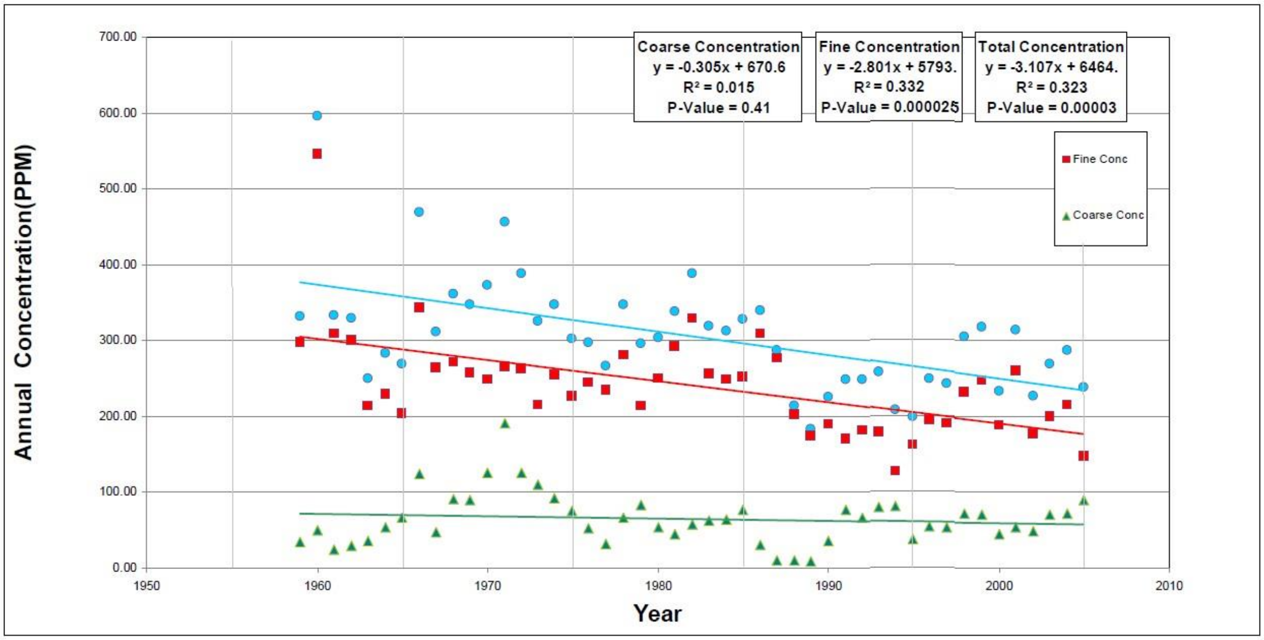


Figure 4.4.7-1. Tarbert Landing Annual Total (blue), Fine (red) and Coarse (green) Sediment Concentrations from 1959 to 2005

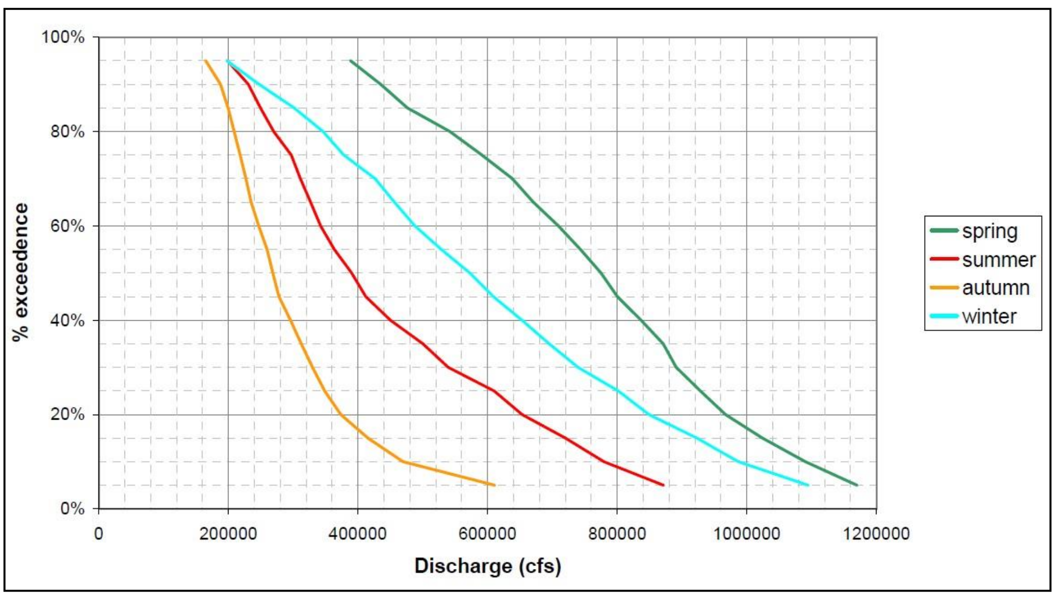


Figure 4.4.7-2. Seasonal Flow Duration Curves at Tarbert Landing from 1963 to 2005 Show the Range in Mississippi River Discharge by Season.

Percent exceedance on the y axis indicates the percent of the time when the discharge was equal to or higher than the discharge on the x axis. For example, the river flow only reached 600,000 cfs or greater 5% of the time from 1963 to 2005 for autumn, and was 160,000 cfs or greater 95% of the time. (Source: Thorne et al. 2008).

Higher river flows suspend and contain more coarse-grained sediments (larger than 63 microns in diameter) that are important in delta building, as they are heavier and settle out faster when water flow slows down or stops. Monthly sediment measurements at Tarbert Landing (Figure 4.4.7-2) show that concentrations of coarse-grained sediments are highest in the winter and spring when flows are highest. Variability is also high in the monthly concentrations of coarse-grained sediments suspended within the river, as shown by the maximum concentration bars compared to the median concentrations (Figure 4.4.7-3).

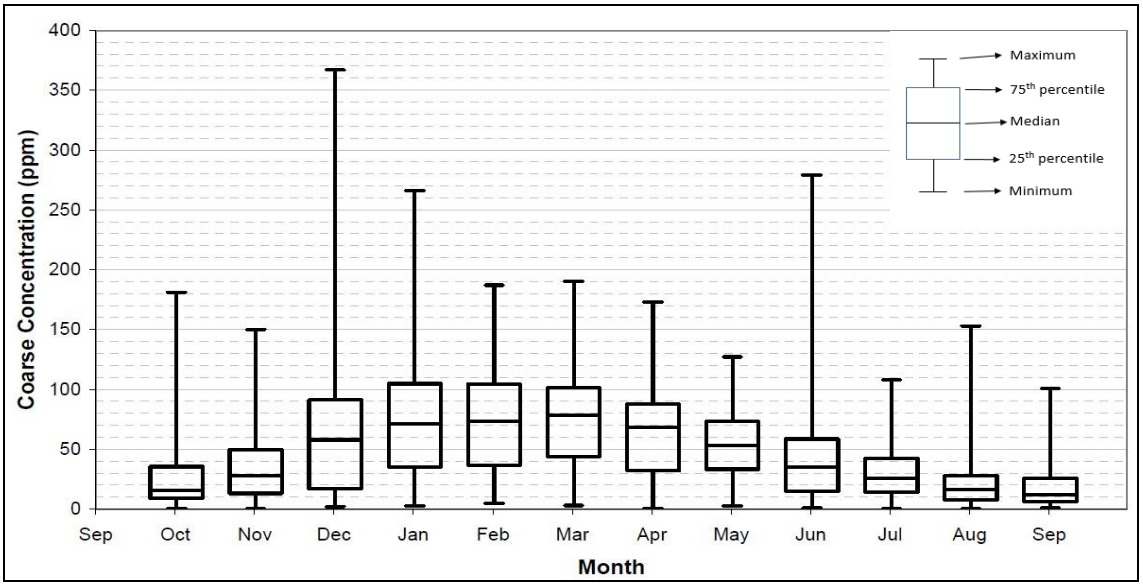


Figure 4.4.7-3. Boxplots Showing Monthly Variation in Concentrations of Coarse-Grained Sediments at Tarbert Landing from 1963 to 2005.

Plots show the minimum (lowest bar around 0), the 25th percentile (low box), the median (middle of box), 75th percentile (high box), and maximum (highest bar) for the month. (Source: Thorne et al. 2008).

In shallow waters of the Barataria Basin, sediment transport is primarily driven by wind and wave effects. Conner and Day (1987) described the sediment transport pattern as "…largely a storm-related phenomena with sediments from other eroding marshes and bay bottoms being deposited." Wind-induced currents re-suspend bottom sediments and transport them around the basin. Waves, either from winds or vessel traffic, erode sediments from shorelines. During storms, the amount of sediment transported within the Barataria Basin is greatly increased (Madden 1988).

4.4.8 Wetland Resources and Waters

The action area is within the Mississippi River Alluvial Plain and Gulf Coast Prairies and Marshes ecoregions in Louisiana, as described above in Section 2.4. These areas are naturally dominated by bottomland hardwood forests, freshwater swamps, and coastal marshes. However, coastal erosion, subsidence, sea level rise, and other factors have resulted in the loss of natural wetlands in coastal Louisiana. To counteract these losses, wetland restoration efforts have been implemented to enhance, restore, and create some of the wetlands in the proposed action area. These include efforts under the Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA) program, funded through Louisiana's 2017 Coastal Master Plan (CPRA 2017), and occurring through CEMVN's program for the beneficial use of dredged material (BUDMAT), which transports material dredged for the maintenance of navigation channels via pipeline to marsh creation cells in the basin. Over the past 25 years, the state of Louisiana has implemented over 30 restoration projects in the Barataria Basin, using state-only funding or in partnership with federal agencies. Since 2007, investments in the restoration of coastal Louisiana and the Barataria Basin have been guided by the state's Coastal Master Plan (CPRA 2017).

Wetland Habitat Functions

Wetlands provide a diverse set of functions and provide ecological, economic, and social benefits. The ability to perform a function is influenced by the characteristics of the wetland and the physical, chemical, and biological processes in it (USACE 2017). Louisiana's coastal wetlands provide habitat for the largest concentration of over-wintering waterfowl in the United States as well as habitat for wildlife, finfish, shellfish, and other aquatic organisms, including threatened or endangered species. Further, they support the largest commercial fishery in the contiguous United States, by volume (NMFS 2017). Wetlands improve water quality by removing organic and inorganic toxic materials, suspended sediments, and nutrients via plant uptake and sedimentation. Primary productivity, decomposition, and other chemical processes also contribute to the removal of certain chemicals from the water (Mitsch and Gosselink 2000). Wetlands also provide a level of flood control; wetland vegetation can attenuate waves and storm surges, and communities sheltered by wetlands may sustain less damage from storm surges (Day et al. 2007). Further, due to their anoxic, wet conditions, wetlands provide a natural environment for sequestration and storage of carbon from the atmosphere. Most wetlands are net carbon sinks when methane emissions and carbon sequestration are balanced (Mitsch et al. 2012).

Wetland Types in the Proposed Action Area

The Barataria Basin comprises a network of interconnecting waterbodies along with natural and artificial levees, coastal habitat, and wetlands (Conner and Day 1987). Salinity is the primary driver of wetland vegetation assemblages in the basin and accounts for the change from freshwater forested wetlands and marshes in the upper basin to saltwater marshes in the lower basin. The salinity gradient in the basin ranges from 0 ppt in the upper basin to 32 ppt in the lower basin (see Section 3.5.2.2 of the EIS for more information about ambient water quality in the proposed action area). Salinities are typically lower in the spring when more rainfall occurs, and higher in the winter due to lower rainfall (Conner and Day 1987). Prior to Mississippi River levee construction, freshwater marshes were more prevalent in the basin (Day et al. 2000, Turner 1997, Connor and Day 1987).

Wetland types within the proposed action area include forested, scrub/shrub, and emergent wetlands, which are further classified by their salinity regimes and tidal influence. Wetlands in the Barataria Basin and Mississippi River delta are typically classified as freshwater, intermediate, brackish, or saline based on salinities and the corresponding plant communities present (Chabreck 1972, CPRA 2017). Wetland types on the west bank of the Mississippi River near the proposed Project diversion structure (RM 60.7 AHP) include mostly freshwater forested and scrub/shrub wetlands, as well as some areas of freshwater emergent wetlands (CPRA 2017). Batture vegetation communities refer to vegetation formed on sediment along the levee; these occur where the Mississippi River meets the crest of the levee, and include seasonally flooded forested wetlands in the immediate vicinity of the proposed Project diversion structure. However, revetments and other areas of impervious substrates limit vegetation growth where they are installed. Farther downstream (near RM 11.0 AHP and Venice, Louisiana), freshwater scrub/shrub and emergent wetlands predominate. Table 4.4.8-1 summarizes the acreage and percentage of the proposed action area covered by each wetland type, based on vegetation data from CPRA's 2017 Coastal Master Plan; these data are also depicted in Figure 4.4.8-1 (CPRA 2017).

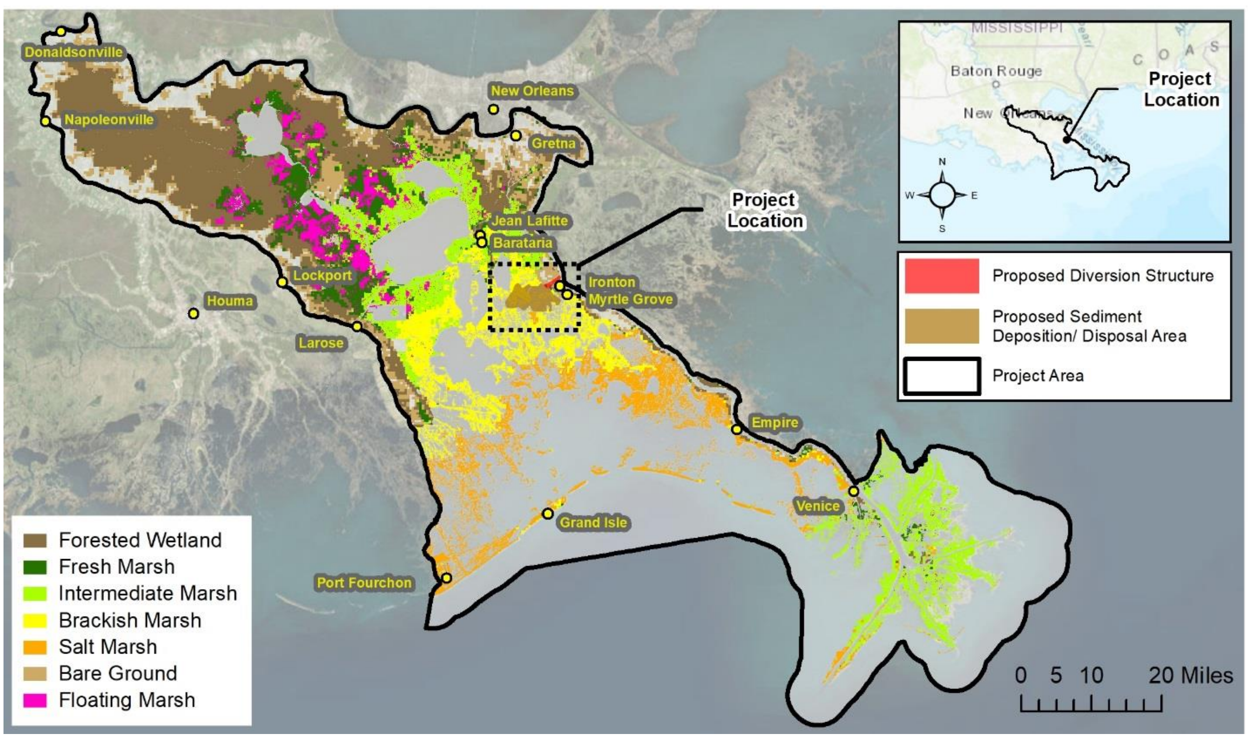


Figure 4.4.8-1. Wetland Types in the Action Area (Source: CPRA 2017)

Table 4.4.8-1 Wetland Habitat Types Occurring with the Action Area

Wetland Type	Total Acres within the Action Area	Percent of the Action Area
Palustrine Wetlands		
Forested Wetlands (including swamp forest)	398,220	18
Freshwater Marsh (including floating marsh)	190,865	8
Estuarine Wetlands		
Intermediate Marsh	216,950	10
Brackish Marsh	144,015	6
Salt Marsh	141,235	6

Source: CPRA 2017

4.5 Historical and Existing Aquatic Resources and Habitat in Barataria Basin

Aquatic resources in the Barataria Basin presented here include the following: aquatic vegetation; benthic resources; fish, shellfish, and fisheries; and invasive species. The aquatic resources in the basin reflect strong salinity, inundation, and corresponding habitat gradients, combined with the influence of factors such as freshwater inputs of sediments and nutrients, wind and wave action, hurricanes, and other climate events (Fitzgerald et al. 2008, Twilley and Rivera-Monroy 2009). Conductance (Section 4.2.1), salinity (Section 4.2.2), temperature (Section 4.2.3), dissolved oxygen (Section 4.2.4), turbidity (Section 4.2.5), and wetland vegetation (Section 4.4.8) are presented in earlier sections of this document, but are referenced here as appropriate.

The Mississippi River Delta, including the Barataria Basin, was formed from river sediments deposited during seasonal pulses of fresh water from the Mississippi River; coarse depositions formed natural levees along the river course, and finer sediments accumulated landward of the levees, into the basin (Twilley and Rivera-Monroy 2009). As the delta grew, emergent marsh vegetation became established, which slowed water velocities and increased sediment deposition, resulting in the formation of expansive marsh systems that further stabilized the delta and provided habitat for a diversity of flora and fauna.

Construction of flood control projects (for example, levees and channels) in the early and mid-1900s disrupted the hydrologic connection between the Mississippi River and its adjacent wetlands, reducing or eliminating freshwater and sediment inputs to the delta (Conner and Day 1987, Day et al. 2000, Turner 1997). Historical alterations in salinity, sediments, nutrients, wave energy, and other environmental factors are reflected in the productivity, trophic level interactions, nutrient cycling, vertebrate food chains, and subsequent changes in assemblages of flora and fauna in the Barataria Basin. Further loss of benthic resources and coastal fish and shellfish populations is anticipated with additional loss of habitats that are critical to their growth and survival (Browder et al. 1989, Chesney et al. 2000, Beck et al. 2001).

The effects of the DWH oil spill and subsequent remediation efforts in the Barataria Basin are important in the context of describing historical conditions of the system. Oiling exposure in Louisiana from the spill was extensive, with over 1,100 linear kilometers of marsh shoreline oiling state-wide. Marsh oiling in Louisiana represented about 95% of the total marsh oiling Gulf-wide (DWH NRDA Trustees 2016a, Nixon et al. 2015). Within Louisiana, the majority of the heaviest oiling occurred in Barataria Bay (see Figure 4.5.1-1). Impacts of oiling on sediment, soil, benthic infauna, oysters, shrimps, crabs, and benthic feeding fishes in Barataria Bay were also documented.

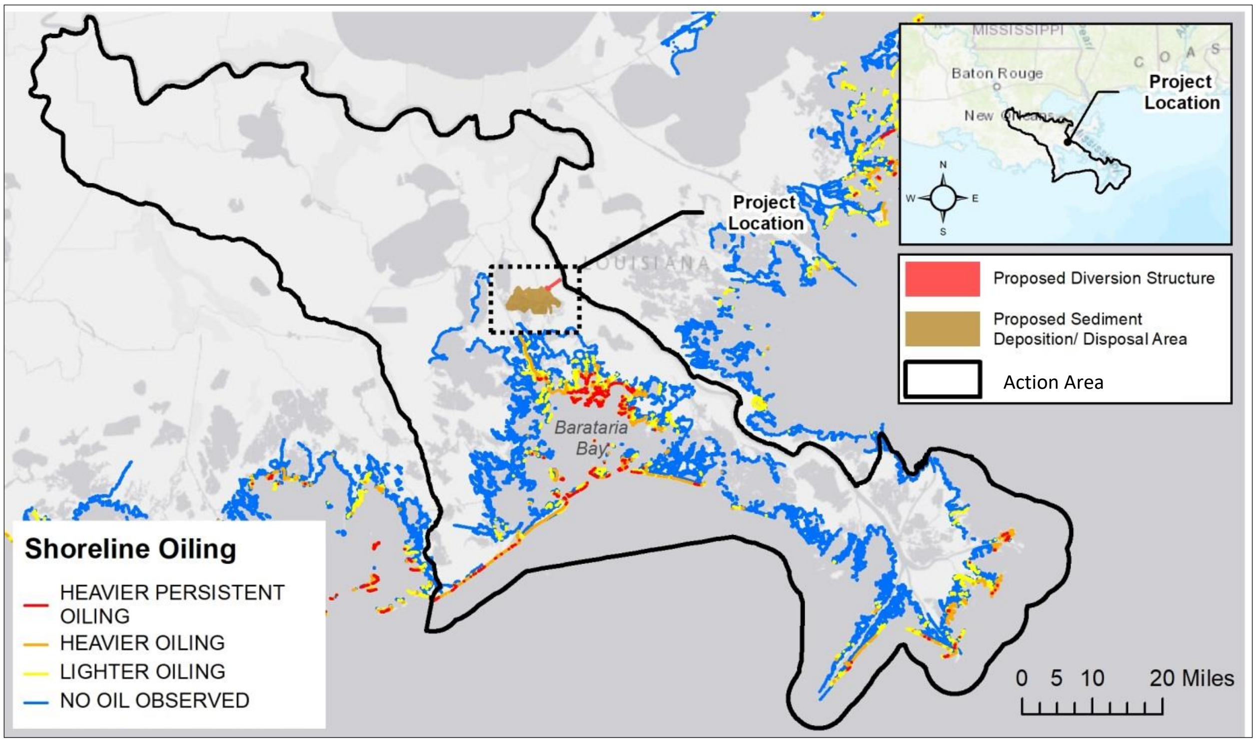


Figure 4.5.1-1. Observed Shoreline Oiling in and around the Action Area (Source: Nixon et al. 2015)

Submerged Aquatic Vegetation (SAV)

The distribution of aquatic vegetation in the Barataria Basin, like wetlands vegetation, reflects salinity and inundation gradients, but is also influenced by sediment deposition, nutrient and light availability, erosion, subsidence, sea level rise, and storm surge (Paola et al. 2011, Alexander et al. 2012). SAV, as well as vegetation of barrier islands, are described in this section, while wetland vegetation is described in Section 4.4.8.

The Barataria Basin's habitats exhibit a salinity gradient, ranging from freshwater swamps in the uppermost basin, followed by intermediate habitats, brackish habitats, and then extensive salt marshes at the coast, with estuarine and marine SAV becoming more prevalent in the open water. SAV supports a diverse epiphytic biota, exports organic matter and nutrients into the water column, oxygenates the water column, and stabilizes bottom sediments by reducing current velocity and wave energy. In turn, these processes affect species composition, biomass, and distribution of the SAV as well as the fauna that rely on SAV for habitat (Koch 2001).

SAV species distributions and biomass in the northern Gulf of Mexico are influenced by salinity, water depth, turbidity, as well as other variables. Hillmann et al. (2016) documented 14 SAV species in the coastal areas, 4 of which—coontail (*Ceratophyllum demersum*), Eurasian water milfoil (*Myriophyllum spicatum*), widgeon grass (*Ruppia spp*.), and hydrilla (*Hydrilla verticillata*)—accounted for 73 percent of the above-ground biomass collected. Coontail, widgeon grass, and lesser pondweed (*Potamageton pusillus*) were collected across freshwater, intermediate, brackish, and saline zones. Hydrilla was collected only in freshwater habitat; common water nymph (*Najas guadalupensis*) and wild celery (*Vallisneria americana*) in all but fully saline habitat; and Eurasian water milfoil, in all but freshwater habitat. Turtle grass (*Thalassia testudinum*), shoal grass (*Halodule wrightii*), and manatee grass (*Syringodium filiforme*) are the primary seagrass species with star grass (*Halophila engelmannii*) also occurring in some areas. Other relationships among SAV and environmental variables found by Hillmann et al. (2016) included the following:

- SAV species distribution corresponded significantly to environmental variables (salinity, water depth, and turbidity).
- Vegetation biomass was significantly lower in the saline zone when compared with other zones, when all samples were combined (including those without SAV).

The factors controlling SAV distribution across salinity regimes in the northern Gulf Coast are not well documented; this makes predictions of resource availability difficult (Hillmann et al. 2016). Consequently, SAV coverage is predicted as a group rather than by species (Visser et al. 2013, 2017). Changes in salinity, water depth, and light transmission can result in changes in biomass, productivity, species composition, and distribution of SAV (Hillmann et al. 2017). SAV declines in the middle and upper Barataria Basin have been attributed to saltwater intrusion associated with hurricanes and flood control activities. SAV increased in the upper and middle basin coincident with the Davis Pond Freshwater Diversion Project (operational in 2002), but declined following salinity increases and scouring associated with Hurricanes Gustav in 2002 and Ike in 2008.

SAV has been described as "the most significant form of complex cover for aquatic animals in the Barataria Basin" (LDWF 2015a). Diverse SAV communities are often scattered throughout the marshes and provide important food and cover to a wide variety of fish and wildlife species, including juvenile and overwintering shrimp and crabs; coastal fishes such as drum, croaker, seatrout, and flounder; and habitat and foraging areas for invertebrates and fish (Hillmann et al. 2017, LDWF 2005, Fonseca and Bell 1998). SAV in intermediate and brackish areas provides nursery grounds and shelter for many species of fish and shellfish (Rozas and Odum 1988, LWDF 2005). Rozas et al. (2012) found that the density and biomass of the most abundant faunal taxa were higher within seagrass areas than within Spartina marsh.

Benthic Resources

Coastal regions are among the most productive ecosystems in the world, and links between benthic and open water environments are significant in the transfer of energy between these habitats (Valiela 1995, Marcus and Boero 1998). For example, marsh epifauna, such as periwinkles, graze on algae and fungi that grow on the stems of marsh vegetation and soils, support organic matter production and nutrient cycling within the marshes. They turn provide prey for salt-marsh species like blue and mud crabs, turtles, large fishes, and wading birds (Montague et al. 1981, Kemp et al. 1990, Sillman and Bertness 2002).

Benthic resources of the Barataria Basin described in this section include benthic algae, infauna (organisms that live in the sediment), and epifauna (organisms that live on top of the sediment). These benthic producer species and lower trophic level consumer species can also live on the shoots of marsh grasses and SAV, as well as the oyster reefs. Benthic macroinvertebrates such as grass shrimp, penaeid shrimp, and crabs are often referred to as benthic resources. The penaeid shrimps (brown shrimp, white shrimp) and blue crab are addressed in detail in the EIS and EFH Reports because they support valuable commercial fisheries and are key ecological species for coastal Louisiana. Likewise, oysters are sessile bivalves often addressed under benthic resources in assessment reports and environmental impact statements (for example, DWH NRDA Trustees 2016a). Eastern oysters are presented in the EIS and EFH Reports because they also support a valuable commercial fishery and important ecological functions in Louisiana estuaries.

Within the Barataria Basin, these lower trophic level benthic groups include benthic algae (chlorophytes, cyanophytes, and diatoms), infauna (amphipods, polychaetes, nematodes, and oligochaetes), and epifauna (small clams, snails, and marsh periwinkles). Changes in the distribution and composition of benthic resources have been linked to shifts in food web structure, increases in invasive species, and declines in the abundance of historical fish populations in other major U.S. estuaries (Kimmerer 2002, Kimmerer 2004, Dynamic Solutions 2012, Tango and Batiuk 2013, Kimmerer and Thompson 2014, Adamack et al. 2017). However, no benthic monitoring program exists for Barataria Basin and Birdfoot Delta, and there are not many available ecological field studies evaluating how habitat and environmental conditions affect benthic resources in coastal Louisiana.

Growth of benthic algae depends on temperature, light, and nutrients. Like most aquatic organisms, benthic taxa have lower and upper threshold values for these conditions, outside of which they cannot grow. Cold temperatures generally reduce growth of benthic algae, infauna, and epifauna. Increased turbidity reduces light availability and generally reduces algal growth. Benthic taxa exhibit increasing growth with increasing temperature, light availability, and nutrient concentrations to some optimum growth based on these conditions (Thomann and Mueller 1987, Thornten and Lessem 1978); however, growth can become limited or even reduced if these functions get too high. For example, increased nutrient availability generally increases growth, but excessive nutrient concentrations can cause algal blooms, which can reduce light and DO levels for the benthic lower trophic level groups.

The DWH oil spill severely impacted benthic species, including amphipods, fiddler crabs, and marsh periwinkles along oiled marsh shorelines, including the Barataria Basin (DWH NRDA Trustees 2016a). The "heavier" and "heavier persistently" oiled marsh sites in Louisiana (see Figure 3.10-1) were expected to reduce survival of amphipods by 36 to 95 percent in 2010 (Powers and Scyphers 2015). Densities of periwinkles were reduced by 80 to 90 percent at the oiled marsh shoreline edge and by 50 percent in the oiled marsh interior due to oiling and cleanup actions (Zengel et al. 2015). An estimated 204 metric tons of periwinkles were lost in the 38.5 miles of heavy persistently oiled marsh edge shorelines in Louisiana (Powers and Scyphers 2015). Recovery of the periwinkles was expected to take three to five years if wetland vegetation recovered enough to support the animals, but normal-sized ranges of the snails are not expected to recover until at least 2021 (Powers and Scyphers 2015). Reductions in the benthic resources along the oiled marsh shoreline and interior habitats resulting from the oil spill could affect the prey availability and distribution of shrimp, crab, and fish that depend on the benthic resources for growth and recruitment in the Barataria Basin.

4.6 ESA Listed Species Occurrence in the Action Area

Potential interactions with ESA listed species are a function of the timing and life stages present within the action area. Use of Mississippi River, Barataria Basin, and Birdfoot Delta by ESA listed species is seasonal for some species while other species may be present year-round (see Table 4.6-1)

Table 4.6.1 Potential Presence and Habitat Use of Action Area by ESA Species

ESA Species	Life Stage*	Habitat	Presence in Proposed Action Area											
			Winter		Spring			Summer			Fall			Winter
			Jan	Feb	March	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Pallid Sturgeon	larvae	R: water column	x	x	x	x	x	x	x	x	x	x	x	x
Pallid Sturgeon	juveniles	R: water column	x	x	x	x	x	x	x	x	x	x	x	x
Pallid Sturgeon	adults	R: water column	x	x	x	x	x	x	x	x	x	x	x	x
Black Rail	eggs	U: marsh	--	--	--	--	x	x	x	--	--	--	--	--
Black Rail	fledglings	U: marsh	--	--	--	--	--	x	x	x	x	x	--	--
Black Rail	juveniles	U: marsh	x	x	x	x	x	x	x	x	x	x	x	x
Black Rail	adults	U: marsh	x	x	x	x	x	x	x	x	x	x	x	x
Piping Plover	juveniles	U: marsh, shorelines	x	x	x	x	--	--	x	x	x	x	x	x
Piping Plover	adults	U: marsh, shorelines	x	x	x	x	--	--	x	x	x	x	x	x
Red Knot	juveniles	U: marsh, shorelines	x	x	x	x	x	x	x	x	x	x	x	x
Red Knot	adults	U: marsh, shorelines	x	x	x	x	x	--	--	x	x	x	x	x
West Indian Manatee	juveniles	B: seagrass	--	--	--	--	x	x	x	x	x	--	--	--
West Indian Manatee	adults	B: seagrass	--	--	--	--	x	x	x	x	x	--	--	--
Green Sea Turtle	juveniles	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Green Sea Turtle	adults	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Hawksbill Sea Turtle	juveniles	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Hawksbill Sea Turtle	adults	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Kemp's Ridley Sea Turtle	juveniles	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Kemp's Ridley Sea Turtle	adults	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Leatherback Sea Turtle	juveniles	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Leatherback Sea Turtle	adults	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Loggerhead Sea Turtle	eggs	U: shorelines	--	--	--	--	--	--	x	x	x	x	--	--
Loggerhead Sea Turtle	hatchlings	B: water column	--	--	--	--	--	--	x	x	x	x	--	--

ESA Species	Life Stage*	Habitat	Presence in Proposed Action Area											
			Winter		Spring			Summer			Fall			Winter
			Jan	Feb	March	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Loggerhead Sea Turtle	juveniles	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Loggerhead Sea Turtle	adults	B: seagrass, water column	x	x	x	x	x	x	x	x	x	x	x	x
Loggerhead Sea Turtle	nesting adults	U: shorelines	--	--	--	--	x	x	x	x	--	--	--	--

*Only showing life stages reported to occur within the action area
R = Mississippi River; U = Upland; B = Barataria Basin and Birdfoot Delta
x = potentially present; -- = not present

4.6.1 Pallid Sturgeon Use of Action Area

The current distribution of pallid sturgeon is reduced and fragmented relative to its historical range. The Coastal Plains Management Unit of pallid sturgeon contains spawning populations in the Mississippi River from the Missouri River confluence downstream to the Gulf of Mexico (USFWS 2014). Pallid sturgeon are documented as occurring in the lower Mississippi River adjacent to the Barataria Basin (LDWF 2014). No spawning sites have been documented, but spawning habitat use by this species is poorly understood and sampling efforts in this specific area to date are too limited to draw conclusions. To date only 2 young-of-year pallid sturgeon have been collected between RM 85 and 33 (USACE 2017). The low numbers observed south of RM 85 may be due to low abundance, but they could also reflect the limited sampling effort in this area to date (J. Kilgore, USACE Research Fisheries Biologist personal communication, 2018). Critical habitat has not been designated for the pallid sturgeon.

4.6.2 Eastern Black Rail Use of Action Area

Eastern black rail is an elusive and cryptic species, making accurate assessment of its range and habits difficult. Between 2010 and 2017 only a small number of observations were recorded in Louisiana (USFWS 2018). Black rails are known to winter in the marshes of Cameron and Vermilion parishes. Anecdotal black rail observations in the vicinity of Grand Isle have been recorded in eBird.com, a collaboration by the Cornell University Lab of Ornithology and Audubon Society. To date there have been 33 record observations of black rails in Louisiana, the majority from an Audubon survey focused on Cameron and Vermilion parishes (eBird 2019, E. Johnson pers com., 2019). Black rail occurrence in the action area cannot be discounted based on the cryptic nature of this species, documented observations in the vicinity, and the presence of suitable habitats throughout the Barataria Basin.

4.6.3 Piping Plover Use of Action Area

Piping plovers spend winters in coastal Louisiana and may be present for 8 to 10 months annually. Piping plovers arrive as early as late July and remain until late March or April. Piping plovers forage on intertidal beaches, mudflats, sand flats, algal flats, and wash-over passes with little or no emergent vegetation. Roosting sites may include areas with debris, detritus or topographic relief offering plovers protection from high winds and cold weather. Piping plovers occur infrequently during migration within mudflats and estuarine habitat in the Barataria Basin. The eBird database described in 5.6.2 contains 2,247 piping plover observation records from Louisiana, nearly all of which are from coastal barrier islands (eBird 2019). Wintering piping plovers have been documented on the barrier islands of the lower Barataria Basin including Grand Isle and Elmers Island, and the barrier islands adjacent to the South Pass entrance to the Mississippi River (Elliot-Smith et al. 2015).

Critical habitat is designated for wintering piping plover in the coastal shoreline and barrier islands extending from the western edge of the action area east to the Grande Terre Islands, and selected barrier islands in the Birdfoot Delta at the mouth of the Mississippi River.

4.6.4 Red Knot Use of Action Area

Outside of the breeding season, the red knot is found primarily in intertidal, marine habitats, especially near coastal inlets, estuaries, and bays (Baker et al. 2013); within the proposed action area, this habitat may be present along beaches and barrier islands along the Gulf of Mexico (NatureServe 2017). Red knots are found in Louisiana during spring and fall migrations and the winter months (August through May). Migrating and overwintering red knots commonly forage on bivalves, gastropods, and crustaceans on beaches, oyster reefs and exposed bay bottoms. They aggregate in large numbers, roosting on high sand flats, reefs and other sites protected from high tide. The species is considered rare to uncommon along the Louisiana coast and barrier islands, although it has been a regular visitor to Grande Isle (Fontenot and DeMay 2014). The eBird database contains 31 observational records of red knots in Plaquemines Parish, concentrated along barrier islands adjacent to the Gulf of Mexico (eBird 2019). Those individual observations typically represent numerous birds. For example, the Barataria-Terrebonne National Estuary Program conducted a single-day survey of the Grand Isle Caminada Headlands in May of 2015. They recorded red knot observations at 3 locations totaling nearly 600 individuals (DeMay et al. 2015). Based on the presence of known wintering and migratory staging habitat and documented species occurrence, this species is likely to occur in the action area during fall, winter, and spring months.

4.6.5 West Indian Manatee Use of Action Area

The limited data on West Indian manatees suggest that this species could be present within the proposed action area, but only as a transient visitor (particularly during the warmer months), and not as a resident species. There were about 121 reported sightings of the West Indian manatee in Louisiana waters between 1990 and 2005. The most likely origins of manatees occurring along the northern Gulf Coast are the wintering populations from southwest Florida or Mexico (Fertl et al. 2005). Manatee are most likely to be present during summer and fall and could occur in any portion of the action area except for the Mississippi River mainstem.

4.6.6 Green Sea Turtle Use of Action Area

Upland Areas

There are no records indicating nesting of green sea turtles on Louisiana beaches (LDWF 2004). Since green sea turtles are not known to nest within the proposed action area, they are therefore unlikely to use upland areas. The closest documented nesting beaches for green sea turtles are more than 500 kilometers to the east in western Florida (Valverde and Holtzwart 2017). Locations of the NWA DPS green sea turtle's nesting areas are shown in Figure 4.6.6-1.

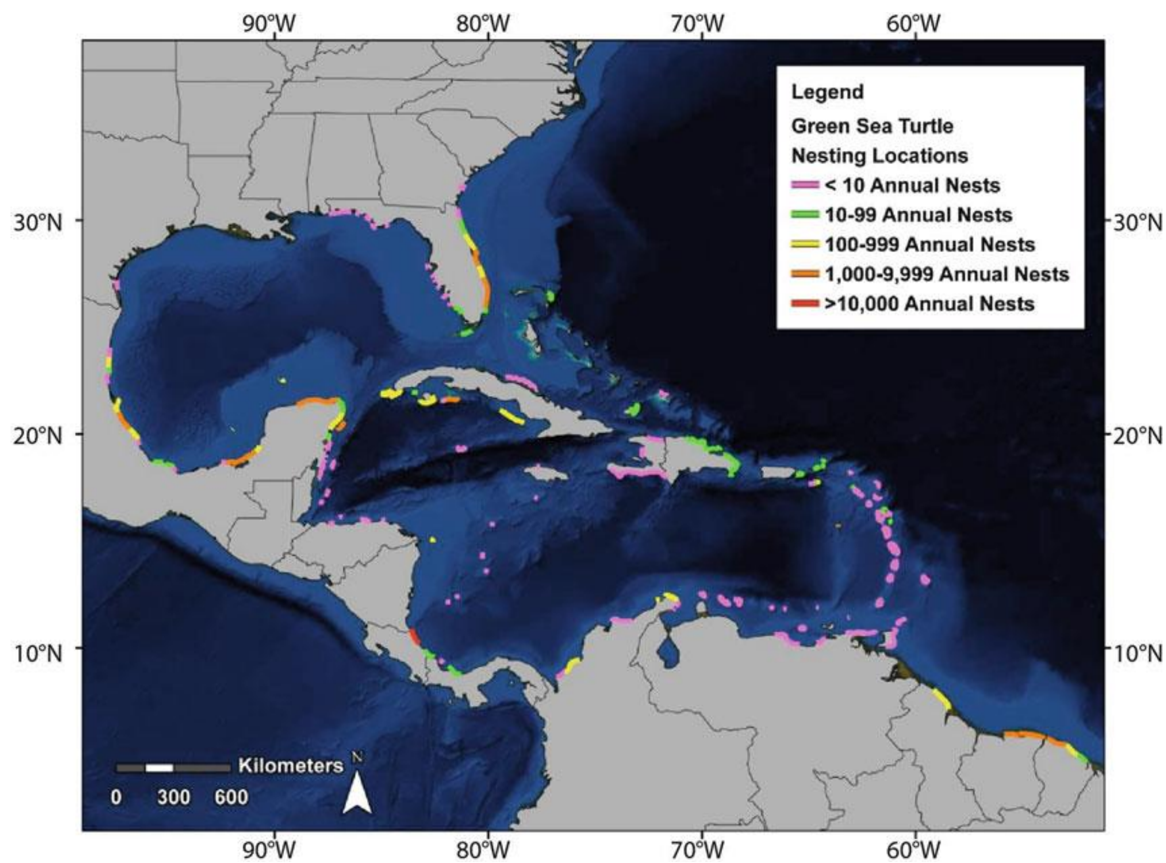


Figure 4.6.6-1. Generalized Nesting Locations of the Green Sea Turtle in the Gulf of Mexico, Caribbean, and Northwest Atlantic Ocean

(interpreted from Dow et al. 2007 and SWOT 2010b) (Source: Valverde and Holzwart 2017)

Aquatic Areas

Juvenile or adult green sea turtles may occur in marine portions of the proposed action area while migrating, resting, or foraging. Green sea turtles are typically found in warm bays and oceans, often associated with seagrass beds or macroalgae resources in or near estuaries. Green sea turtles were once harvested commercially from seagrass beds around the Chandeleur Islands of southeastern Louisiana. Green sea turtles migrate from foraging areas to natal nesting beaches and may travel hundreds or thousands of kilometers each way (Hays et al. 2001). Available information on green turtle migratory behavior indicates that long-distance dispersal is seen only in juvenile turtles, suggesting that larger adult-sized turtles return to forage and stay within the region of their natal nesting beaches (Monzón-Argüello et al. 2010).

Observations of green sea turtles in or near Barataria Basin are taken from fishermen surveys, stranding records and research studies. Fishermen have reported green turtle observations just south of the barrier islands (Fuller et al. 1987). Stranding observations between 1998 and 2019 show that 3 stranded green turtles were detected in the inshore component of National Marine Fisheries Service Statistical Area 13, which includes the action area (NOAA 2019). Researchers have also reported green turtles feeding on macroalgae along jetties and rocky substrates near the barrier island passes along the southern edge of Barataria Basin (K. Hart USGS research scientist, personal communication 2019). In addition, a juvenile green sea turtle whose location was tracked for 109 days was recorded in Barataria Bay and near the barrier islands of Louisiana in 2013 (Coleman 2017). These observations suggest that green sea turtles are likely present within the action area. Based on the low numbers of observed individuals in the action area, their abundance in this area is predicted to be low and will likely be restricted to the lower basin close to the barrier islands, where SAV provides forage for the turtles.

4.6.7 Hawksbill Sea Turtle Use of Action Area

Upland Areas

There are no records indicating nesting of hawksbill sea turtles on Louisiana beaches. As hawksbill turtles are not known to nest within the proposed action area, they are unlikely to utilize upland areas there. The closest known nesting beaches for hawksbill sea turtles are found over 500 kilometers to the east along Florida's west coast (Valverde and Holzwart 2017).

Locations of hawksbill sea turtle nesting areas are shown in Figure 4.6.7-1.

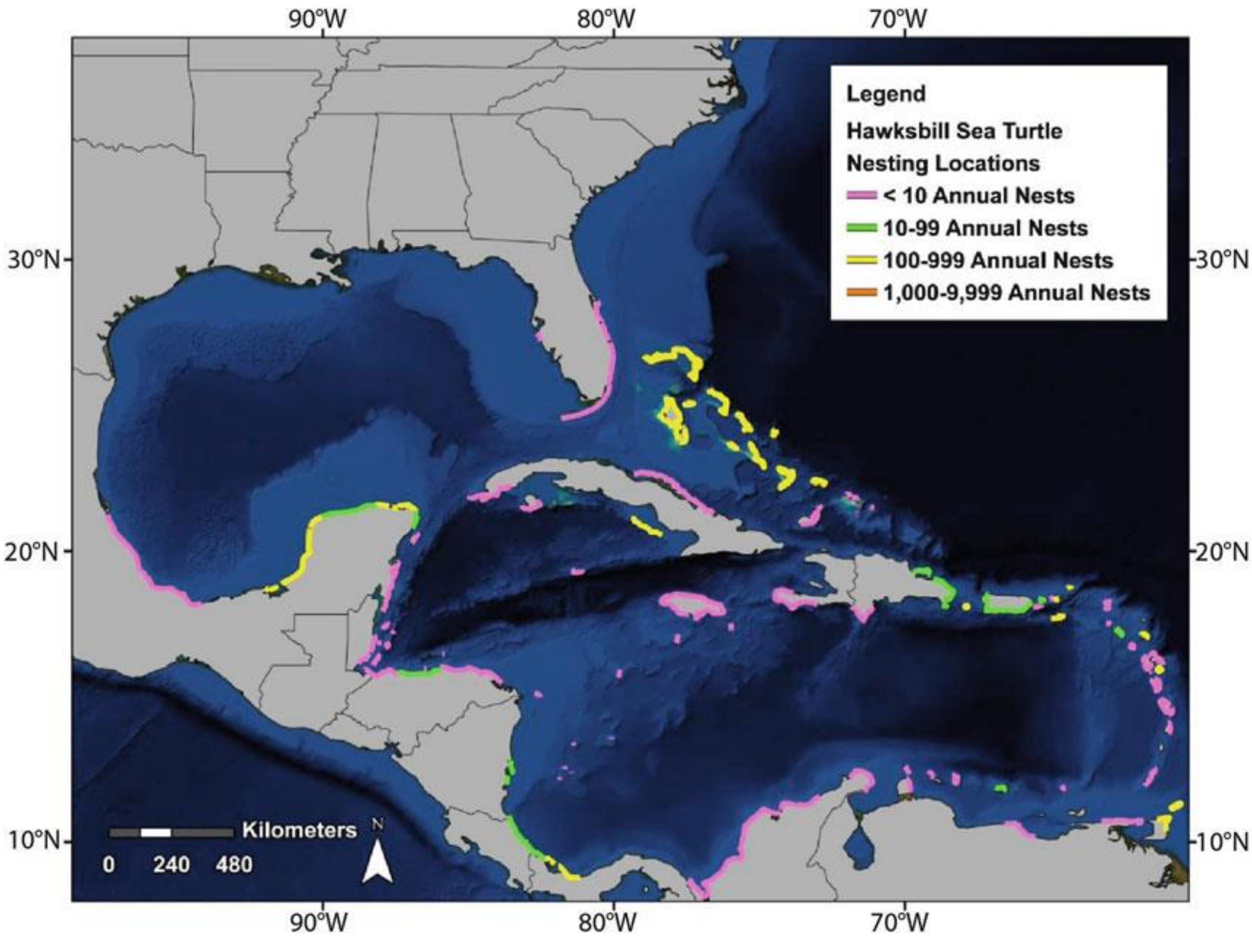


Figure 4.6.7-1. Generalized Nesting Beach Locations of the Hawksbill Sea Turtle in the Gulf of Mexico, Caribbean, and Northwest Atlantic Ocean
(interpreted from Dow et al. 2007 and SWOT 2008) (Source: Valverde and Holzwart 2017)

Aquatic Areas

Juvenile or adult hawksbill sea turtles may potentially occur in marine portions of the proposed action area while migrating, resting, or foraging. Hawksbill sea turtle hatchlings swim offshore immediately after hatching to mature among floating algal mats and drift lines before returning to coastal foraging grounds as subadults. Adult hawksbill turtles migrate from foraging areas to natal nesting beaches and may travel long distances each way (NOAA 2017b, USFWS 2018).

There are no reports of hawksbill sea turtles in or near Barataria Basin in stranding reports (NOAA 2019), field surveys (Fuller et al. 1987) or from field researchers. These observations suggest that hawksbill sea turtles are very unlikely to be present in the action area. While their occurrence cannot be entirely ruled out, any occurrence of hawksbill sea turtles in the action area would be considered rare and incidental. For this analysis their presence in the action area is considered unlikely.

4.6.8 Kemp's Ridley Sea Turtle Use of Action Area

Upland Areas

Nesting by Kemp's ridley sea turtles is concentrated on the beaches of the western Gulf of Mexico, primarily in Tamaulipas and Veracruz, Mexico with a few historical records in Campeche, Mexico. Nesting also occurs regularly in Texas and infrequently in Alabama, Florida, Georgia, the Carolinas, and Virginia (USFWS 2017). There are no records of Kemp's ridley sea turtles nesting within the proposed action area. The closest recorded nesting beaches are found over 200 kilometers away in Alabama and Texas. As Kemp's ridley sea turtles are not known to nest within the proposed action area they are unlikely to utilize upland areas. Locations of Kemp's ridley sea turtle nesting areas are shown in Figure 4.6.8-1.

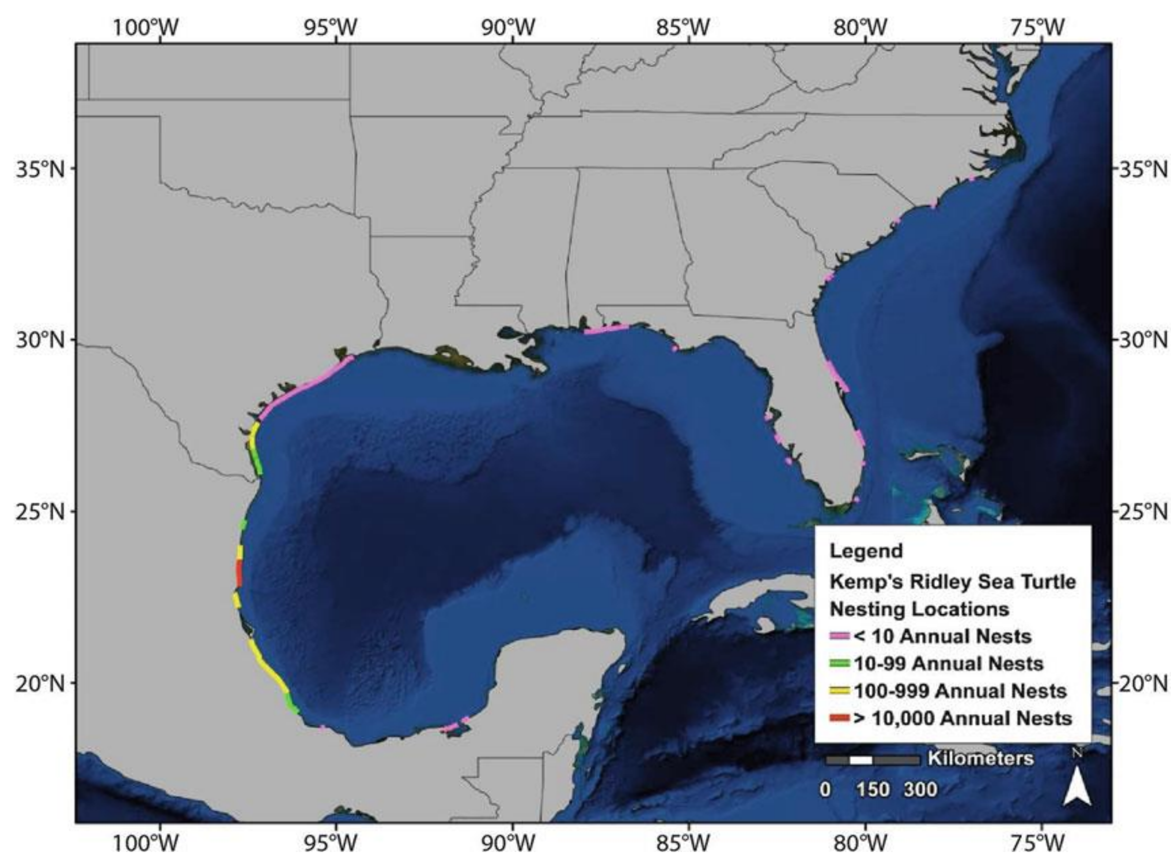


Figure 4.6.8-1. Generalized Nesting Beach Locations of the Kemp's Ridley Sea Turtle in the Gulf of Mexico and Southeast U.S. Atlantic Coast (interpreted from Dow et al. 2007, SWOT 2010a, NMFS et al. 2011) (Source: Valverde and Holzwart 2017)

Aquatic Areas

Juvenile or adult Kemp's ridley sea turtles may occur in marine aquatic areas of the proposed action area while migrating, resting, or foraging. The primary range of Kemp's ridley sea turtles is within the Gulf of Mexico basin, though they also occur in coastal and offshore waters of the U.S. Atlantic Ocean (USFWS 2017). This species often returns to the same foraging areas each year. For example, Shaver et al. (2013) identified several foraging hotspots off the coast of Louisiana, including south of the marine component of the action area seaward of Barataria Basin, outside of the barrier islands. Of 24 satellite-tagged Kemp's ridley turtles monitored over a 13-year period, Shaver et al. (2013) found that over 115 turtle days were spent foraging outside of the barrier islands of Barataria Basin (in "hotspots") and 59 turtle days to 114 turtle days were spent foraging in locations to the east and west of the Birdfoot Delta (in "warm spots"). A portion of the foraging warm spots overlap with the southern edge of the action area, within low-impact areas from the Project. These areas are consistent with Kemp's ridley observations described below.

Kemp's ridley sea turtles are the most common sea turtle in stranding records within and near Barataria Basin, representing at least 47 of the 55 sea turtles observed between 1998 and 2019 (NOAA 2019). Kemp's ridleys are known to concentrate in shallow coastal waters, bays, estuaries and sounds of the Gulf of Mexico (Valverde and Holzwart 2017). These habitats comprise much of the action area. Coleman et al. (2016) monitored movements of juvenile Kemp's ridley sea turtles and identifies the lower portion of Barataria Basin including most of Barataria Bay as "core use habitats" for the species. While Mississippi Sound on the east side of the Mississippi River delta appears to be the primary wintering and use areas for juvenile Kemp's ridley sea turtles, they were observed to move into Barataria Basin in the spring (March-May) and appear to forage in the Mississippi River delta and offshore of the barrier islands throughout the year (Coleman et al. 2016). Movements of Kemp's ridley sea turtles between hatching and when they recruit to nearshore waters as subadults are largely unknown due to limited monitoring of open ocean habitats and the cryptic nature of small sea turtles. Large-scale ocean circulation models are one potential solution for modeling the predicted distribution of these individuals. Such modeling efforts suggest that there could be large differences in annual recruitment to nearshore areas like Barataria Basin driven primarily by oceanic patterns (Putnam et al. 2013). Telemetry studies suggest that sub-adult and adult Kemp's ridleys move into coastal areas of Louisiana in the spring of each year, between April and May, and move to wintering habitat in deeper or more southern waters each fall between September and November (Shaver et al. 2013, Valverde and Holtwart 2017). This spring to fall period is consistent with stranding records which indicate that Kemp's ridleys are found most frequently encountered between April and September of each year (NOAA 2019). Turtles smaller than 24 kg may spend longer periods in the nearshore areas.

Kemp's ridleys are often associated with channels and passes where eddies may concentrate important prey such as blue crab (Valverde and Holtwart 2017). Abundances of Kemp's ridley sea turtles appear to match blue crab population abundance and/or size (Valverde and Holtwart 2017). Tracking studies indicate that juvenile and adult Kemp's ridley sea turtles that use Barataria Basin are also using habitats south of the barrier islands and along the coastal shelf (USGS 2019) for foraging, rearing, and migrating.

4.6.9 Leatherback Sea Turtle Use of Action Area

Upland Areas

There are no records indicating nesting of leatherback sea turtles on Louisiana beaches. As they are not known to nest within the proposed action area, they are unlikely to utilize upland areas there. In the Gulf of Mexico, leatherback sea turtles nest at low densities along the Florida, Alabama, and Mexican coasts. The closest known nesting beaches for leatherback sea turtles are found in Alabama, over 400 kilometers to the east of the action area (Valverde and Holzwart 2017).

Locations of leatherback sea turtle nesting are shown in Figure 4.6.9-1.

Aquatic Areas

Juvenile or adult leatherback sea turtles may occur in marine aquatic areas of the proposed action area while migrating, resting, or foraging. Non-nesting, adult female loggerheads are reported throughout the U.S. Atlantic, Gulf of Mexico, and Caribbean Sea (NOAA 2017c).

While leatherback sea turtles are not uncommon in Louisiana, they are typically observed offshore (Fuller et al. 1987). There are no stranding or observational records of leatherback sea turtles in Barataria Basin. Juvenile leatherbacks are believed to use waters warmer than 26^{o} C found in tropical and sub-tropical latitudes, whereas adults may range into temperate waters as cold as 8^{o} C (Eckert 2002). Reports of leatherback sea turtles in Louisiana are typically associated with divers, such as those diving on offshore oil platforms (Fuller et al, 1987). While leatherbacks are a wide-ranging species, they are primarily pelagic and only found in coastal waters when mating or nesting (Eckert et al. 2012), and therefore are not expected to occur in the action area. Leatherback sea turtles may occur incidentally and for short durations in the action area if they are injured, if they are following large aggregations of prey, or they are migrating through adjacent waters.

4.6.10 Loggerhead Sea Turtle Use of Action Area

Upland Areas

Loggerhead sea turtles rarely nest within the proposed action area. Therefore, they are unlikely to use upland areas in the action area. Two records of adult female loggerhead sea turtles nesting on Grand Isle on June 29 and July 3, 2015 represent the first confirmed sea turtle nesting on the coast of Louisiana for 30 years (Louisiana Sportsman 2015). Since sea turtles typically return to their natal beaches, it is possible that future nesting activity will occur near Grand Isle when the hatchlings mature and return to nest. Turtle nesting crawls (i.e., aborted nesting attempts) have been observed on other nearby islands including Elmer Island; however, no other nest has been confirmed (LDWF 2016). The northern Gulf of Mexico subpopulation of loggerheads is one of the smallest nesting aggregations in the Atlantic and the second smallest in the western North Atlantic (TEWG 2009). The nesting beaches of this subpopulation are concentrated in the Florida Panhandle, with a consistent but small amount of nesting in other Gulf states, mostly Alabama and Texas. Locations of loggerhead sea turtle nesting areas are shown in Figure 4.6.10-1.

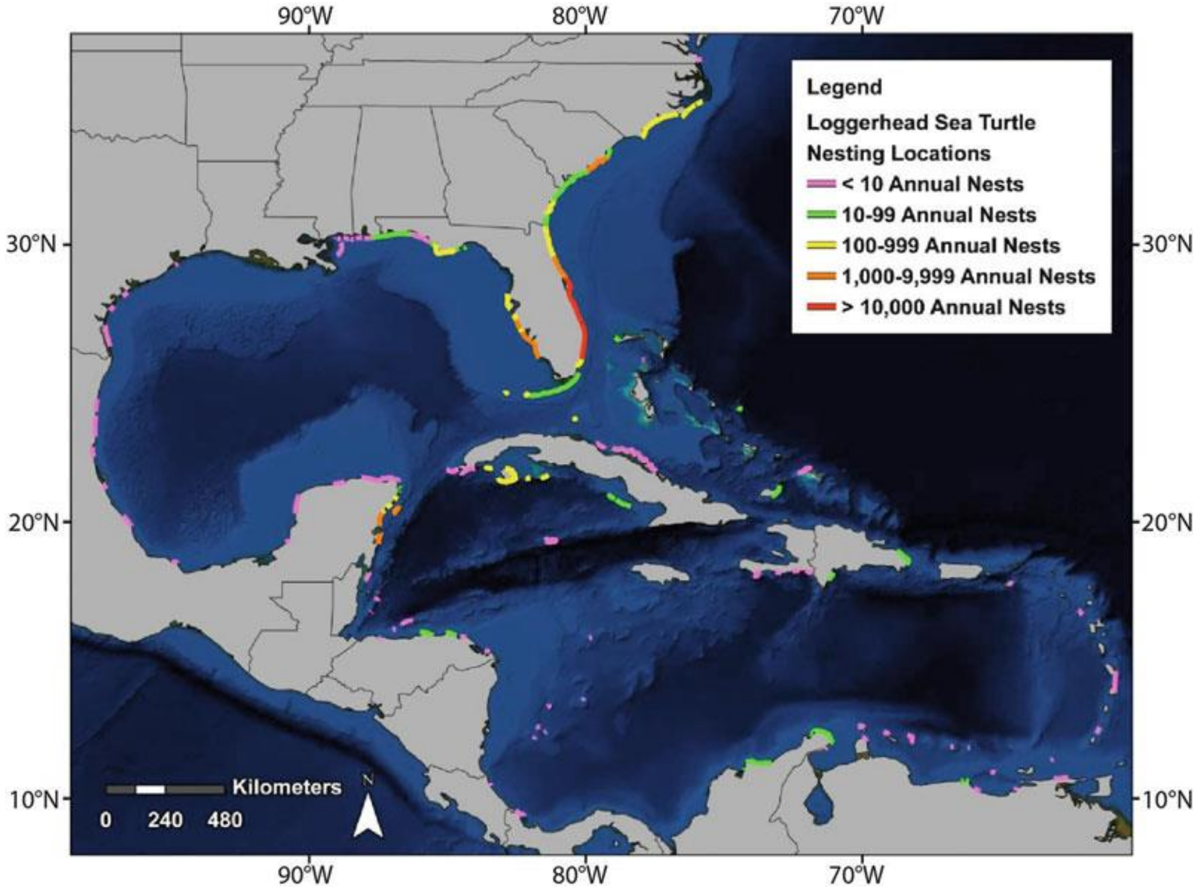


Figure 4.6.10-1. Generalized Nesting Beach Locations of the Northwest Atlantic Ocean DPS of Loggerhead Sea Turtles

(interpreted from Dow et al. 2007, NMFS and USFWS 2008, SWOT 2007a) (Source: Valverde and Holzwart 2017)

Aquatic Areas

Post-hatchling and adult loggerhead sea turtles may use marine aquatic areas of the proposed action area while migrating, resting, or foraging (Valvedere and Holzwart 2017). Post-hatchling loggerheads are observed in both deep neritic and pelagic waters (Witherington et al. 2012). Juvenile loggerheads rear in pelagic waters where they develop for several years before returning to neritic and nearshore habitats as sub-adults that are larger than 30-40 cm straight carapace length (Valverde and Holzwart 2017). Adults are often associated with hard substrates including reefs and anthropogenic structures (Rosman et al. 1987).

Essentially all shelf waters along the Gulf of Mexico shoreline are inhabited by loggerheads (Conant et al. 2009), but shelf habitat does not occur within the proposed action area. Since 1998, a single loggerhead stranding has been reported that occurred (in 2018) within the reporting area that includes Barataria Basin (NOAA 2019). Large juvenile and adult loggerhead sea turtles are likely present at low abundances in the action area.

4.7 Other Projects included in the Environmental Baseline

Several ongoing marsh restoration projects in the area are either permitted, under construction, or recently completed and have completed their federal consultations. Those projects include the following:

- **Spanish Pass Increment of the Barataria Basin Ridge and Marsh Creation** – beneficial re-use of dredged materials from authorized navigation projects to restore a historic ridge backed by a marsh platform.
- **Bayou L'Ours Marsh Terracing** – creating marsh terraces using existing soil to create segments of marsh that are aligned to reduce erosion.
- **Caminada Headland Back Barrier** – creating and/or nourishing 385 acres of back barrier marsh by pumping sediment from an offshore borrow site. Also, creating a platform upon which the beach and dune can migrate.
- **West Grand Terre Beach Nourishment and Stabilization Project** – construction of approximately 12,700 feet of beach and dune with an area of 235 acres, a back-barrier marsh, and a rock revetment to protect the restored marsh.

Because these projects have completed their federal ESA consultation, the effects of these projects are incorporated into the baseline conditions.

5.0 ANALYSIS OF EFFECTS

This section describes the potential direct and indirect effects from the Project on listed species and designated critical habitat. This includes potential effects from Project elements for construction, operation, and maintenance as summarized in the Deconstruction Table of Project Construction and Operation Activities and Effects (Table 5.1-1). This information is then compared to the ESA listed species' general life history (Section 3.2), potential presence in the action area (Section 4.6), and species tolerances (Section 5.3) to describe the range of potential effects to listed species and habitats. This discussion is then followed by a review of interactions between habitat, prey resources, and predator-prey relationships to assess trophic level effects associated with the proposed Project. Where appropriate, indirect effects such as feedback loops are assessed.

Proposed conservation measures for the proposed Project discussed in Section 2.3.6, and monitoring and adaptive management measures discussed in Section 6.0 are also considered in the effects analysis. This information is summarized in the Effects Determination (Section 7.0).

5.1 Deconstruction Table of Project Construction and Operation Activities and Effects

The Deconstruction Table (Table 5.1-1) summarizes the relationship between Project activities and environmental attributes that may affect ESA listed species. The deconstruction table addresses Project activities and major environmental attributes and habitat qualities (i.e., Project effect pathways) important to listed species and their habitats that may be affected by the Project. Potential effects range from no interaction or effect to minor and major effects from the Project activity on the environmental effect pathway. Minor effects are interactions where there is an expectation that the effect pathway will be affected; however, that affect will be small and may not be detectable at the scale of the Project and may be within the range of disturbance attributable to seasonal or natural variation (i.e., insignificant and/or discountable). Major effects are those where a measurable and potentially substantial change to an effect pathway is predicted to occur. The major changes to an effect pathway are then further analyzed to determine the potential extent of effect to listed species or designated critical habitats.

Table 5.1-1. Deconstruction Table of Project Construction and Operation Activities and Effects

Project Description							Effect Pathways												
Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
Diversion Complex	All	Diversion Operation	Operation	Baseline diversion flow (5,000 cfs diverted)	A	R,B	xx	xx	xx	xx	x	xx	xx	xx	n	xx	n	x	n
Diversion Complex	All	Diversion Operation	Operation	Intermediate flow (between 5,000 and 75,000 cfs diverted)	A	R,B	xx	xx	xx	xx	xx	xx	xx	xx	n	xx	n	xx	n
Diversion Complex	All	Diversion Operation	Operation	High diversion flow - river >1,000,000 cfs (75,000 cfs diverted)	A	R,B	xx	xx	xx	xx	xx	xx	xx	xx	n	xx	n	xx	n
Diversion Complex	All Diversion Complex Features (Intake Channel, Diversion	Construction Activities: Phase 1	Site Prep	Clearing and grubbing: limits of terrestrial construction	T	R, B	n	n	n	n	n	n	n	n	n	n	n	n	n

Project Description							Effect Pathways												
Project Element	**Project Feature**	**Project Action & Phase**	**Action Category**	**Action**	**Habitat (Aquatic/ Terrestrial)**	**Location of Aquatic Interaction (River/Basin)**	**Salinity**	**Turbidity/SS**	**Nutrients (N and P)**	**Dissolved Oxygen**	**Water Temp**	**pH**	**Sediment Deposition/ Land Creation**	**Water Quality/ Contaminants**	**Sound/Noise Effects**	**Entrainment/Stranding**	**Physical Disturbance of Species**	**Change in Habitat Type**	**Shading (over water structure)**
	Structure, Outfall Transition Feature, Guide Levees)			Access: haul road excavation and construction, unloading areas, parking pads, fencing	T	R, B	n	n	n	n	n	n	n	n	n	n	n	n	n
				Staging: constructing and/or stabilizing staging areas	T	R, B	n	n	n	n	n	n	n	n	n	n	n	n	n
				Clearing and Grubbing: limits of aquatic construction	A	R, B	n	x	n	n	n	n	n	n	n	n	x	x	n
				Access: dredging for barge access (basin side)	A	B	n	x	x	x	n	n	n	n	x	x	x	x	x
				Equipment and materials staging: barge	A	R	n	n	n	n	n	n	n	n	n	n	n	n	x

Project Description							Effect Pathways												
Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
		Construction of Foundation Systems: Phase 1-3	Construction	Pile driving (cofferdam and trestle)	A	R	n	n	n	n	n	n	n	n	x	x	x	n	n
				Trestle	A	R	n	n	n	n	n	n	n	n	n	n	x	x	x
				Surcharge area with excess fill to consolidate sediments	T	R, B	n	x	n	n	n	n	n	x	N	n	n	n	n
				Dewatering/ rewatering	A	R, B	n	x	n	n	n	x	n	x	n	x	x	n	n
				Excavation	A	R, B	n	x	n	n	n	n	n	x	n	n	x	x	n

Project Description							Effect Pathways												
Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
	Inlet Channel	Construction Activities: Phase 2-3	Construction	In-the-dry construction: headworks - sheet pile installation/ removal	A	R	n	n	n	n	n	n	n	n	x	n	x	n	n
				In-the-dry construction: dewatering/ rewatering	A	R	n	x	n	n	n	n	n	n	n	x	x	x	n
				Sediment excavation and disposal	A	R	n	x	n	n	n	n	n	x	n	n	x	x	n
				Staging during construction. Barge delivered materials and equipment	A	R	n	n	n	n	n	n	n	n	n	n	n	x	x
	Diversion Structure	Construction Activities: Phase 1-3	Construction	In-the-dry construction: sheet pile installation	A	R	n	n	n	n	n	n	n	n	x	n	x	n	n

Project Description							Effect Pathways												
Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
				In-the-dry construction: dewatering/ rewatering	A	R	n	x	n	n	n	x	n	n	n	x	x	n	n
				Sediment excavation and disposal	A	R	n	x	x	n	n	n	n	x	n	n	n	x	n
				Sediment excavation: mechanical	A,T	R	n	x	x	x	n	n	x	x	n	n	x	x	n
				Sediment excavation: hydraulic	A,T	R	n	x	x	x	n	n	x	x	n	n	x	x	n
				Sediment excavation: laydown area for processing clay borrow	A,T	R	n	n	n	n	n	n	n	n	n	n	n	n	n
	Diversion Structure & Transition Structure	**Construction Activities: Phase 2-3**	**Construction**	In-the-dry construction: transition walls and installation of armoring	A,T	B	n	x	n	n	n	x	n	n	n	x	x	n	n

Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
Project Description							Effect Pathways												
Basin Outfall Area	Outfall Transition Feature	Construction Activities: Phase 1	Site Prep	Clearing and grubbing the limits of aquatic construction	A	B	n	x	n	n	n	n	n	n	n	n	x	x	n
		Construction Activities: Phase 2-3	Construction	If in-the-dry construction: drive sheet pile	A	B	n	n	n	n	n	n	n	n	x	n	x	n	n
				If in-the-dry construction: dewatering/ rewatering	A	B	n	x	n	n	n	n	n	n	n	x	x	n	n
				Staging during construction: barge stored materials and equipment	A	B	n	n	n	n	n	n	n	x	n	n	x	x	x
				Dredging/ excavation	A	B	n	x	n	n	n	n	n	x	n	n	n	x	n
Auxiliary Features	Linear Infrastructure	Auxiliary Activities	Railway Bridge (NOGC)	Railway: bridge construction	T	R	n	n	n	n	n	n	n	n	n	n	n	x	n
			Highway LA 23	Raised and Relocated	T	R	n	n	n	n	n	n	n	n	n	n	n	x	n
			Utilities - Power	Relocation: existing power right-of-way	T		n	n	n	n	n	n	n	n	n	n	n	n	n

Project Description							Effect Pathways												
Project Element	Project Feature	Project Action & Phase	Action Category	Action	Habitat (Aquatic/ Terrestrial)	Location of Aquatic Interaction (River/Basin)	Salinity	Turbidity/SS	Nutrients (N and P)	Dissolved Oxygen	Water Temp	pH	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Sound/Noise Effects	Entrainment/Stranding	Physical Disturbance of Species	Change in Habitat Type	Shading (over water structure)
			Utilities - Fiber Optic	Relocation: existing Fiber Optic right-of-way	T		n	n	n	n	n	n	n	n	n	n	n	n	n
			Utilities - Water	Relocation: 16-inch water main for Plaquemines Parish	T		n	n	n	n	n	n	n	n	n	n	n	n	n
			Drainage System	Siphon drain option	A,T	R	x	x	x	n	n	n	n	n	n	n	n	x	n
	Beneficial Use Placement Areas		BU Areas	BU Areas	A	B	n	x	n	x	n	n	n	n	n	n	n	x	n
Diversion Complex	ALL	Maintenance of Sediment Diversion	Maintenance	Debris management	A	R, B	n	x	n	n	n	n	n	n	n	n	x	n	n
				Channel repairs/ modifications	A	R, B	n	x	n	n	n	n	n	n	x	x	x	n	n

SS = suspended sediments; N = nitrogen; P = phosphorus; DO = dissolved oxygen; BU = beneficial use placement
n = No effect or negligible effect
x = Minor effect (e.g., short duration, small geographic extent).
xx = More than Minor effect (to be assessed in more detail)

5.2 Delft3D Model Overview

The Barataria Basin is a dynamic system which has experienced land loss, and is predicted to continue to experience land loss into the future. The basin is also predicted to be impacted by sea level rise. These changing baseline conditions are expected to influence a wide range of environmental conditions within Barataria Basin. Therefore, the Project team has worked with The Water Institute of the Gulf (TWI) to develop a basin-wide model that can be used to assess conditions in the basin at various points in time with and without the Project and Project alternatives.

TWI has developed successive versions of the basin-wide Delft3D model to simulate morphological changes and water quality-related dynamics in the Mississippi River and in the Barataria and Breton Sound basins, including the Birdfoot Delta. The Delft3D model incorporates the existing Breton Sound Basin connections to the Mississippi River at Fort St. Philip and Bayou Lamoque, as well as Breton Sound Sediment diversion operations (Sadid et al. 2018). The Delft3D model is a modeling suite developed by Deltares (2014) and designed to model "hydrodynamics, sediment transport and morphology and water quality for riverine, estuarine, and coastal environments" (Sadid et al. 2018). As developed by TWI, the Delft3D model integrates several modules, including hydrodynamics, morphodynamics, nutrient dynamics, and vegetation dynamics. Vegetation dynamics are modeled using 2 Louisiana-specific vegetation modules to simulate the spatial distribution of wetland vegetation and allocate biomass above and below-ground.

The results presented here and used in the evaluation of alternatives are based on Version 3 of the basin-wide Delft3D model, implemented specifically to model the proposed Project and Project alternatives. The Delft3D model predicts how conditions would change over 50 years for each Project alternative, including changes in wetland area, water level, water quality (including salinity), and vegetation characteristics. Many of the results from the Delft 3D model are expressed as the difference between the "future with Project" (FWP) and "future without Project" (FWOP) scenarios. Delft3D modeling predictions allow for comparisons of environmental conditions over time with and without the proposed Project.

Model Description

The model domain covers Barataria and Breton basins and the Mississippi River delta. Adjacent bays were included to account for water and nutrient exchange and longshore currents. The model domain was intentionally sized larger than the action area to allow for potential far-field effects of larger-scale restoration projects and to avoid influences from model boundaries. Model outputs are at multiple scales with the finest grid resolution (100 m X 100 m) near the proposed sediment diversion outfall with the grid size gradually increased (and resolution reduced) with distance from the outfall areas. Most of Barataria Basin is characterized at 100 m X 100 m grid size, increasing to 200 m X 200 m grid size in the Birdfoot Delta and 400 m X 400 m grid size in outer basin areas. Far field locations in the Gulf of Mexico are characterized at a 2 X 2 km to 4 X4 km grid size.

Model Assumptions

Sea level rise (SLR) is incorporated in the model based on the 2017 Master Plan moderate prediction of a 1.5-meter increase by 2100 (CPRA 2017). Relative to the NAVD88, sea level elevations are predicted to increase from 0.0 meters in 2015 to 0.04 meters in 2020, 0.13 meters in 2030, 0.25 meters in 2040, 0.39 meters in 2050, 0.54 meters in 2060 and 0.72 m in 2070.

The model implementation uses a series of assumptions about the Mississippi River hydrograph, the landscape in the basin, and representative simulation of initial conditions for vegetation distribution in the basin. Historic hydrograph conditions from the past 50 years are used in the model to represent future conditions on a decadal scale (see Table 5.2-1). Hydrograph locations and geographic references for the Project are shown in Figure 5.2-1.

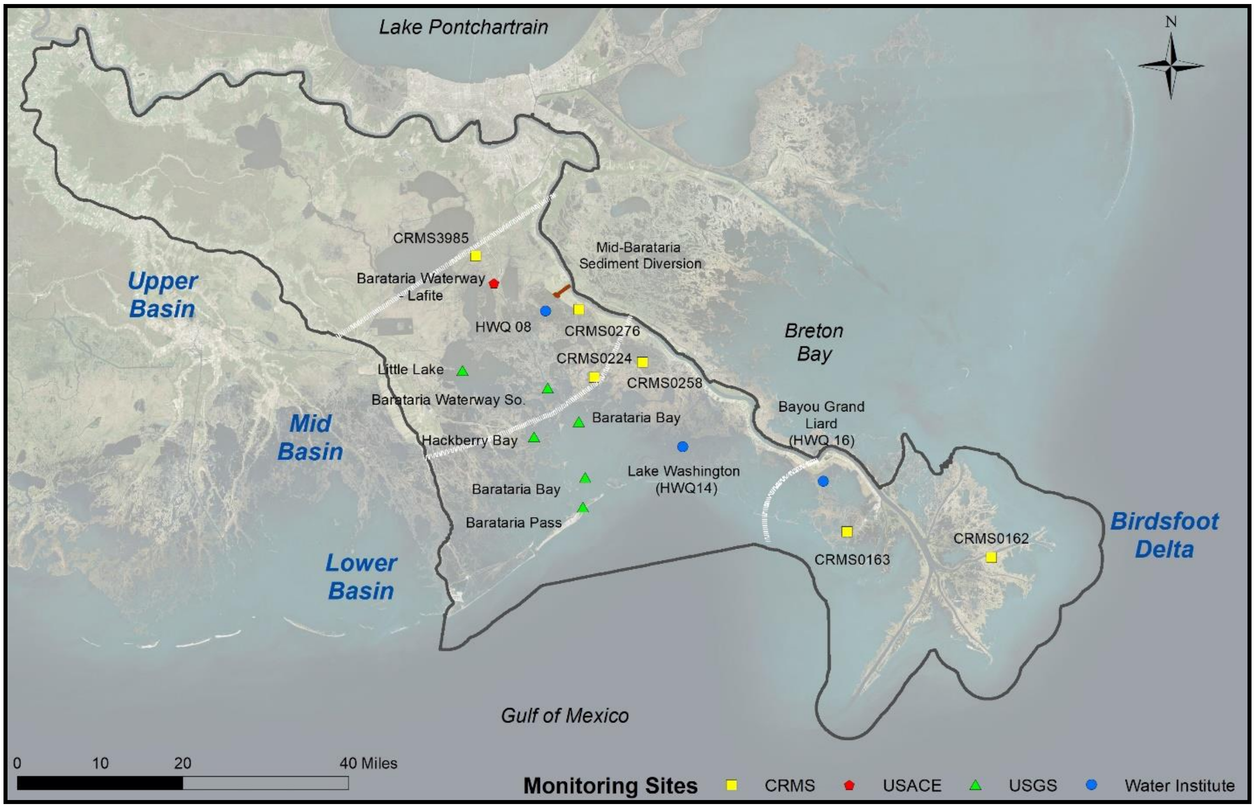


Figure 5.2-1. Hydrograph Locations and Geographic References

(Source: Modified from The Water Institute)

Table 5.2-1. Delft3D Model Simulation Components

Decade	Cycle	Time Period	Simulation Length (yrs)	Hydrology and Water Quality Simulations			
				Representative Year*	Simulated Landscape**	Model Name	Additional Simulations
Initialization	Initialization	2015-2019	5	2014	2015	Yr 0	1994, 2006, 2010, 2011
First	0	2020-2029	10	1970	2020	Yr 1	1994, 2006, 2010, 2011
Second	1	2030-2039	10	1975	2030	Yr 10	1994, 2006, 2010, 2011
Third	2	2040-2049	10	1985	2040	Yr 20	1994, 2006, 2010, 2011
Fourth	3	2050-2059	10	2002	2050	Yr 30	1994, 2006, 2010, 2011
Fifth	4	2060-2069	10	2008	2060	Yr 40	1994, 2006, 2010, 2011
Sixth	5	2070	1	2008	2070	Yr 50	1994, 2006, 2010, 2011

*Used to estimate vegetation spatial distribution and organic accretion
** Topography/bathymetry/vegetation distribution

Sediment transport and deposition by the Mississippi River has been measured over a lengthy historical record. While average annual conditions are captured in the historical record, the sediment in the river is also subject to storm-scale hysteresis effects where larger amounts of sediment are mobilized during the rising limb of a storm event's hydrograph than during the peak and falling limb. Hysteresis effects exist where peak sediment concentrations generally *precede* peak river discharge rates and, in turn, sediment concentrations are usually *lower* during the peak river discharges of most high flow events. This occurs because the finest sand fractions are mobilized during initial phases of the event. Stratigraphic signatures of flood events suggest that sediment volume is more closely related to the duration and total suspended load of the event rather than the magnitude of the peak discharge (Benedetti 2003). TWI has generated Delft3D model outputs using both traditional and hysteresis assumptions for sediment concentrations. Mossa (1989) found that the lower Mississippi River has pronounced hysteresis effects, especially during high discharge years when sediment concentrations and load peaks precede discharge peaks by several months. Therefore, the analysis presented here relies on the hysteresis outputs, which more closely describe the sediment transport conditions expected for this Project.

Model Caveats

Numerical models can be used to describe coastal systems by providing information on the physical and environmental conditions of water and can be used to predict oceanographic variables. The Delft 3D model is a simplified representation of existing and future conditions with and without the Project. There are aspects of the natural environment, such as sediment movement during low flow events and sudden large changes/oscillations in river flow, which models do not predict well. In general, models are best suited for predictions regarding events that are like the historical record and may have limited utility as actual conditions diverge from that record. Further, predictions tend to be best for near-term predictions. Long-term prediction are built on earlier predictions, and thus any errors or inaccurate predictions become magnified. Therefore, uncertainties in future predictions tend to become magnified as predictions get further from the current period. In this case the future scenarios are also based on future predictions for SLR, which may ultimately be higher or lower than the predictions used here.

5.3 Direct and Indirect Effect Pathways

Direct effects include all immediate effects (adverse and beneficial) from Project-related actions, as well as disturbances that are directly related to Project elements that occur very close to the time of the action itself. Indirect effects include those effects that are caused or would result from the proposed action and are later in time but are still reasonably certain to occur (50 CFR 402.02). This section discusses the effect pathways generated from Project activities (e.g., changes to salinity, turbidity, etc.). These effect pathways are described in terms of frequency, intensity, duration, and or areal extent at appropriate temporal scales (e.g., construction years 1-3, operation years 1-10, 11-20, etc.) corresponding with available data and modeling output. Effects associated with the construction of the diversion complex and auxiliary structures are primarily evaluated by evaluating the geography and timing when activities occur and evaluating the potential influence on effect pathways. Long-term diversion operations are evaluated using a physical model (Delft 3D) to generate predictions of future conditions with and without the Project. The Delft 3D model, which is described further in Section 5.2 above, was used to simulate hydrodynamic conditions (e.g., water level, water flow velocities, and salinity) and resultant physical condition in Barataria Basin.

The main drivers of habitat changes throughout the action area due to the Project are salinity, water temperature, and land formation/reduced wetland loss. Many of these characteristics are also forecast to change as a result of future climate and sea level conditions. Figure 5.3-1 illustrates the major trends of these drivers of change in the FWOP and FWP scenarios. These drivers and additional details are discussed throughout Section 5.3.

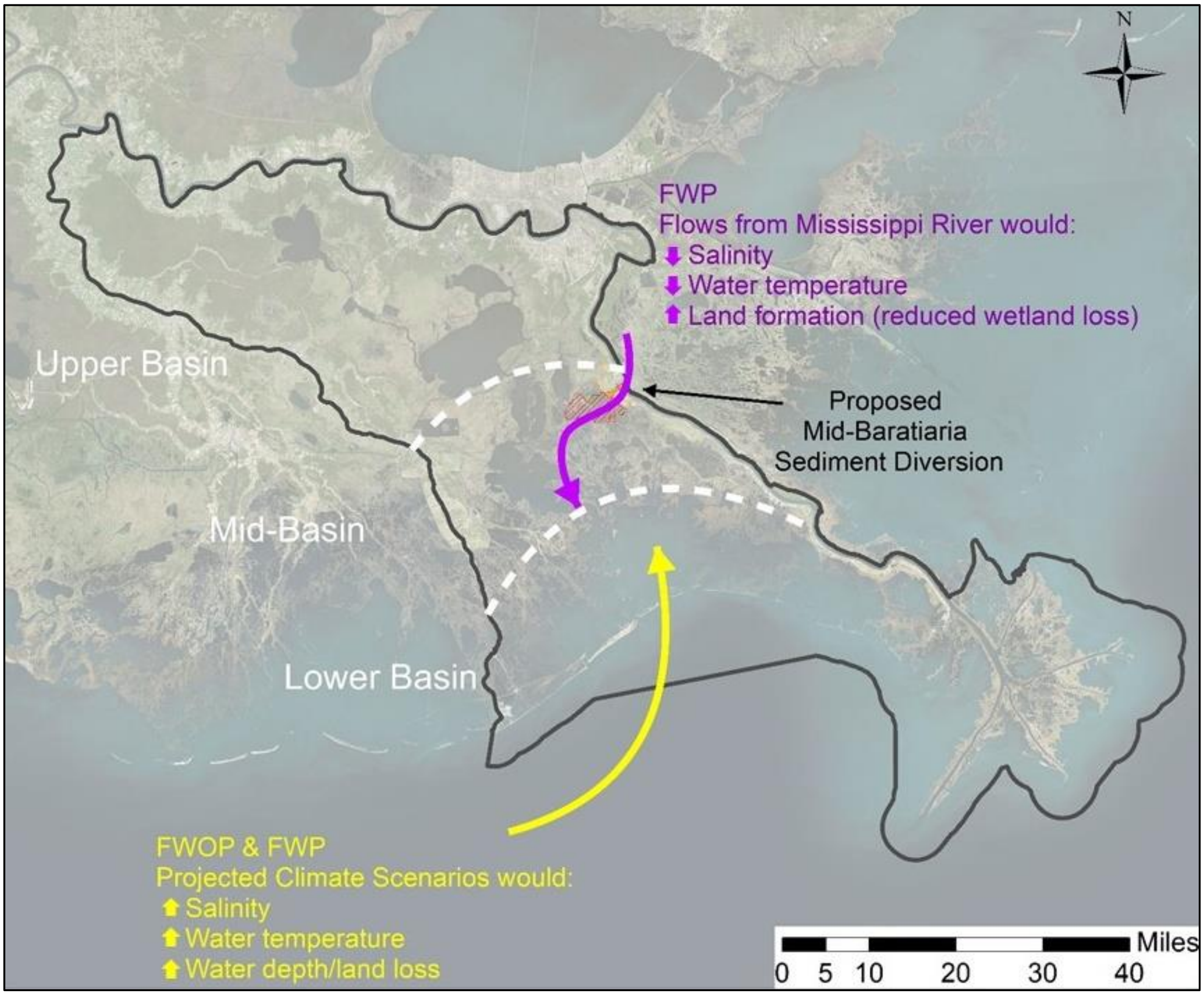


Figure 5.3-1. FWP and FWOP Main Effects on Habitat in Barataria Basin and the Birdfoot Delta

5.3.1 Project Effects on Salinity

Project Construction

There would be no Project effects on salinity due to construction activities.

Project Operation

In response to SLR, saltwater intrusion into the estuary is anticipated to increase over time. As such, average salinity throughout the basin is anticipated to increase over time, though effects would be most consequential in the lower–mid and lower regions of Barataria Basin. The projections of effects of the Project described below include the influence of the predicted conditions in the FWOP trajectories and trends, as effects from climate change and SLR would continue to exert their influence.

The proposed Project would divert fresh water from the Mississippi River into the brackish Barataria Basin (current salinity conditions throughout the action area are described in more detail in Section 4.2.2 above). The majority of Barataria Basin is estuarine, with low year-round salinity (0-10 practical salinity units [psu]) in the upper and mid-basin, and regular seasonal influxes of more saline marine waters in the lower basin (10-20 psu). Compared to the FWOP, the Project is anticipated to decrease salinity throughout Barataria Basin, with strongest effects on the southern half of the basin below the diversion outfall. Modeled Project effects predict lowered salinity throughout the mid and lower regions of Barataria Basin, throughout all seasons of the year. Salinity effects due to the diversion are primarily during periods of operation and immediately following the reduction of the diversion to base flow; however, base flow (5,000 cfs) would continue to exert an influence on salinity. Salinity effects are most remarkable during months of highest river flow (above 450,000 cfs) when the diversion would be in operation, increasing incrementally until a maximum diversion of 75,000 cfs. Salinity changes due to the Project are not anticipated to extend north of the diversion into the predominantly fresh upper Barataria Basin. Salinity in the Birdfoot Delta is predicted to be minimally higher as a result of the Project diverting volumes of fresh water upriver from the delta. An overview of predicted salinity trends over time within the action area is shown above in Figure 5.3.1, while a more specific comparison of salinity conditions in FWP and FWOP, within each Delft3D modeled region, is shown below in Figure 5.3.1-1.

The largest changes to salinity would occur in the mid-basin region of Barataria Basin, near the diversion outfall. Changes to salinity as a result of the Project are most noticeable during periods of peak river flow and diversion flow (January – June). The diversion would be adding a new source of freshwater flow into the basin, decreasing salinity substantially (by ≤ 8 ppt[2] lower than the FWOP, to a minimum of 0 ppt) adjacent to the diversion outfall. The magnitude of salinity changes decreases with increasing distance from the diversion outfall. The second greatest changes to salinity are anticipated to be on the north side of the barrier islands, in South Barataria Bay, during all periods where the diversion is operating. The barrier islands are an area of mixing of the basin's estuarine waters and more saline nearshore Gulf waters, but Project additions of flow and fresh water would likely move the mixing zone slightly south, and decrease salinity substantially (by ≤ 6 ppt lower than the FWOP) just north of the barrier islands during springtime months. However, the barrier islands, due to mixing with high salinity Gulf waters, have higher salinity than the rest of Barataria Bay during most of the year; therefore, the lowest monthly salinity predicted at the barrier islands due to the Project is estimated to be approximately 1.1 ppt, as compared to a minimum of 3.9 ppt in FWOP in 2070. Salinity conditions south of the barrier islands are primarily driven by nearshore and oceanic processes in the Gulf and effects due to the Project are constrained to the immediate vicinity of the barrier islands.

2 Although salinity is more commonly measured in the field as "practical salinity units" (psu), this document describes salinity changes in units of "parts per thousand" (ppt), as reported by the Delft3D modeled outputs. Practical salinity units are approximately equivalent to ppt.

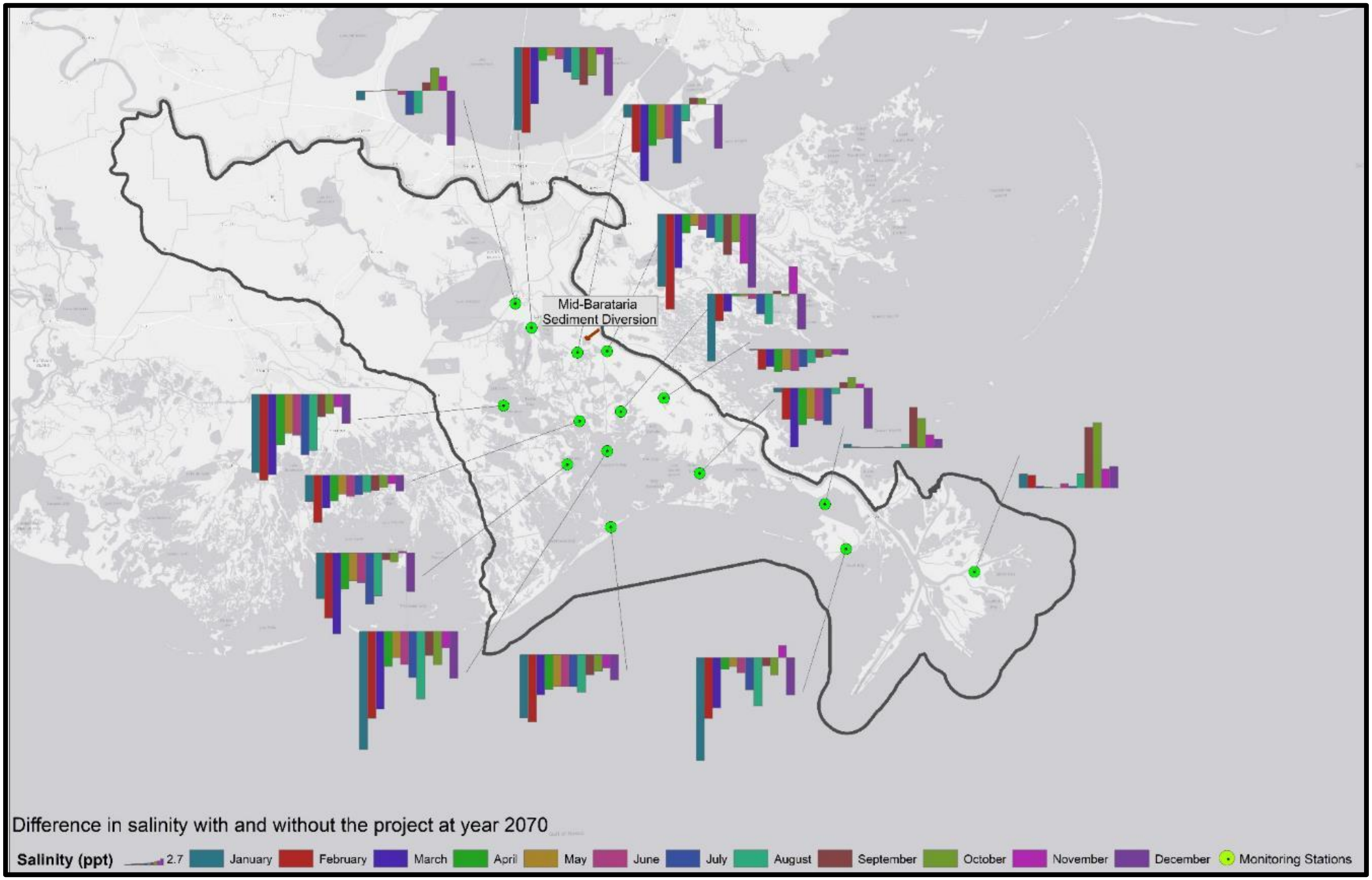


Figure 5.3.1-1. Project Effects on Salinity over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

Changes to salinity can have consequences to aquatic plants and animals in the basin, expanding areas available for some species, and restricting suitable areas for others. Project driven changes to salinity have the potential to result in changes to habitats (e.g., species of composition of marsh vegetation) and are predicted to shift marsh areas in the mid-basin from brackish marsh to fresh and intermediate marshes. These are in addition to changes over time in both the FWP and FWOP scenarios where saline marshes in mid and lower Barataria Basin are predicted to continue to reduce in area, followed by subsequent losses of substantial areas of brackish marsh due to continued SLR and coastal erosion. Average salinities throughout the basin with and without the Project, divided by habitat type, are described below in Table 5.3.1-1. The Project would not alter the predominantly fresh upper basin.

Table 5.3.1-1. Predicted Average Salinity in Barataria Basin by Habitat Type

Time Period	Habitat Type	Salinity (psu)	
		FWOP	FWP (75k cfs)
Cycle 0 (2020 - 2029)	Fresh + Intermediate	1.0	0.4
	Brackish	3.8	2.4
	Saline	7.7	9.9*
Cycle 1 (2030 - 2039)	Fresh + Intermediate	1.2	0.4
	Brackish	3.5	2.9
	Saline	8.5	9.0*
Cycle 3 (2040 - 2049)	Fresh + Intermediate	1.6	0.3
	Brackish	3.6	3.6
	Saline	8.5	8.1
Cycle 4 (2050 - 2059)	Fresh + Intermediate	1.8	0.5
	Brackish	3.7	2.7
	Saline	7.7	6.5
Cycle 5 (2060 - 2069)	Fresh + Intermediate	1.5	0.2
	Brackish	3.1	2.6
	Saline	NA	NA
Year 50 (2070)	Fresh + Intermediate	1.7	0.4
	Brackish	3.8	NA
	Saline	NA	NA

NA = not applicable, no habitat of this type
Source: The Water Institute 2019
* Note: Salinity calculations are based on the area within each habitat type. In the FWP scenarios at all time periods the area that is saline decreases. In some time periods this may cause salinity to appear to increase in saline habitat areas. This is because lower salinity areas in this habitat class shift to brackish habitat in the FWP.

5.3.2 Project Effects on Temperature

Project Construction

There would be no Project effects on temperature due to construction activities.

Project Operation

In response to global climate change, water temperatures throughout Barataria Basin are anticipated to increase over time, with changes being most pronounced in winter months (up to 3°C increase). The following analysis of effects of the Project includes the influence of the predicted conditions in the FWOP trajectories and trends, as they would continue to exert their influence.

The proposed Project would divert predominantly colder flows from the Mississippi River into the Barataria Basin. Annual river temperature in the Mississippi River ranges from 6.6°C to 30.0°C, while the average water temperature in Barataria Basin ranges from 16°C to 30°C. The temperature differential between the Mississippi River and Barataria Basin is the highest between February and May, with model results predicting a maximum of 6.6°C differential adjacent to the outfall during cycle 3 (2040 to 2050). Additional detail on current water temperature conditions throughout the action area are described in Section 4.2.3 above.

As predicted by the Delft3D modeling results, the Project would add cooler flows from the Mississippi River to the Barataria Basin, decreasing temperatures through much of the basin, with the main effects restricted to the mid-basin region of Barataria Basin near the diversion outfall (see Figure 5.3.2-1). Changes to water temperature due to the Project are primarily during peak river flow and diversion flow, and when the temperature differential between the Mississippi River and Barataria Basin is the highest. Project effects on water temperature are predicted to be substantial adjacent to and south of the diversion outfall, through the mid-basin and into the northern half of the lower basin. The temperature differential between the Mississippi River and Barataria Basin is the highest between February and May, with a maximum of 6.6°C differential predicted adjacent to the outfall during cycle 3 (2040 to 2050). Over time, the magnitude of seasonal temperature effects are predicted to decrease slightly, but would largely remain the same through the first 50 years, with effects being mostly restricted to the mid-basin region of Barataria Basin. No temperature effects from the Project are anticipated in the southernmost region of Barataria Basin, along the barrier islands, or in the Birdfoot Delta.

The addition of Mississippi River flow to the mid-basin would change the distribution and timing of temperature-bounded habitats and species in the Barataria Basin. Changes to water temperature can have consequences to aquatic plants and animals in the basin, expanding habitat available for some species, and restricting available habitats for others.

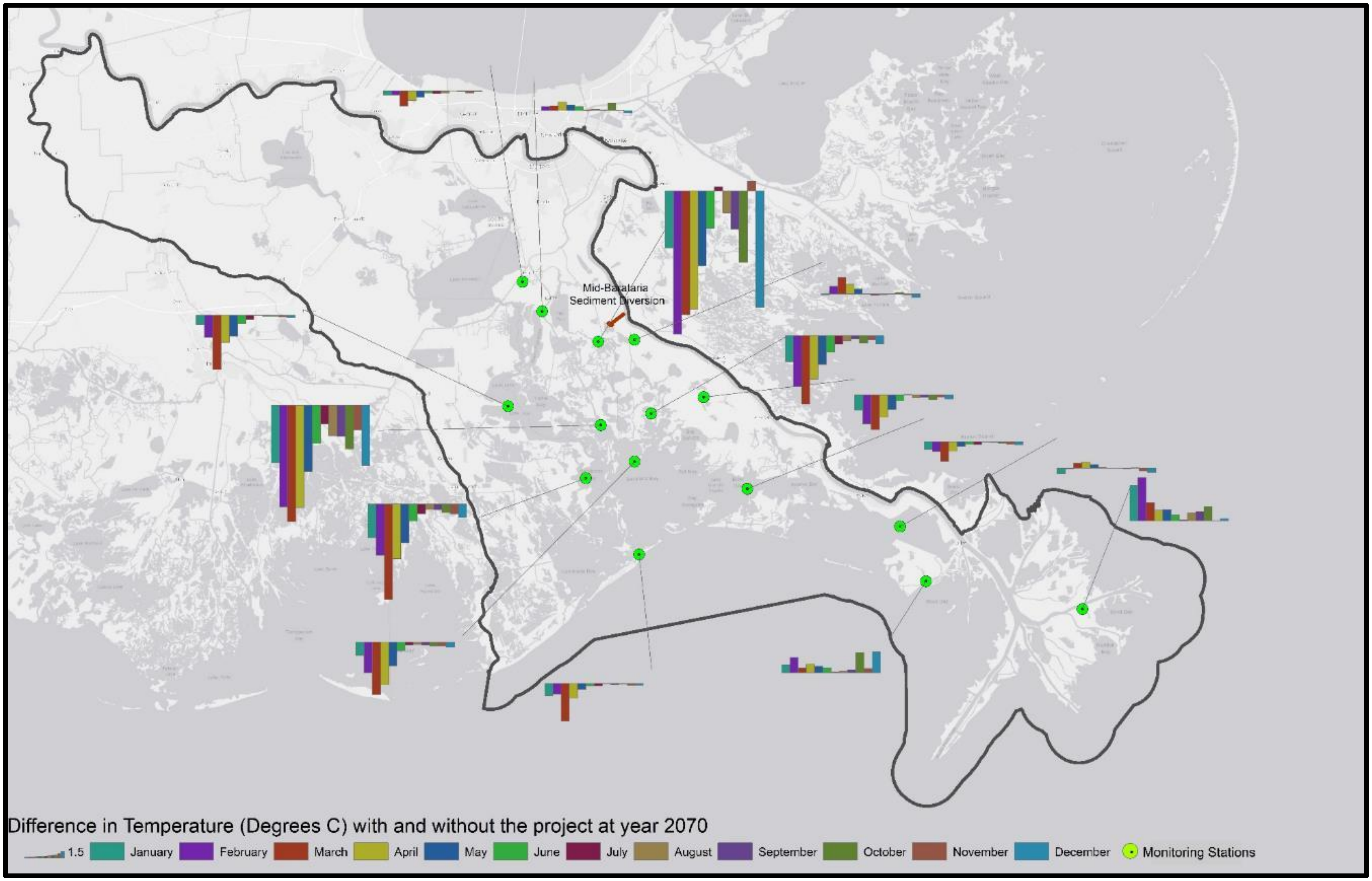


Figure 5.3.2-1. Project Effects on Temperature over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

5.3.3 Project Effects on Turbidity and Suspended Sediment

Project Construction

Numerous construction activities would cause temporary increases in turbidity and suspended sediments within both the Mississippi River and Barataria Basin.

Project construction activities on the riverside of the diversion would include approximately 8 acres of excavation and the construction of a cofferdam composed of 60-foot-wide cells at the diversion intake apron, as well as the construction of a temporary barge-access facility integrated into the downstream side of the cofferdam. Excavation, cofferdam, and pier construction activities are planned to occur during the first phase of construction and remain in place throughout the 5-year construction period. The cofferdam is expected to contain most turbidity effects along the Mississippi River; however, removal of the cofferdam near the end of construction may mobilize sediment and cause a temporary increase in turbidity as river currents are re-introduced to this area.

Project construction activities on the basin-side of the diversion would include the excavation of approximately 34 acres of material during diversion outfall construction, 2 miles of channel dredging (70 feet wide by 4 feet deep) to support outfall construction, and deposition of large quantities of excavated materials throughout the construction process within designated BU placement areas in Barataria Basin.

Each of these construction activities would temporarily increase turbidity and suspended sediments in and around the areas where they occur. Increased turbidity can have consequences to plants and animals present, such as smothering of benthic vegetation and invertebrates, reducing DO levels, and species displacement from highly turbid areas.

Project Operation

In the FWOP, total suspended solids (TSS) is generally low (<50 g/m^3) year-round throughout the majority of Barataria Basin (see Figure 5.3.3-1). The model also shows periods when TSS is elevated near the barrier islands (<100 g/m^3). The analysis of the Project below includes the influence of predicted conditions in FWOP trajectories and trends in addition to Project effects, as they will continue to exert their influence.

The proposed Project would divert sediment-laden water from the Mississippi River into the mid-basin region of Barataria Basin (current turbidity and suspended sediment conditions throughout the action area are described in more detail in Section 4.2.5 above). The purpose of the diversion is to increase sediment deposition in the Barataria Basin in support of land building and marsh maintenance into the future. Project operations would increase the frequency of sediment input into the basin as compared to the FWOP, and would result in changes to the distribution and maintenance of land area and emergent marsh habitats in the basin over time.

The proposed Project is anticipated to add high-flow Mississippi River waters to Barataria Basin that would have higher suspended sediment concentrations; this would contribute substantial suspended sediment loads and elevated turbidity at and adjacent to the Project outflow and into the northern portion of the lower basin. Turbidity would also be increased by the flow of the outfall itself. During operations, turbidity adjacent to the Project outfall is anticipated to increase 50% to 200%, to a maximum of ≤375 g/m^3 TSS. TSS are expected to be elevated throughout the mid and lower basin with levels of TSS decreasing in magnitude with distance from the diversion (Figure 5.3.3-1). Along the barrier islands, seasonally elevated TSS is anticipated to increase over time under both the FWOP and FWP scenarios during winter months, up to 50% higher than the existing conditions (up to a maximum of ≤150 g/m^3 TSS)[3].

Increased turbidity can have consequences to plants and animals present, such as smothering of benthic vegetation and invertebrates, reducing DO levels, and displacing species from highly turbid areas. Increased turbidity may reduce light transmission into the water column, thereby reducing the water depths where SAV can thrive. In addition, aquatic vegetation may be buried or growing shoots may be covered with sediment reducing or preventing photosynthesis. Prolonged exposure to these effects may make habitat unsuitable for vegetation. Increased turbidity and sedimentation can affect normal fish behaviors (including ability to feed, move and/or shelter). Fish can experience injury from sediment abrasion on gill surfaces, and highly turbid waters may diminish the ability of fish to detect prey or predators.

3 This report uses both NTU and TSS as measures of turbid water. NTU measurements are used as a surrogate for TSS, which is the characteristic of turbid water that has the most impact on fish and fish habitat. NTU can be measured in the field and measures the light-scattering characteristics of turbid water while TSS is a direct measurement of suspended solids in the water, which is measured in a laboratory. The relationship between NTU and TSS is dependent on the composition of the suspended solids. Conversions of NTU to TSS are not possible at the scale of Barataria Basin because sediment composition is site specific and that sediment composition determines the relationship between NTU and TSS. Both measures of turbid water respond in the same direction with an unknown scalar relationship between NTU and TSS.

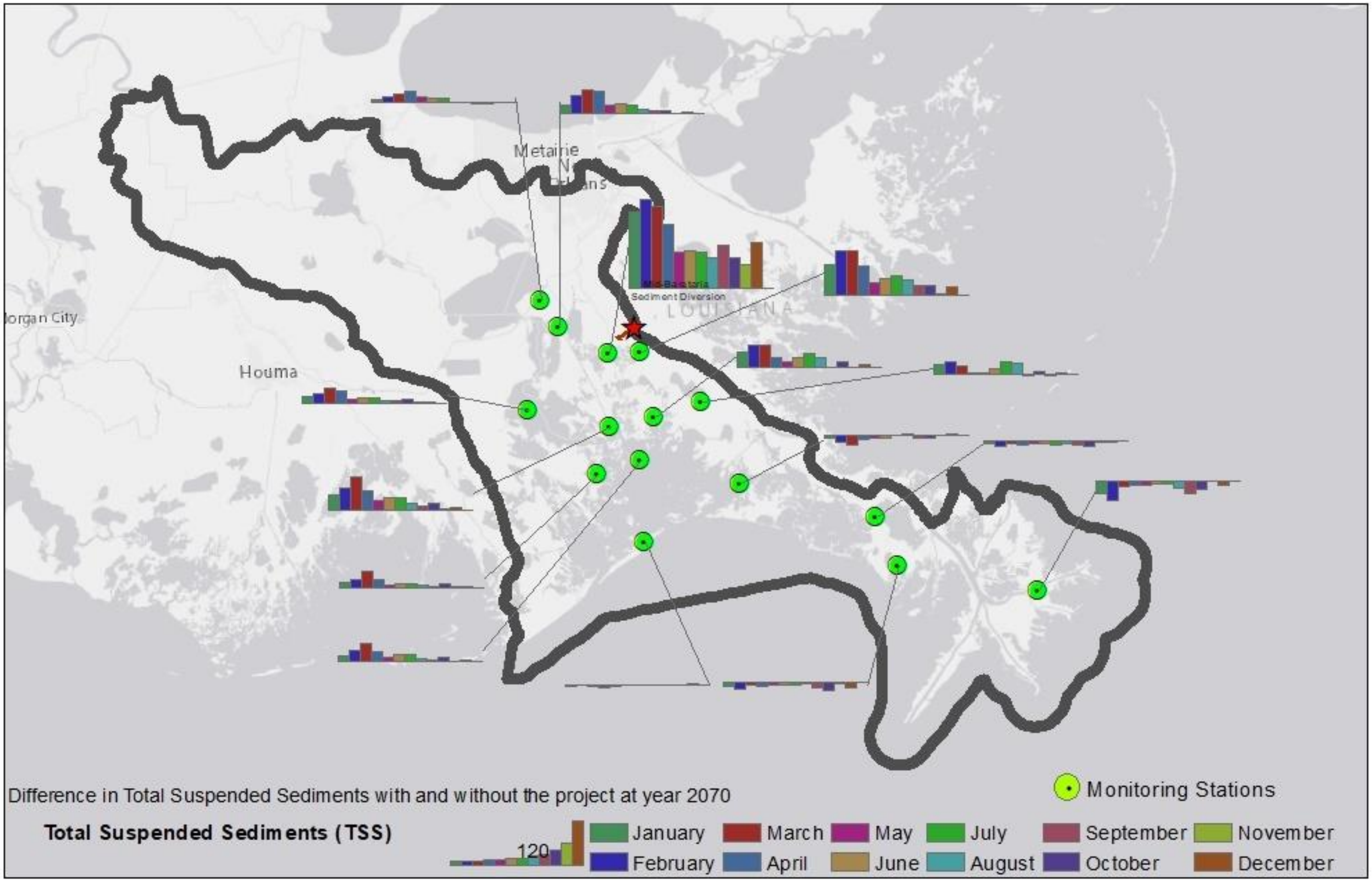


Figure 5.3.3-1. Suspended Sediment over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

5.3.4 Project Effects on Sediment Transport and Wetland Creation

Project Construction

The construction of the diversion structures and access dredging would generate large quantities of excavated earthen material from upland and aquatic sources. Where material is deemed unsuitable for construction of the Conveyance Channel levees and the Mississippi River levee system, it would be used to directly support marsh creation or restoration. Two sites have been identified for restoration that are near the outfall in Barataria Basin. These sites are the West BU Area, due west of the outfall, and the East BU Area, southeast of the outfall (near Myrtle Grove Marina).

Project Operation

Land loss is an ongoing major area of concern in Barataria Basin, with a cumulative loss of 1,120 km^2 from 1932 to 2016 (Couvillion et al. 2017). In response to SLR and other factors (e.g., storms), land loss across basin shorelines and marshes is anticipated to continue, with models predicting 84% loss of land over the next 50 years in Barataria Basin in a FWOP scenario. Submergence of marsh habitats would result in the loss of marsh habitat functions throughout the basin. The analysis of effects of the Project below include the influence of the predicted conditions in the FWOP trajectories and trends, as they would continue to exert their influence.

The purpose of the proposed Project is to restore deltaic processes in the Barataria Basin. By causing sediment deposition, areas that are currently or would otherwise become open water may become or remain wetland/marsh habitats. The proposed Project is anticipated to contribute sediment from the Mississippi River that would add up to 17,259 acres of land above future predicted sea level to Barataria Basin by 2050 as compared to the FWOP[4]. As sea levels continue to rise over time, by 2070, the Project is anticipated to have added up to 13,400 acres of land to Barataria Basin as compared to the FWOP. All land building would occur in the vicinity of the Project outfall, isolated to the eastern-mid region of the action area, extending both north and south, as well as slightly westward of the Project outfall. Diversion of flow and sediment into Barataria Basin will decrease sediment deposits at the Birdfoot Delta. Land loss in the Birdfoot Delta is anticipated to be up to 3,000 acres less due to the proposed Project by 2070 (see Figure 5.3.4-1 below). However, landloss due to the project in the Birdfoot Delta would not occur until after 2050, and overall the project would contribute up to 10,400 overall in Barataria Basin and the Birdfoot Delta combined.

4 TWI reports that Delft 3D hysteresis model is likely to overestimate land building by approximately 5%.

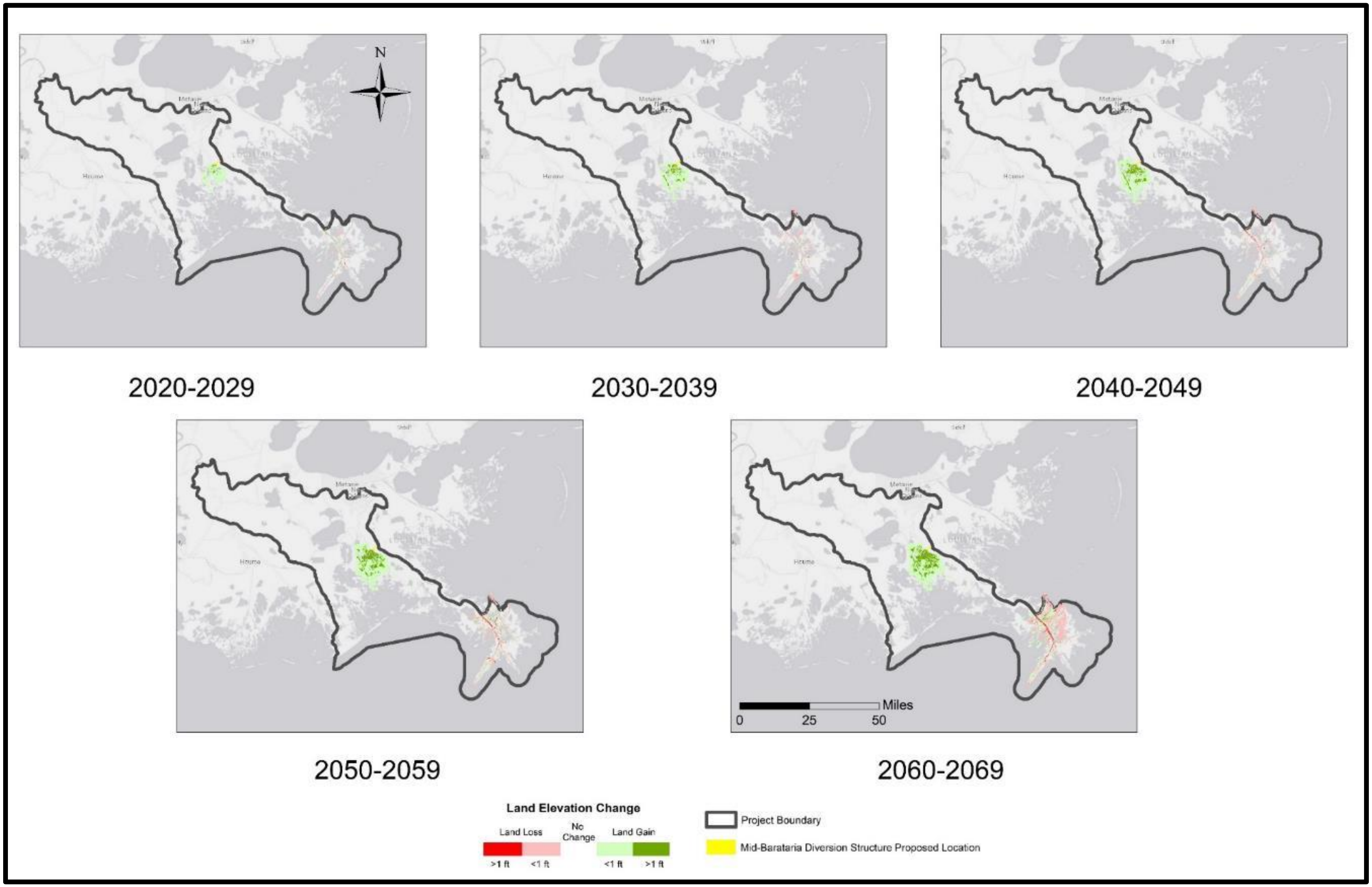


Figure 5.3.4-1. Land Building over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

5.3.5 Project Effects on Water Quality: Nutrients and Dissolved Oxygen

Project Construction

There would be no Project effects on nutrients due to construction activities. There may be minor and temporary Project effects on DO (decreased DO) in areas where construction activities increase turbidity and suspended sediments (see Section 5.3.3 above).

Project Operation

The proposed Project would contribute nutrients from the Mississippi River to the Barataria Basin. There are indications that the Barataria Basin may be nutrient limited in areas and/or at certain times of year, suppressing aquatic plant growth (Turner 2017). The addition of nutrients may support primary productivity and lead to decreases in DO resources if eutrophic conditions occur. However, the Barataria Basin are shallow and well mixed, allowing surface waters rich in DO to mix throughout the water column; this limits the risk of the occurrence of low DO conditions.

<u>Nutrients</u>

The Project's addition of nutrient-rich Mississippi River waters into the mid-basin would seasonally influence nutrient concentrations basin-wide. During spring and summer months, when the diversion is operating, total phosphorus (total P) and total nitrogen (total N) are predicted to be elevated immediately adjacent to the diversion outfall (refer to Figures 5.3.5-1 and 6.3.5-2).

Total P concentrations are predicted to be elevated both immediately adjacent to the outfall (maximum 0.18 ml/l higher than FWOP) and slightly south of the diversion outfall (a maximum elevation of 0.16 ml/l compared to FWOP) during months when the diversion is operating. At the outfall, elevated levels of total N would be sustained throughout the year, though at lower levels than when the diversion is operating (0.05 mg/L to 0.12 kg/l). Total P concentrations are anticipated to decrease with distance south of the outfall until undetectable in the mid-lower basin. The Project is not anticipated to influence total P concentrations north of the Project outfall.

Total N concentrations consist of both nitrate (NO_3) and ammonium (NH_4) concentrations. Changes in total N due to the Project are predicted to be predominantly driven by changes in nitrate concentrations, but all forms of nitrogen are anticipated to become elevated immediately adjacent to the Project outfall. Ammonium concentrations are anticipated to be slightly elevated (a maximum elevation of 0.16 mg/L in 2030-2039 compared to FWOP, which is between 0.00 and 0.01 mg/L) at the Project outfall and immediately south, only during months where the diversion is operating. At the outfall, elevated nitrate concentrations are predicted to be sustained throughout the year (a maximum elevation of 1.38 mg/L in 2030-2039 compared to

FWOP). Southward of the outfall, nitrate concentrations are anticipated to only be elevated during months when the diversion is operating, and to decrease with distance from the outfall until undetectable in the mid-lower basin. The Project is not anticipated to influence ammonium or nitrate concentrations north of the Project outfall.

Changes to aquatic concentrations of nitrogen and phosphorus can have substantial direct and indirect effects on plant growth, DO concentrations, water clarity, and sedimentation rates. Nitrogen's primary role in organisms is protein and DNA synthesis; as well as in photosynthesis. Phosphorus is critical for metabolic processes, which involve the transfer of energy (USEPA 2010). Most effects from nutrient changes are reflected in primary production as discussed below.

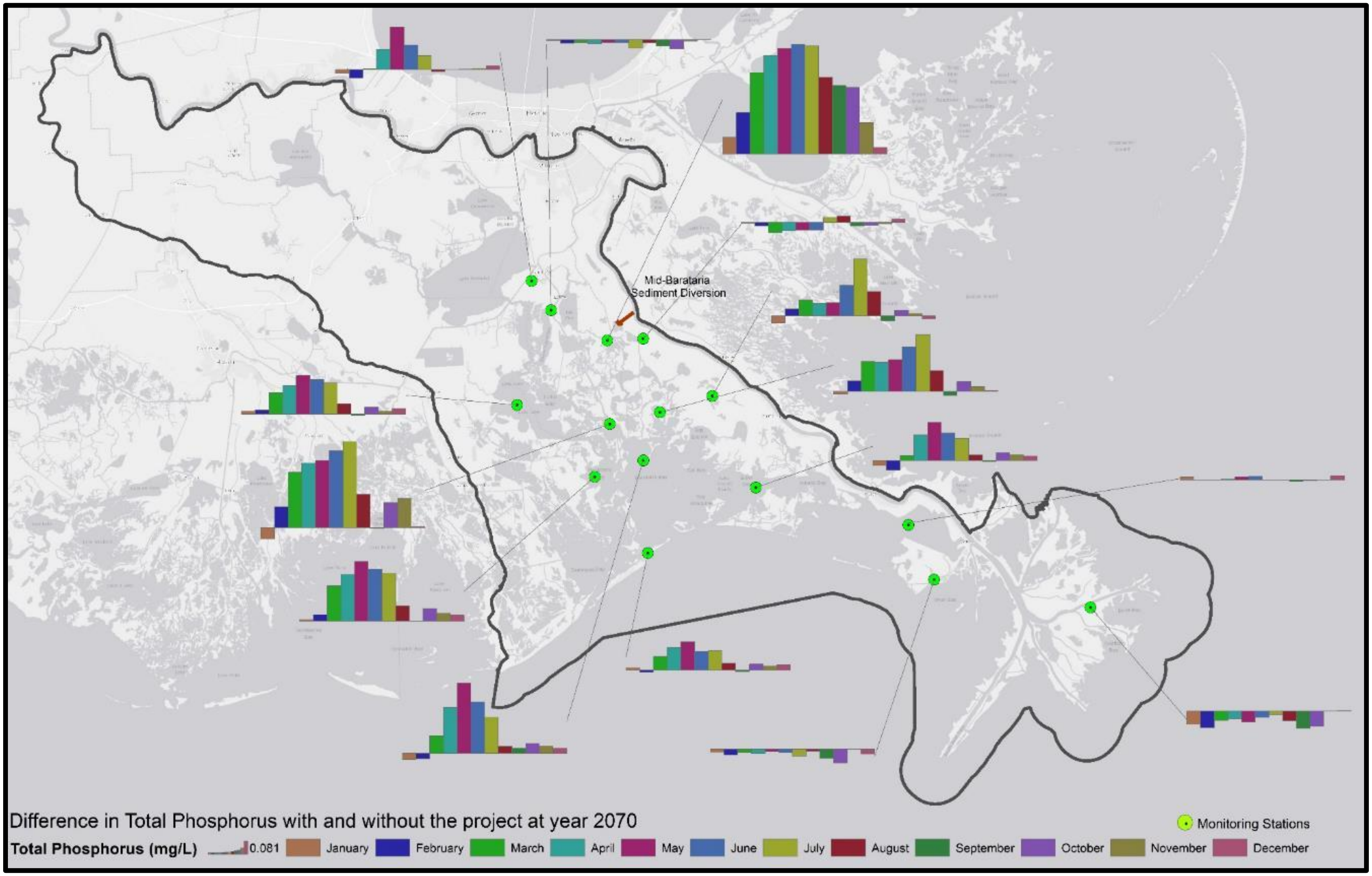


Figure 5.3.5-1. Total Phosphorus over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

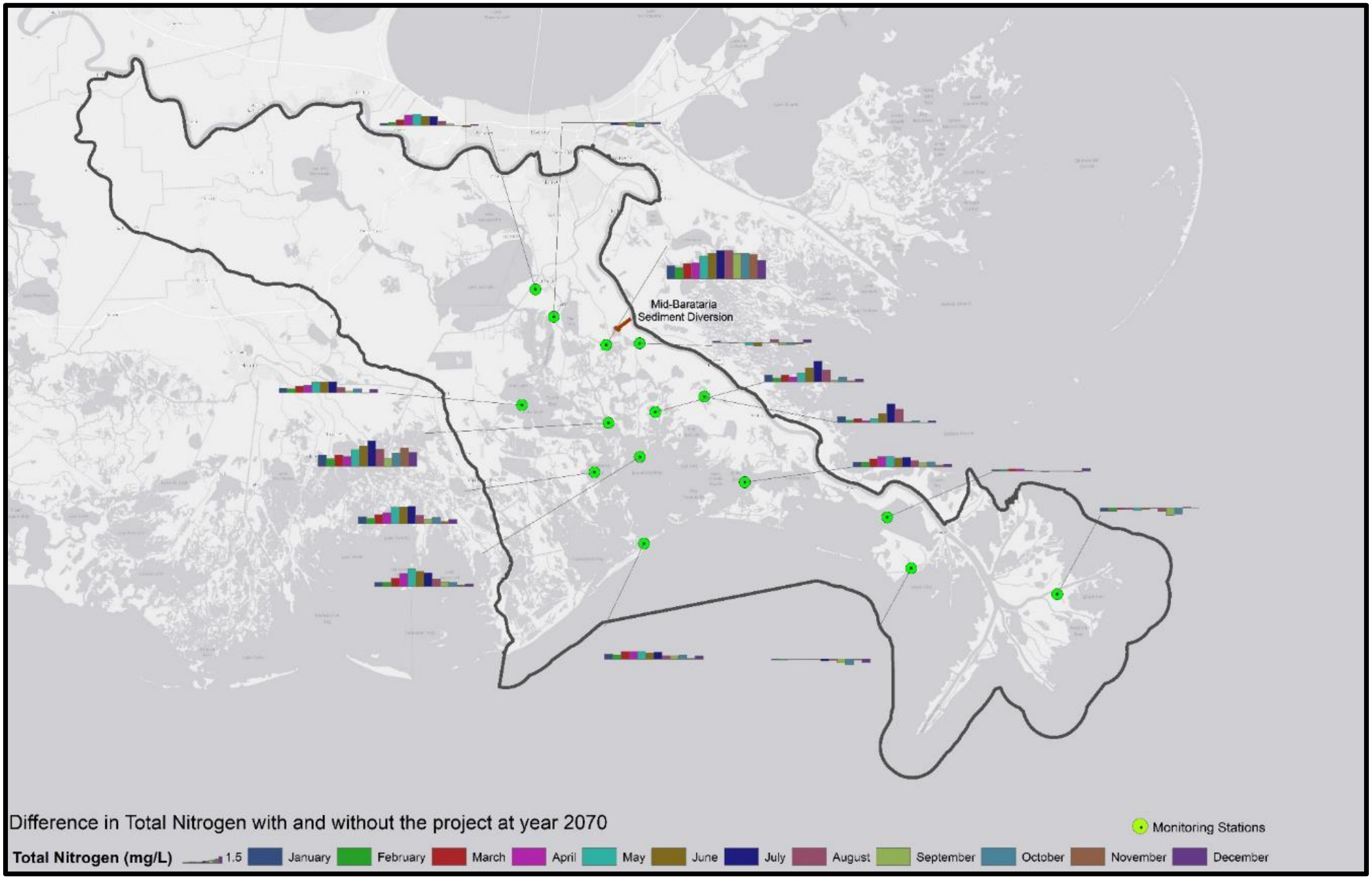


Figure 5.3.5-2. Total Nitrogen over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

Primary Production (Chlorophyll A)

The Project's addition of sediment and nutrient-rich Mississippi River waters into the mid-basin would seasonally influence chlorophyll A (Chl A) concentrations basin-wide (see Figure 5.3.5-3). During spring and summer months, when the diversion is operating, Chl A concentrations are anticipated to substantially decrease immediately adjacent to the diversion outfall (up to 66 μg/L below FWOP) due to high turbidity conditions limiting primary production (discussed above in Section 5.3.3). Expanding southward from the diversion outfall, early spring decreases in Chl A concentrations are anticipated in March, with more historical concentrations in April/May. Throughout the mid and into the central-lower basin, Chl A concentrations, in the form of phytoplankton blooms, are predicted to rise throughout the peak growing season's summer months of May to October (maximum of 74.37 μg/L above FWOP in 2040-2049) in response to the addition of nutrients from the diversion (discussed above).

Elevated summer phytoplankton blooms above FWOP conditions are predicted to be most dense within the southern-mid and lower basin regions, south of the outflow, beyond the greatest influence of turbidity. Chl A concentrations may be higher in the southern-lower basin in summer months due to localized decreases in mixing provided by the barrier islands (up to 61.2 μg/L above FWOP in 2030-2039). By 2070, slight increases in Chl A concentrations are anticipated to be seasonally detectable north of the diversion (up to 21.38 μg/L above FWOP). The Project is not anticipated to substantially influence Chl A concentrations in the Birdfoot Delta.

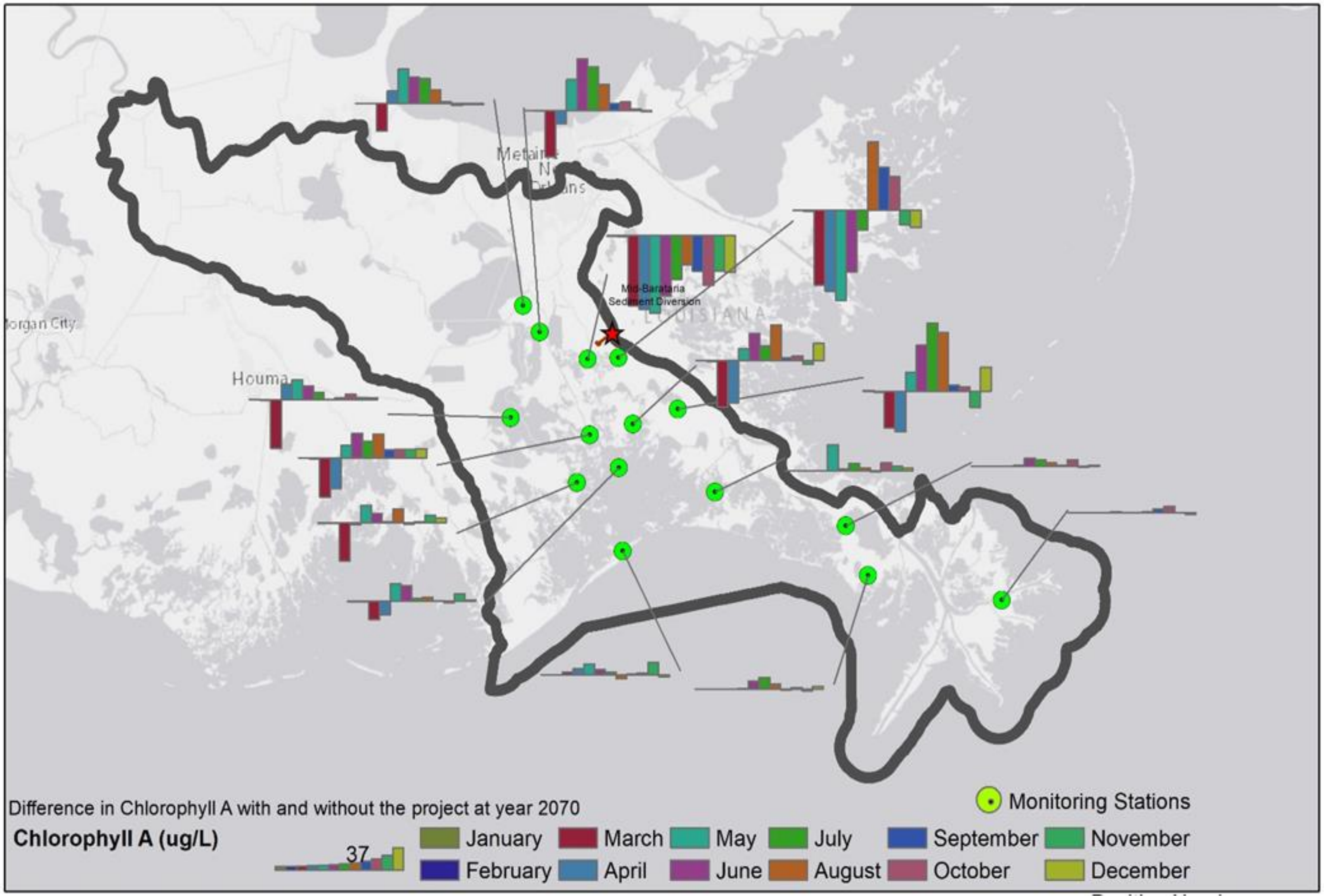


Figure 5.3.5-3. Chl A over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

Elevated phytoplankton blooms and primary production can only benefit aquatic systems to the threshold where they become eutrophic. Over-enrichment can lead to the development of low oxygen areas, harmful algal blooms, and loss of SAV and bottom habitat. Chl A changes due to nutrient enrichment in Barataria Bay are not anticipated to result in eutrophic conditions, and not substantially decrease DO concentrations due to strong mixing of the water column; however, phytoplankton and macrophyte communities may be locally and temporally altered.

Dissolved Oxygen

When operating, the proposed Project is anticipated to minimally decrease DO concentrations immediately adjacent to the outfall, correlated with high turbidity, nutrient enrichment, and increased primary productivity at certain times of year (discussed above in Section 5.3.5).

The Project is predicted to have very little effect on DO concentrations north of the diversion (see Figure 5.3.5-4 below). Though the Project is anticipated to have varied DO effects throughout the central and lower basin, minimally lowering DO throughout the basin on a seasonal basis, the lowest DO concentrations predicted are approximately 6.0 mg/L, within the tolerance range of most organisms.

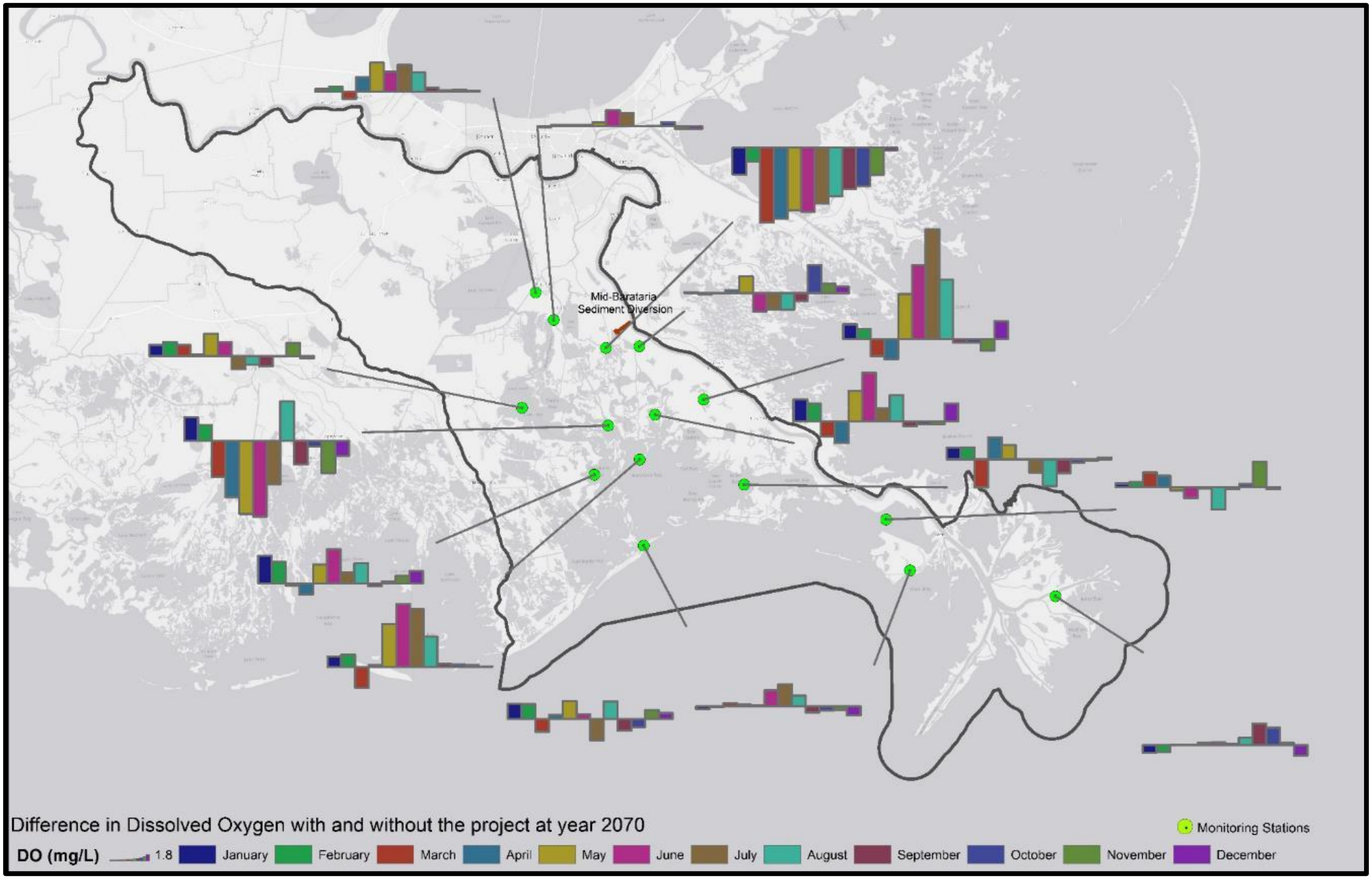


Figure 5.3.5-4. Dissolved Oxygen over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

5.3.6 Project Effects on Water Quality: Contaminants

Project Construction

Potential for spills or contamination events during Project construction would be avoided or minimized by adhering to BMPs (see Section 2.3.6 above). Although not anticipated, effects from accidental releases of contaminants during Project construction would likely follow the same dispersal patterns as nutrients described above in Section 5.3.5. Sediment-bound contaminants would be most likely to aggregate in the same dispersal pattern as described by the sediment transport in Section 5.3.4 above. Base levels of contaminants from the Mississippi River are not anticipated to be detrimental to species or habitats of Barataria Basin or the Birdfoot Delta.

Project Operation

As is the case in all aquatic environments, water diverted from the Mississippi River has the inherent potential to contain contaminants from sediments, upstream sources, or due to accidental releases into the river. During construction and operations, the Project would adhere to BMPs to reduce potential releases of contaminants into the river or basin (conservation measures are described in Section 2.3.6 above). Sediments from the Mississippi River assessed in 2017 were found to contain contaminants at levels under their respective water quality criteria (WQC) or water quality standard (WQS), or below NOAA's TEL screening values, with the exception of zinc. Sediment analysis concluded that Mississippi River sediments were free from COCs at concentrations that would result in detrimental effects on habitats if dredged or transported. As the proposed Project would intentionally divert sediments from the Mississippi River into the low-energy mid region of Barataria Basin, the contaminants within the sediments would likely not experience strong dilution effects (see Section 4.3.1 for more information on sediment quality within the proposed action area).

Aquatic contaminants from accidental releases in the Mississippi would likely follow the same dispersal patterns as nutrients described above in Section 5.3.5. Sediment-bound contaminants would be most likely to aggregate in the same dispersal pattern as described by the sediment transport in Section 5.3.4 above. Base levels of contaminants from the Mississippi River are not anticipated to be detrimental to species or habitats of Barataria Basin or the Birdfoot Delta. Mississippi River sediments have been evaluated and approved for marsh creation by USFWS, NOAA, USEPA, and the USACE.

5.3.7 Project Effects on Water Flow within Barataria Basin

Increased freshwater flows into the estuaries are expected to more strongly influence currents and circulation in the basin, with effects diminishing with distance away from the outfall. Differences in flow conditions for each of the alternatives is shown in Figure 5.3.7-1.

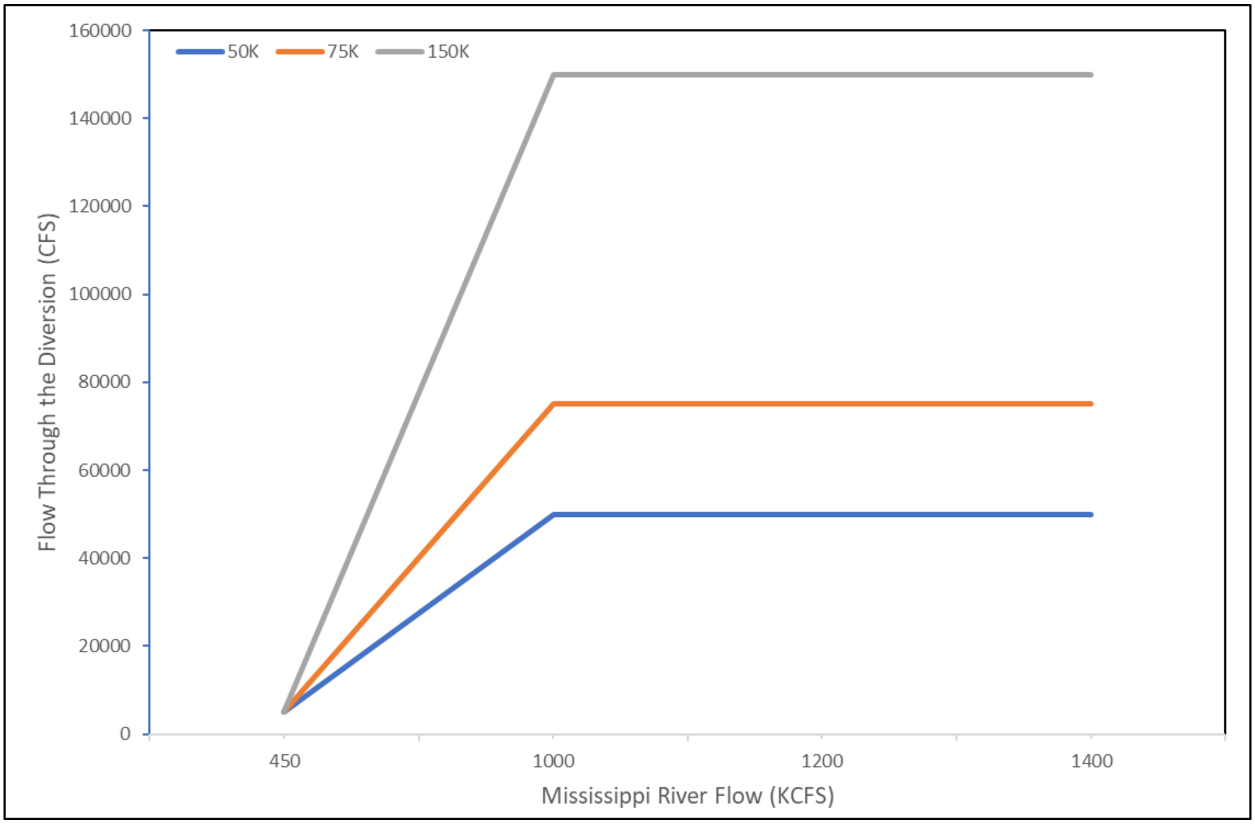


Figure 5.3.7-1. Comparison of Flow through Diversion at Various Mississippi River Flow Rates

Development of the MBSD will create a new net outflow of freshwater from the outfall to the barrier islands. This outflow could affect movement of water masses and passive organisms that rely on the movement of water. A detailed comparison of flow conditions was undertaken using the Delft 3D model outputs (Sadid et al. 2018). The comparison focused on locations where potential effects to currents from diversion operations would be most evident, namely channels and passes where existing flow velocities are the highest (Figure 5.3.7-2). Velocity magnitude was determined for peak flood, slack, and peak ebb tides in May (diversion operating) and October (diversion at base flow) for both FWP and FWOP scenarios. Time series plots showing velocity magnitude and direction were also developed for FWP and FWOP scenarios at the main channels and passes.

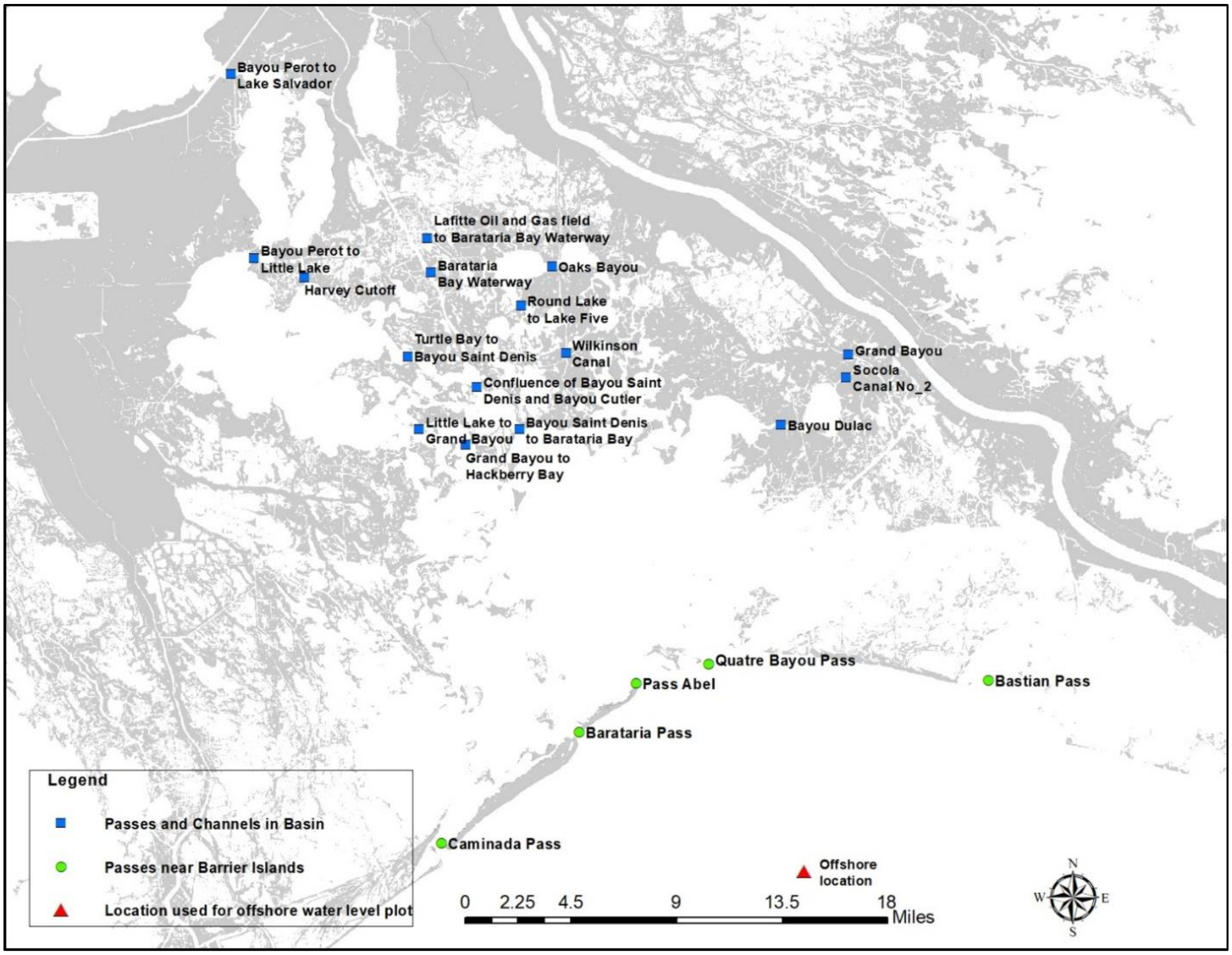


Figure 5.3.7-2. Channels and Passes Evaluated for Changes to Currents

The following categories of results were observed from analysis of currents in the Barataria Basin passes and channels:

- The passes and channels closest to the outfall (Oakes Bayou, Barataria Waterway, Round Lake to Lake Five, Lafitte Oil and Gas Field to Barataria Waterway) generally are modelled to have unidirectional outflow while the diversion is operating at mid flow levels and higher (>20k cfs) (Figure 5.3.7-3). Outflow is generally the dominant signal and the model results do not show much return flow except during larger tidal exchanges. During diversion base flow of 5k cfs, the tidal signal returns and flow vectors go in both directions and at similar velocities between FWP and FWOP scenarios (Figure 5.3.7-4).
- In the interior passes and channels slightly farther from the outfall (Wilkinson Canal, Turtle Bay to Bayou Saint Denis, Confluence of Bayou Saint Denis and Bayou Cutler), tidal flow is evident in both directions, generally similar between FWP and FWOP (Figure 5.3.7-5). However, during smaller tides, flooding tides are sometimes suppressed by outflow. Generally, there is a decrease in velocity of the flood tide in the FWP versus

FWOP scenario while the diversion is operating. Under base flow conditions, flow direction and magnitude are similar in FWP and FWOP scenarios (Figure 5.3.7-6).

- The interior passes farther away from the outfall (Little Lake to Grand Bayou, Bayou Saint Denis to Barataria Bay, Grand Bayou to Hackberry Bay) have flow directions similar between FWP and FWOP during both smaller and larger tides but also show slightly reduced velocities for flooding tides (Figure 5.3.7-7). Under base flow conditions, flow direction and magnitude are similar in FWP and FWOP scenarios (Figure 5.3.7-8).
- The passes and channels at the barrier islands (Barataria Pass, Bastian Pass, Caminada Pass, Pass Abel, and Quatre Bayou Pass) have flow directions and magnitudes that are similar between FWP and FWOP scenarios under all conditions from baseflow of 5k to 75k full diversion operation (Figure 5.3.7-9).

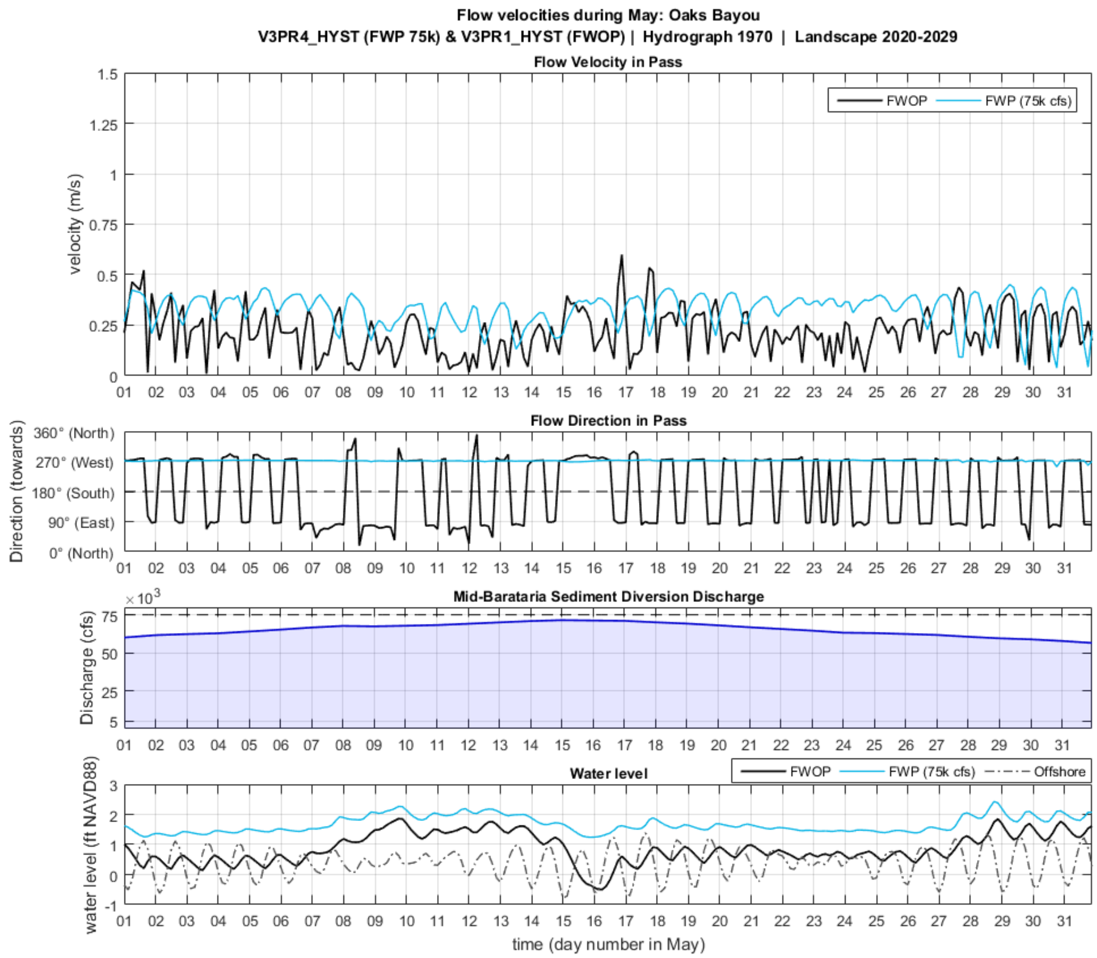


Figure 5.3.7-3. Representative Channel (Oakes Bayou) Near Diversion Outfall with Diversion Operating at ~30,000 cfs

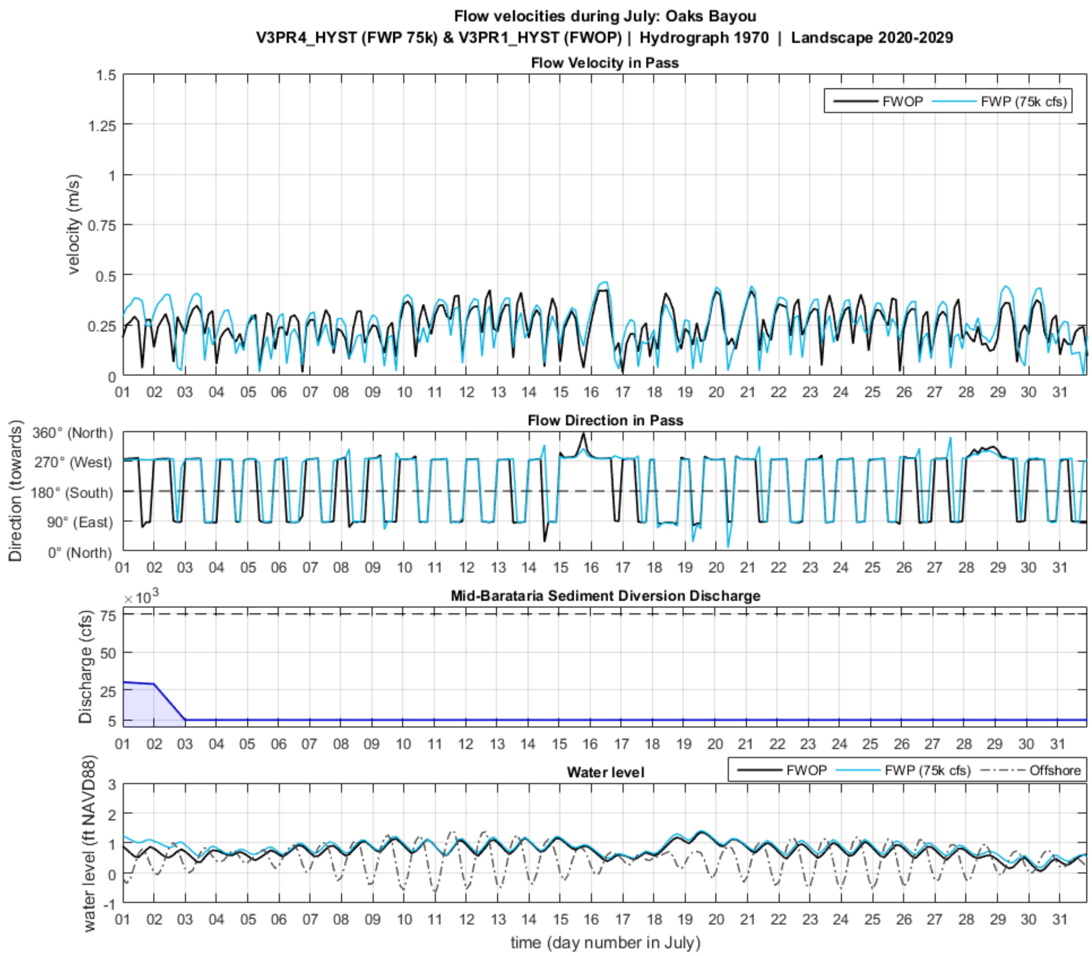


Figure 5.3.7-4. Representative Channel (Oakes Bayou) Near Diversion Outfall with Diversion Operating at Baseflow of 5,000 cfs

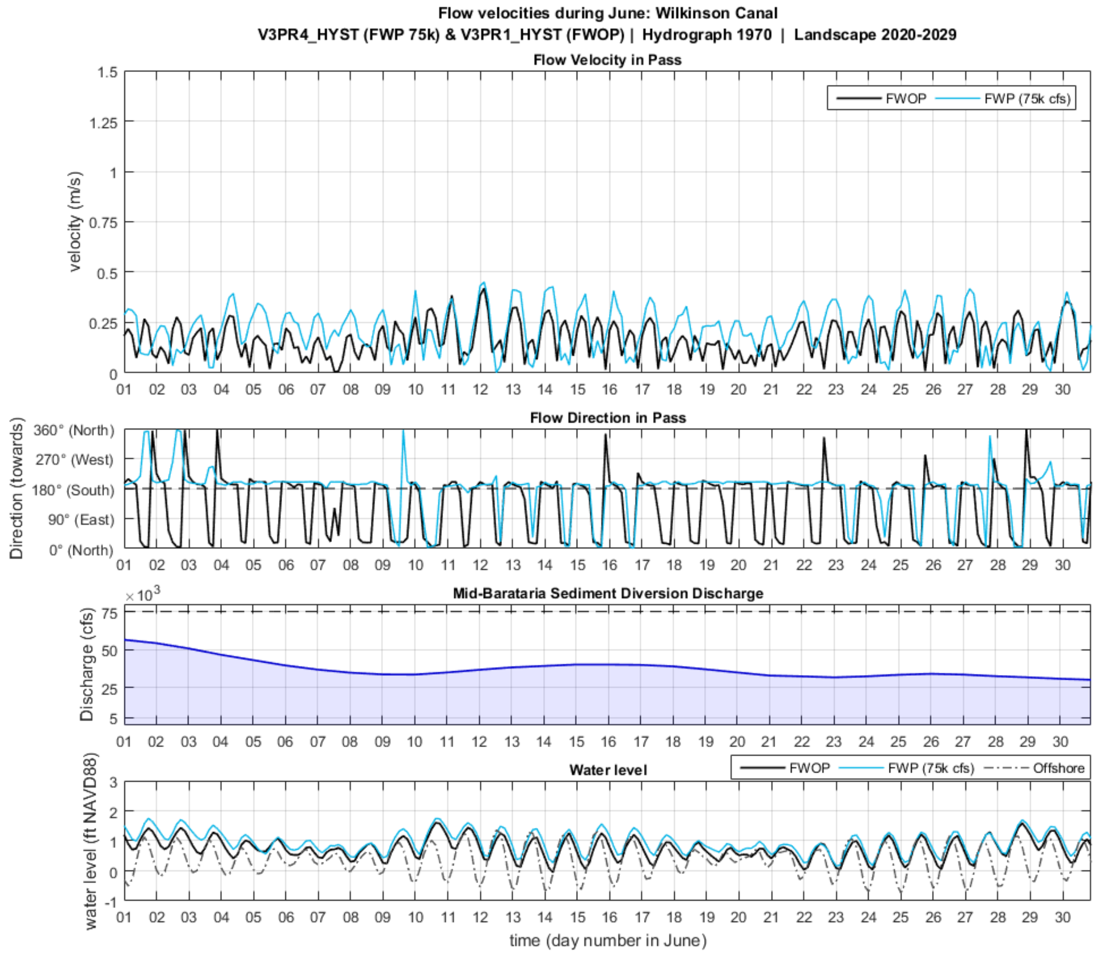


Figure 5.3.7-5. Representative Channel (Wilkinson Canal) Mid-Distance from Diversion Outfall with Diversion Operating between 25,000 and 50,000 cfs

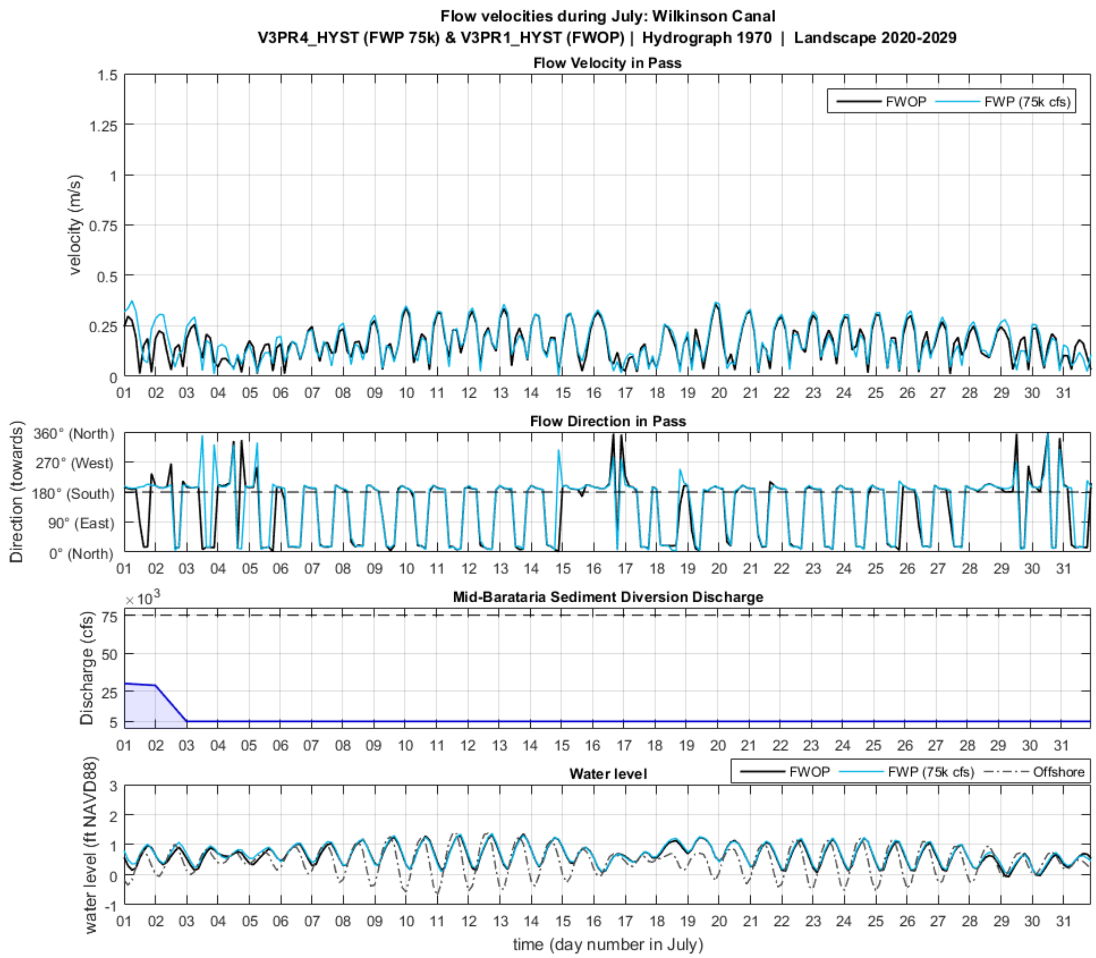


Figure 5.3.7-6. Representative Channel (Wilkinson Canal) Mid-Distance from Diversion Outfall with Diversion Operating at Baseflow of 5,000 cfs

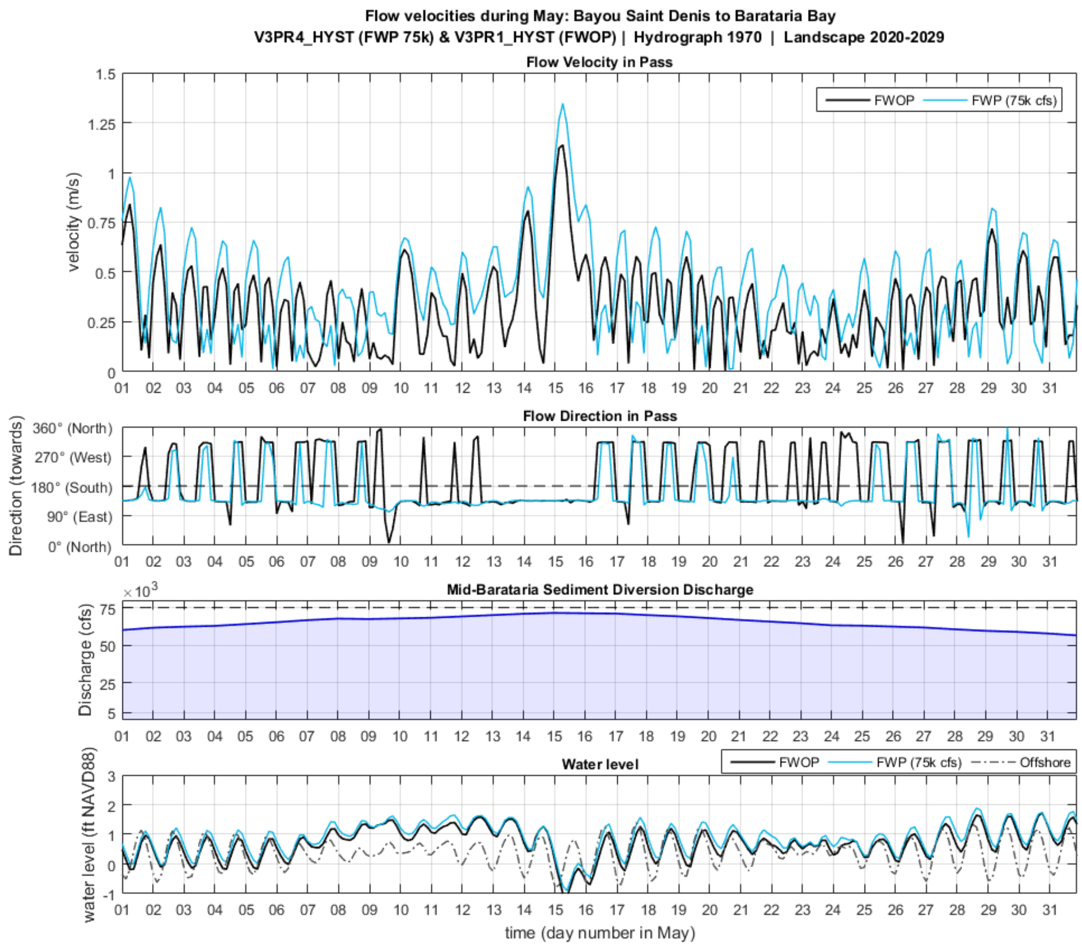


Figure 5.3.7-7. Representative Channel (Bayou Saint Denis to Barataria Bay) Distant from Diversion Outfall with Diversion Operating at 50,000 to 75,000 cfs

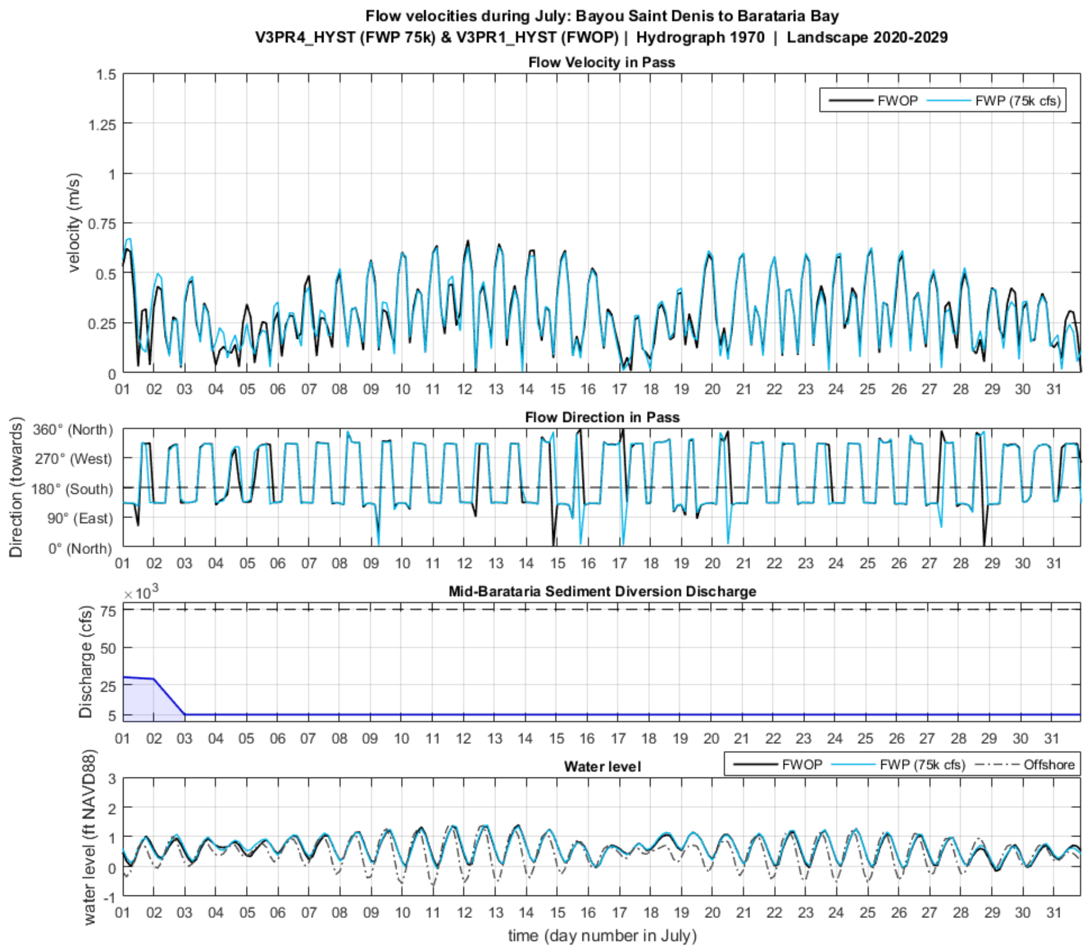


Figure 5.3.7-8. Representative Channel (Bayou Saint Denis to Barataria Bay) Distant from Diversion Outfall with Diversion Operating at Baseflow of 5,000 cfs

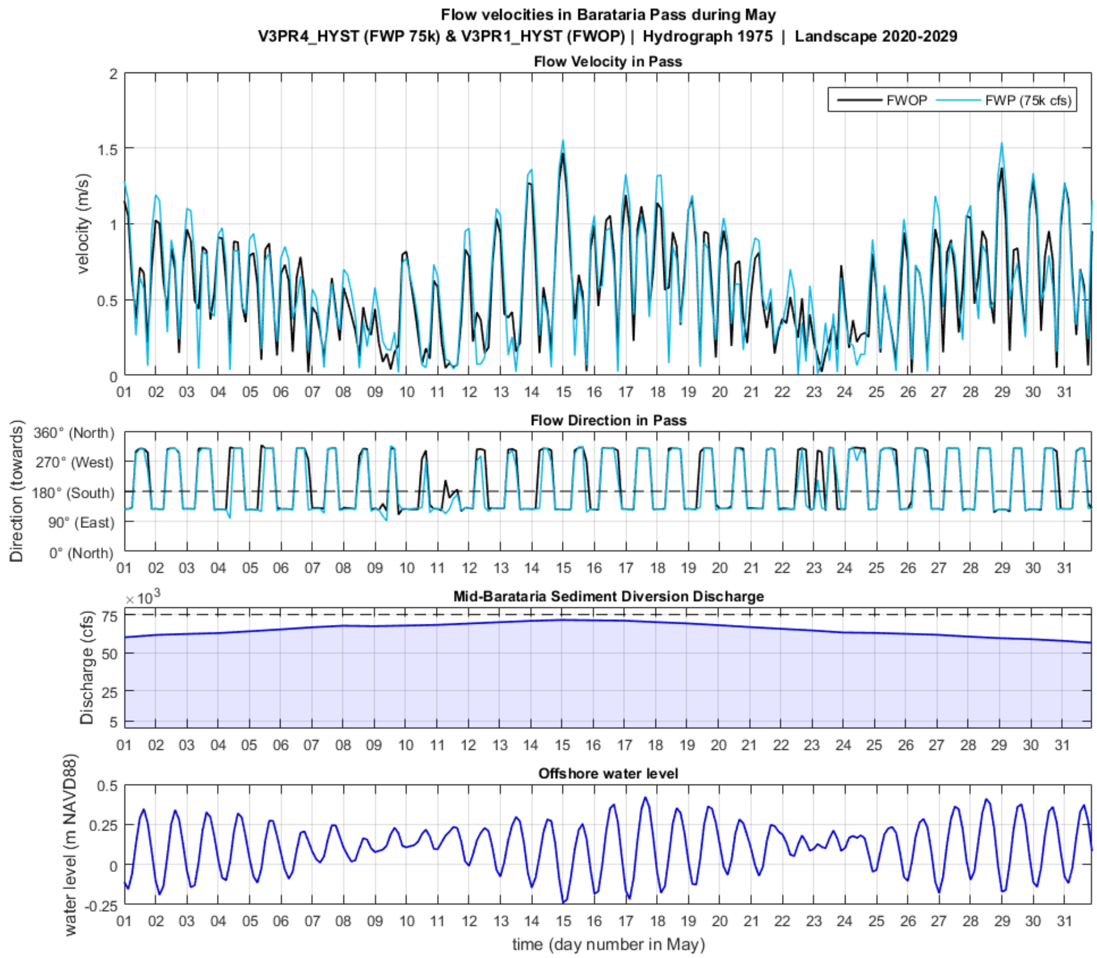


Figure 5.3.7-9. Representative Barrier Island Pass (Barataria Pass with Diversion Operating at 50,000 to 75,000 cfs

5.3.8 Project Effects on Sound

Project Construction

Construction of the proposed Project would require pile driving, dredging, barge operations, and other in-water activities that generate underwater sound. These activities would be added to baseline underwater sound conditions in the Mississippi River and Barataria Basin and have the potential to affect habitat use, generate behavioral modifications or, in extreme cases, result in mortality.

Ambient underwater sound levels represent noise from natural sources, such as wind-driven waves, storms, fish, tidal currents, and vocalizing marine mammals. When anthropogenic sources are added to ambient noises sources, underwater noise levels increase. The extent and duration of increase is variable in time and space and dependent upon the individual and cumulative anthropogenic source types. In the Mississippi River, anthropogenic underwater sound may be generated by smaller fishing and recreational vessels, as well as larger commercial vessels (for example, oil tankers and container ships), pile-driving, and dredging. In the Barataria Basin, sources of anthropogenic underwater sound include commercial fishing and recreational vessels, dredging, pile-driving, and oil and gas production.

As with airborne noise, ambient underwater noise is variable over time due to changes in the intensity and abundance of noise sources. Biological sounds associated with a host of mammals, fishes, and invertebrates can generate broadband noise in the frequency range of about 10 to 10,000 kiloHertz (kHz) (Discovery of Sound in the Sea [DOSITS] 2017). Ambient sound in the mid-frequency range of 500–10,000 Hertz (Hz) is primarily due to sound from breaking waves; the intensity of sound in this frequency range increases with wind speed (DOSITS 2017). Higher-frequency sounds are primarily generated by thermal noise, which is the sound of the random movement of water molecules as a result of water temperature increases (DOSITS 2017). Most underwater sound in the 20–500 Hz range is due to distant shipping, rather than natural sources; vessel traffic generates low-frequency sounds that can travel considerable distances (DOSITS 2017).

Sound measurements are reported as decibel readings, relative to a reference value of 1 μPa, which is a measure of absolute pressure. Decibels have a logarithmic relationship to μPa. Sound energy is commonly reported as sound pressure levels (SPL), which is the average sound intensity for a single sound-producing event. SPL is commonly reported as either peak SPL (dB_{peak}) or as root mean square (RMS) pressure level (dB_{RMS}). Peak SPL is the ratio of the absolute maximum sound pressure to a pressure of 1 μPa for a single sound-producing event.

Measurements of baseline ambient underwater sound in the action area are not available. However, NMFS recognizes the sound level for "effective quiet" or the safe exposure level at which risks for impacts on marine turtles and fish are as low as 150 dB re 1 μPa SEL (NMFS 2016b). Sound below the 150-dB level of effective quiet would not harass marine turtles and fish. Further, as discussed in Section 5.4, NMFS has established thresholds for physical and behavioral effects of underwater noise on sea turtles, fish, and marine mammals due to impulsive (for example, impact pile-driving, seismic airguns) and non-impulsive (for example, vibratory pile-driving, sonar) sound sources.

The Project proposes 5 types of in-water construction and maintenance activities that would have the potential to increase underwater sound levels: dredging, deposition of fill (gravels and rocks), impact pile driving, vibratory pile driving, and vessel operations (see Table 5.3.8-1).

Project construction would also involve temporary airborne sound effects, ranging from minor to moderate. The loudest airborne sound would come from riverside impact pile driving. A comprehensive list of all aquatic construction equipment and associated underwater noise production is listed below in Table 5.3.8-1. Existing sources of noise in the action area typically include anthropogenic noise, such as vessels (including airboats and ships on the Mississippi River) in open water areas, and natural noises such as wildlife vocalizations.

Table 5.3.8-1. Sources and Values of Ambient and Project-Related Underwater Noise

Equipment or Activity	Ambient or Project-Related Noise	Noise Level (L_{max})	Data Sources
Mississippi River			
Mississippi River Ambient	Ambient (mean)	126 dB_{PEAK} @ 0 meters	2
Impact Pile Driving at Riverside Trestle, Cushioned (30-inch or 36-inch Steel Pile)	Project Construction	208 dB_{PEAK} @ 10 meters	1, 6
Impact Pile Driving at Coffer Dam (H/I-shaped Steel King Piles)	Project Construction	190 dB_{PEAK} @ 10 meters	1, 2, 6
Vibratory Pile Driving at Coffer Dam (24-inch Steel Sheet Pile)	Project Construction	165 dB_{RMS} @ 10 meters	1, 2, 6
Cutterhead Dredging*	Project Construction & Maintenance	172-185 dB_{RMS} @ 1 meter	7, 8
Vessel Operations	Project Construction & Maintenance	175 dB_{RMS} @ 1 meter	3, 5, 8
Barataria Basin			
Barataria Bay Ambient	Ambient (mean)	126 dB_{peak} @ 0 meters	2
Impact Pile Driving at Boat Pier (12-inch Timber Piles)	Project Construction	180 dB_{PEAK} @ 10 meters	2, 6
Vibratory Pile Driving at Outfall (24-inch Steel Sheet Pile)	Project Construction	165 dB_{RMS} @ 10 meters	1, 2, 6
Vibratory Pile Driving at Coffer Dam (H-shaped Steel Piles)	Project Construction	137 dB_{RMS} @ 10 meters	1, 2, 6
Pressed Installation of Navigational Markers (12-inch Timber Piles)	Project Construction	negligible	--
Placement of Fill in BU Areas	Project Construction	142 dB_{PEAK} @ 40 meters	9
Cutterhead Dredging*	Project Construction & Maintenance	172-185 dB_{RMS} @ 1 meter	7, 8
Vessel Operations	Project Construction & Maintenance	175 dB_{RMS} @ 1 meter	3, 5, 8

dBA = The A-weighted decibel scale; dB_{RMS} = dBA root mean square pressure level
PSD = (dB re1 $\mu Pa^2/Hz$)
* = The specific method of dredging has not yet been determined. Therefore, cutterhead dredge activities are presented as the most conservative option and the estimated noise range identified as appropriate for the Project area habitat.
Data Sources: 1. Illingworth and Rodkin 2007; 2. WSDOT 2019, 3. de Jong et al. 2010, 4. Dickerson et al. 2001, 5. Reine et al. 2012, 6. CalTrans 2015, 7. Blue Planet Marine 2013 (as cited in Jones et al. 2015) 8. CEDA 2011, 9. Genesis 2011

The main underwater sound increase in the Mississippi River would be from in-water pile driving activities and dredging during Project construction. Project construction both in the Mississippi River and Barataria Basin requires the use of in-water pile driving activities. Both impact and vibratory pile driving methods would be used to install piling and sheet piling as part of the construction of the Project's foundation systems, cofferdam, and trestle on the river side, and outfall construction on the basin side. Dredging is planned for Project construction in the basin.

Where possible, the Project intends to use vibratory pile driving hammers to place in-water piles. The in-water sound generated from vibratory pile driving is generally 10 dBA to 20 dBA lower than impact pile driving (WSDOT 2019). Because vibratory hammers tend to disperse the energy required to drive the pile over time, this method of pile driving is generally considered less harmful to aquatic organisms. A riverine project that installed steel piles in a California river using vibratory pile driving resulted in sound pressure levels below the ambient noise created by the current (Reyff 2006 *as cited in* WSDOT 2019).

Vibratory pile driving, while quieter than impact pile driving, can still result in a cumulative sound energy effect. Hastings and Popper (2005) describe how sound exposure level (SEL) is a means of recording and reporting such cumulative in-water sound and is based on the cumulative sum of the squares of the sound pressure values in a sound wave. This squaring process gives the positive and negative pressure values equivalent contributions to the cumulative energy, and it is always a positive value. An SEL is the constant sound level over 1 second that has the same amount of acoustic energy as the original sound.

The Project proposes to use vibratory methods to remove sheet piling and temporary piling where possible. If piling cannot be removed, it will be cut off near the waterline. Vibratory piling removal is assumed to generate similar levels of underwater sound to installation activities.

Aquatic organisms present in the area during construction may be subjected to elevated sound levels from both impact and vibratory pile driving activities.

The modeled distances and areas also do not account for potential attenuation measures such as bubble curtains, which the proposed Project may on using on the basin-side. Bubble curtains are not a feasible noise attenuation method for activities in high current areas such as the Mississippi River. Noise abatement BMPs can reduce underwater SPLs and would reduce the distance that underwater noise would travel. The primary BMP for reducing underwater noise effects will be the use of vibratory hammers in Barataria Basin and wherever practical within the Project construction footprint. Timber piling are proposed for the boat pier near the outfall in Barataria Basin.

Dredging would occur during both construction and maintenance activities on the basin side of the Project, as would placement of fill in beneficial use areas. Vessel (e.g., barge) operations would occur both river-side and basin-side in support of construction activities and could produce in-water noise disturbance (i.e., exceed 187 dB_{RMS}). However, vessel operation is likely to result in noise levels that are less than the injury effects threshold for fish (i.e., 206 dB_{PEAK}) and composed of a substantially different sound signature (e.g., distribution of sound energy levels across variable frequencies) compared to pile driving activities.

Impact Pile Driving Calculations

Based on underwater noise calculations conducted per NOAA 2018, impact pile driving of 30- to 36-inch pilings used in the Mississippi River are expected to produce underwater sound levels of up to 208 dB_{peak}, 190 dB_{RMS}, and 180 dB SEL.

Timber pilings are proposed for the Barataria Basin. Impact pile driving of 12-inch-diameter timber pilings are anticipated to produce underwater sound levels of up to 180 dB_{peak}, 170 dB_{RMS}, and 160 dB SEL.

Underwater sound attenuation was estimated based on the practical spreading loss model shown in Equation 1.

Equation 1 R1 (in meters) = R2 (in meters)*$10^{(TL/15)}$

R1 = 10*$10^{((TL)/15)}$

Where:

R1 = range in meters of the SPL

R2 = distance from the sources of the initial measurement

TL = transmission loss

A total of 132 30- to 36-inch pilings are proposed to be installed in the Mississippi River over a duration of 1 to 2 months. For the Mississippi River construction activities, the noise model includes assumptions for the maximum number of impact pile strikes during a pile driving day which has been estimated at 10,000 or fewer for this Project. According to the spreading loss model, construction-related underwater noise exceed effective quiet noise conditions (150 dB) within approximately 15,230 feet of construction activities.

The onset for potential injury for fish is underwater sounds exceeding 183 dB SEL; this potential injury distance is approximately 3,281 feet from the piling installations resulting in a maximum area of potential injury to small fish from impact pile driving of approximately 362 acres. Behavioral effects may extend to approximately 15,230 feet and affect approximately 1,285 acres of Mississippi River area. Because sound waves travel directionally, the affected areas are limited to portions of the River with line-of-site to the sound sources. Impact pile driving is expected to occur for 1-2 months in the vicinity of the Mississippi River trestle.

In the Barataria Basin vibratory pile driving is planned for most pile installations; however, 30 12-inch timber piles are proposed to be installed using impact pile driving to support a 142-foot long pier associated with a boat ramp near the outfall. Impact pile driving in the Barataria Basin is predicted to last approximately 5 workdays.

For the Barataria Basin construction activities, the assumption for a maximum number of pile strikes per day is 200 or fewer for this project. Noise calculations suggest underwater noise would exceed behavioral thresholds for fish within approximately 705 feet of pile installation, resulting in an affected area of approximately 27.6 acres for impact pile driving in the Barataria Basin habitats. The onset for potential injury for fish is 183 dB cumulative SEL. Pile driving calculations predict a potential injury distance for fish of approximately 10 feet from piling installations (assuming 20 strikes per pile and 200 total strikes per day) and affecting approximately 0.25 acres.

Vibratory Pile Driving Calculations

Based on underwater noise calculations conducted per NOAA (2018), vibratory driving is expected to produce underwater sound levels of 182 dB_{peak}, 165 dB_{RMS}, and 165 dB SEL.

Underwater sound attenuation was estimated based on the practical spreading loss model shown in Equation 1 (see above).

According to the spreading loss model, construction-related underwater noise would attenuate to ambient background noise conditions within 330 feet, which would result in an area of elevated sound of up to 804 acres for vibratory pile driving in Barataria Basin and 76 acres for vibratory pile driving in the Mississippi River. No fish injury thresholds have been identified for vibratory pile driving, and there are no impacts expected from this activity for any listed species. Underwater noise from other construction activities (e.g., dredging, placement of fill, and vessel operations) is expected to be less than or equal to vibratory pile driving and therefore contained within the area of effect for vibratory pile driving (Figure 5.3.8-1). Vibratory pile driving is expected to occur for 5-10 months in the Mississippi River cofferdam vicinity, and up to 4 months in the basin outfall vicinity.

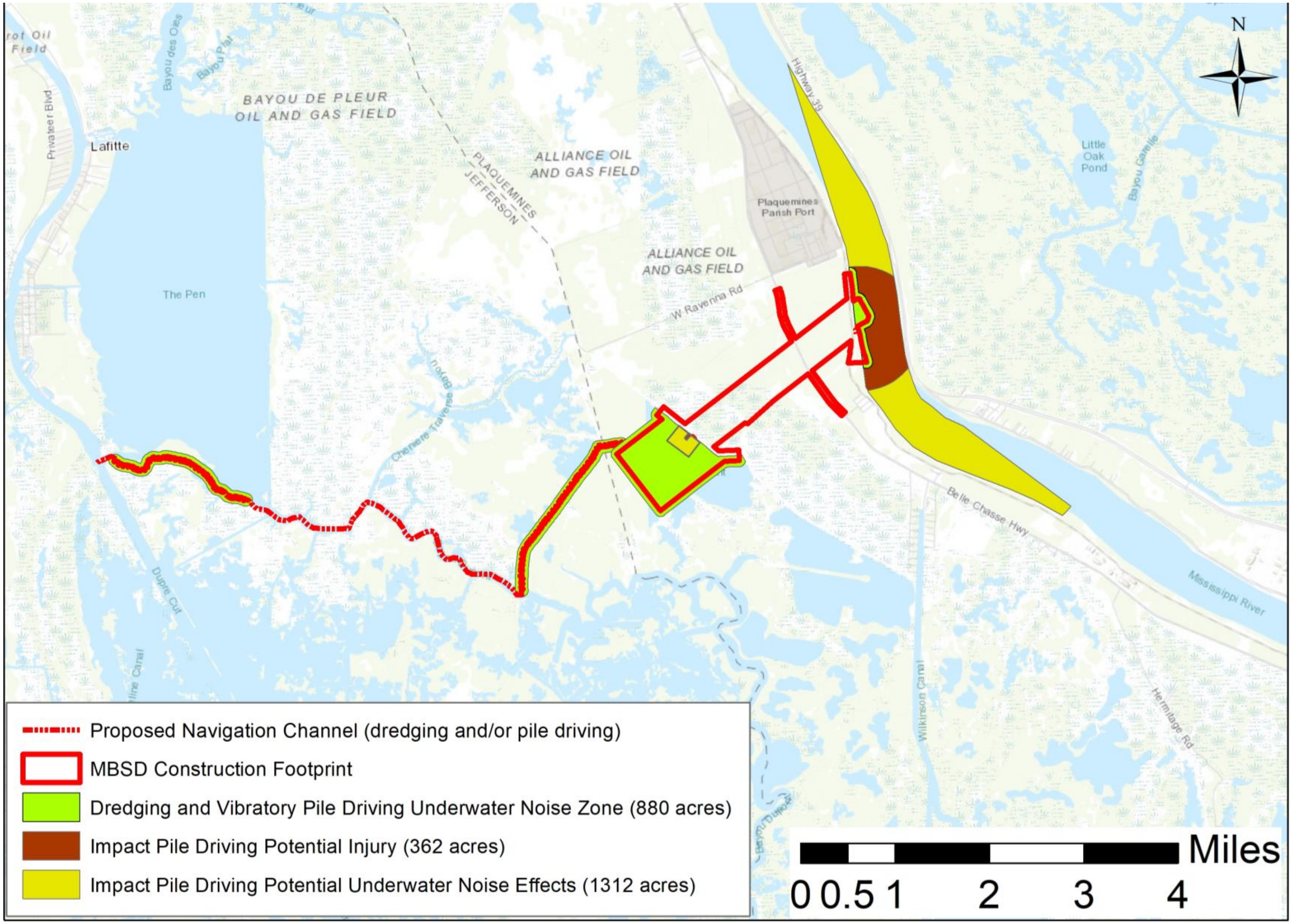


Figure 5.3.8-1. Maximum Potential Extent of Underwater Noise from Project Pile Driving

Project Operation

There would be no Project effects on underwater sound due to Project operation.

5.3.9 Project Effects on Entrainment/Stranding

Project Construction

Entrainment or stranding of fish during Project construction activities may occur during initial dewatering of the river-side cofferdam, or dewatering of basin-side isolated areas. During construction, on both the river and basin-side of the limits of construction, some areas would be isolated using sheet pile cofferdams and partially or completely dewatered to support construction. Dewatering activities may result in fish resources being stranded within dewatered areas or entrained within pumps. About 9.25 acres of aquatic habitat is expected to be isolated and partially or fully dewatered during construction using cofferdams in the Mississippi River.

Project Operation

Diversion operations may result in some fish being drawn from the Mississippi River into the Barataria Basin as flows are diverted year-round. During the first few decades, the diversion is anticipated to operate at high-flow for an average of 9 months out of the year, (depending on annual flow cycles), and is predicted to slowly increase peak flow operation to a maximum of 11 months out of the year by 2070 (based on predicted future river flow volumes). During baseflow operations, organisms are less likely to be entrained from the Mississippi River than during high-flow conditions. No deterrent strategies are proposed at the diversion intake, and there are no return mechanisms built into the diversion. All species entrained by the diversion would be considered lost to the Mississippi River system. Barataria Basin species are not anticipated to experience any entrainment or stranding from the Project. In addition, due to the unidirectional flow and velocity of flow from the diversion, no species from Barataria Basin are expected to move into the Mississippi River.

5.3.10 Project Effects on Habitat Area

Project Construction

Construction would include isolation of about 9.25 acres of the Mississippi River using a cofferdam with approximately 60-foot wide cells supported by a stability berm. Within the isolated area about 8 acres of area would be excavated for the intake structure development.

Within the Barataria Basin, about 34 acres would be modified for the development of the outfall transition feature. About 515,000 cubic yards (cy) of material are predicted to be removed during development of the outfall transition feature. In addition, it is estimated that up to 2 miles of 70-foot wide, 4-foot deep channel dredging would occur to support construction access to the diversion outfall. This is an additional 16 acres and approximately 100,000 cy of dredging. Minor losses of marsh and SAV would be expected within the project footprint within Barataria Basin. There would be no major Project effects on habitat area due to the restricted area of construction activities. Minor effects to upland habitat would be constrained to within the Project limits of construction.

In addition, a new 60-foot-wide trestle and 160-foot by 100-foot mooring facility would be constructed to support material handling during construction in the Mississippi River. This structure would be integrated into the downstream portion of the proposed cofferdam associated with construction of the intake structure. A permanent boat ramp and dock capable of launching a 35-foot boat is planned downstream of the intake structure.

Project Operation

This section will focus on changes to habitat area resulting from the proposed Project, including changes to fresh- and saltwater marshes, wetlands, SAV, and other relevant habitat features for listed species and prey resources. Existing aquatic resources and habitat throughout the action area are described in more detail in Section 4.5 above.

Marsh Habitat

The FWOP is predicted to lose up to 85% of emergent marsh habitats throughout the action area due to climate and sea level predictions. With the Project, over 16,500 acres of marsh habitats are anticipated to be maintained or created within Barataria Basin by 2050 as compared to the FWOP (see Table 5.3.10-1 below).

During initial high-flow periods of Project operation, initial sediment deposition may temporarily decrease the quality of marsh habitat adjacent to the diversion by inundating vegetation (e.g., SAV) and smothering benthic organisms. After initial high-flow periods, marsh vegetation and invertebrates are anticipated to recover in area and quality (see Figure 5.3.10-1 below).

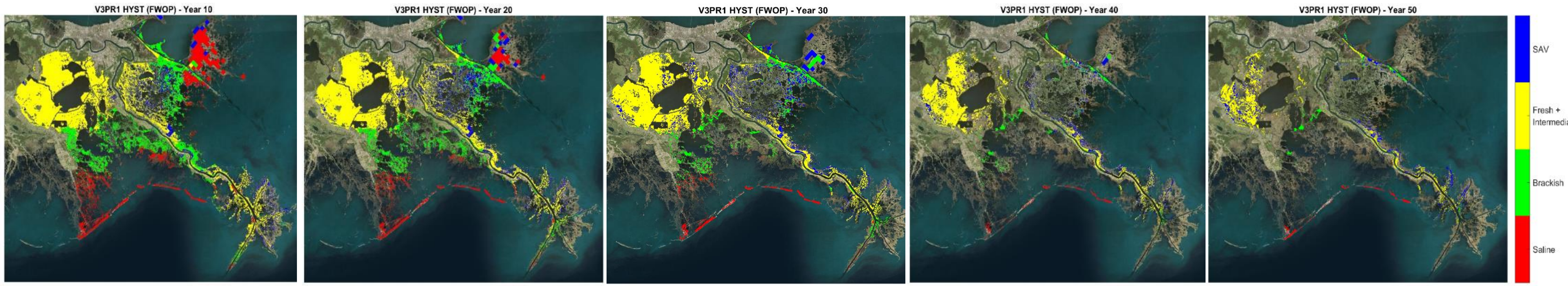
Compared to the FWOP, the Project is anticipated to retain or create more vegetation in fresh and intermediate marsh habitats (see Figure 5.3.10-2). The largest amount of vegetation would be created and maintained adjacent to the diversion, in areas where land would be created or maintained. Because of the freshwater influence of the diversion, some marsh areas adjacent to the Project and in the mid Barataria Basin would shift from brackish communities to fresh-intermediate marsh communities. In the lower basin, small areas of saline-vegetated habitats would initially shift to brackish marsh habitats, but ultimately the same amount of saline-vegetated habitat is predicted to be retained in the FWP and the FWOP.

Many species rely upon marsh habitats across a wide range of salinities and would benefit from the retention of marsh habitat with Barataria Basin. Marsh communities may shift in composition of vegetation and invertebrates due to shifting water quality conditions (e.g., temperature and salinity). These changes may have a diverse array of effects on marsh species and species that rely on specific marsh habitats or prey.

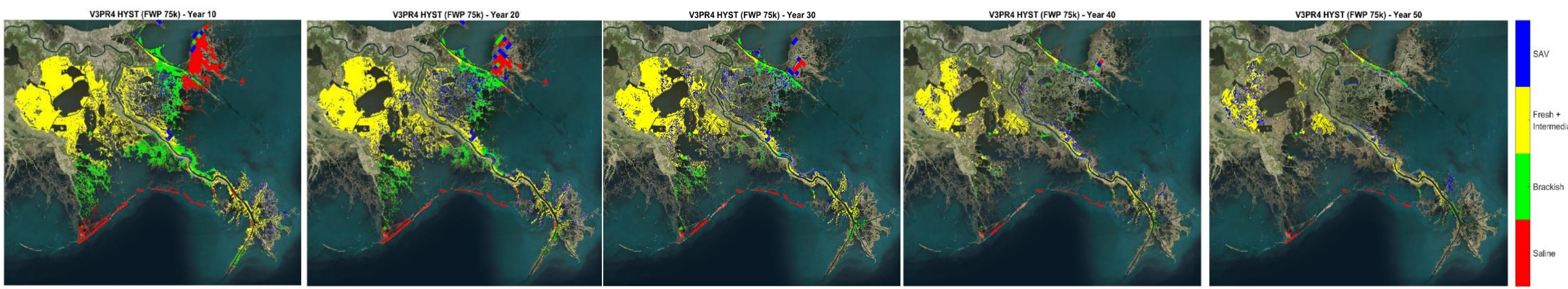
Table 5.3.10-1 Predicted Marsh Habitats within the Action Area

Marsh Habitat Type	Predicted Marsh Habitat (acres)					
	Cycle 0	Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5
	2020-2029	2030-2039	2040-2049	2050-2059	2060-2069	2070
FWOP						
FWOP Fresh + Intermediate	278,081	263,712	228,253	189,589	130,924	66,396
FWOP Brackish	80,969	73,217	55,635	29,069	11,972	6,352
FWOP Saline	70,923	44,900	28,651	16,478	6,967	6,454
Total	429,973	381,829	312,539	235,136	149,863	79,202
FWP						
FWP Fresh + Intermediate	313,995	304,926	273,240	219,914	153,215	79,526
FWP Brackish	68,637	57,918	34,037	20,461	5,804	3,219
FWP Saline	47,367	23,062	16,533	11,308	7,335	6,248
Total	429,999	385,906	323,810	251,683	166,354	88,993
Difference (FWP-FWOP)						
Fresh + Intermediate	35,914	41,214	44,987	30,325	22,291	13,130
Brackish	(12,332)	(15,299)	(21,598)	(8,608)	(6,168)	(3,133)
Saline	(23,556)	(21,838)	(12,118)	(5,170)	368	(206)
Total	**26 (+<1%)**	**4,077 (+1%)**	**11,271 (+4%)**	**16,547 (+7%)**	**16,491 (+11%)**	**9,791 (+12%)**

Future without Project Vegetation



Future with Project (75,000 CFS diversion) Vegetation



Year 10 Year 20 Year 30 Year 40 Year 50

Figure 5.3.10-1.Aquatic Vegetation over Time in Barataria Basin and Birdfoot Delta with and without the Proposed Project (75k diversion scenario)

Submerged Aquatic Vegetation

Although the Project would have the most notable effects on marsh habitats, the Project is also anticipated to affect SAV communities within Barataria Basin, such as algae and macroalgae; including *Sargassum.* Most *Sargassum* occurring in the project area is pelagic and development of *Sargassum* mats are likely driven by seasonal, regional wind and current patterns that are not affected by the project. Nutrients carried by the diversion through Barataria Basin may support phytoplankton (Wissel et al. 2005) and nutrient loading may enhance Sargassum growth locally (Brooks et al. 2018). Widgeon grass which may occur in low levels near the barrier islands may also be minimally affected.

In the FWOP, SLR is predicted to decrease SAV throughout the lower basin, with SAV declining with increasing depth and decreasing light availability. In the FWP, as the Project creates land and additional shallow water habitat, it is projected to decrease SAV loss as a result of the project. SAV was evaluated using multiple approaches; however, the premise of the SAV Likelihood of Occurrence Model (SLOO) (DeMarco et al. 2018) is believed to be the most representative data for this project. Without the project, SAV is projected to decline from approximately 9% of the basin area to 2% over the 50-year evaluation period. As a result of the project, this model approach indicates the area suitable for SAV is about 2% (1,500 acres) higher in the fresh/intermediate portion of the project area at the end of the project life (USFWS 2020b). Operation of the diversion is projected to build new land and will also increase the elevation of existing marshes or sediment beds (Carle et al. 2015).

As the project would seasonally decrease salinities in the mid-basin, species composition of SAV in the mid basin would likely change. Operation of the proposed Project would likely result in increased habitat suitability for SAV species in the Barataria Basin that thrive in or tolerate intermediate to fresh water, while decreasing the habitat suitability of those that are adapted to more saline waters. Aquatic vegetation in general is more diverse and abundant in low-salinity habitats (Hillmann et al. 2016), which would likely benefit from the Project.

Water Column Depth

The FWOP is predicted to lose up to 85% of emergent marsh habitats throughout the action area. With the Project, more than 17,259 acres of marsh habitats (measured in land above sea level) are anticipated to be preserved within Barataria Basin by 2050 as compared to the FWOP (see Section 5.3.4 above for more description of the Project effects to sediment transport). Changes in water depth in the action area are summarized in Figure 5.3.10-2. Most benefits are concentrated in the immediate vicinity of the diversion outfall, while small decreases in sediment depth are anticipated in the Birdfoot Delta due to diversion of sediment into Barataria Basin. Sediment additions by the Project would also help to preserve areas of shallow water habitat that would otherwise be lost to SLR in the FWOP. For species that rely upon shallow water habitats, the Project would provide benefits through habitat preservation.

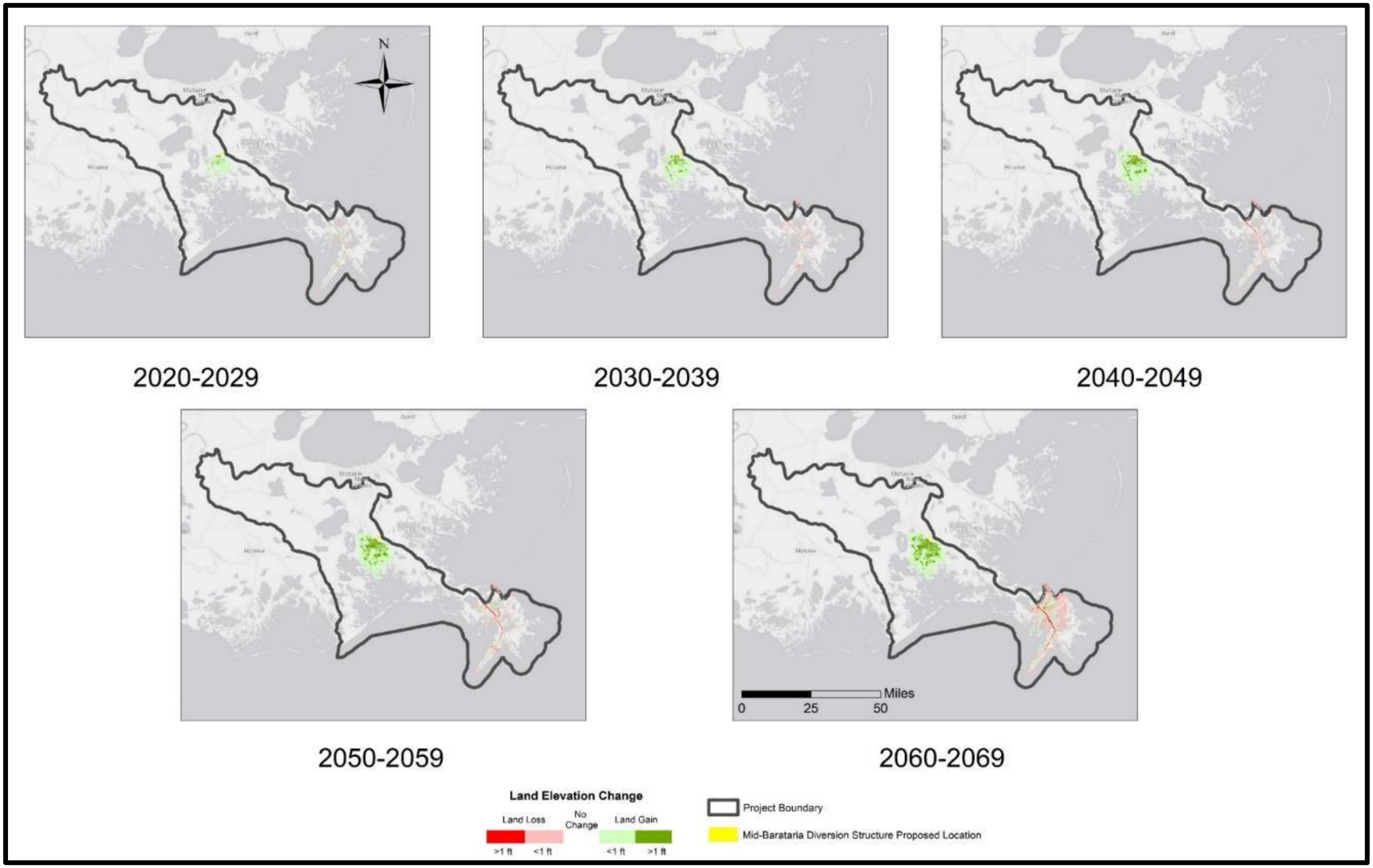


Figure 5.3.10-2.Depth over Time in Barataria Basin and Birdfoot Delta with Proposed Project (75k diversion scenario), compared to the Future without Project

5.3.11 Project Effects on Prey Base/Food Web

Project Construction

Project construction would have minor negative effects on the Barataria Basin prey base/food web. Aquatic construction activities (e.g., dredging, pile driving) would temporarily increase turbidity and alter marsh habitats within the limits of construction. All Project construction effects would be temporary, hyper-localized, and much smaller than the effects of Project operation (described below). Project operation effects to habitat are discussed in Section 5.4 below.

Project Operation

Prey base/food webs within marsh habitats in the mid-basin would experience temporary reductions in quality and biomass with initial peak Project operation flows. Peak flows are predicted to change the water quality and sediment delivery within the mid-basin, initially inundating some marsh vegetation adjacent to the diversion and shifting brackish marsh habitats to fresh/intermediate marsh habitats over the course of the first decade of the Project. The shift to intermediate marsh habitat, which is often more productive than brackish marsh habitat, may result in a net benefit to local food webs. Recoveries of both emergent vegetation and invertebrate communities are anticipated rapidly after initial flows, and ultimately the Project would substantially increase the amount of marsh habitat within the mid-basin in the FWP as compared to the FWOP.

The main drivers of prey base/food web effects from the Project would be primary production and salinity. Project operation is anticipated to enhance primary productivity during summer months within the mid and lower basin, in response to nutrient additions from Mississippi River flows (see Section 5.3.5 above). This Project-driven enhanced primary productivity is predicted to subsequently increase pelagic and benthic food resources for fishes and invertebrates.

However, the benefits of enhanced productivity are tempered by the mixed effects of reduced salinity throughout the mid-basin. Species tolerant of low salinity, such as blue crab, juvenile Gulf menhaden, red drum, and largemouth bass, would benefit most from the enhanced primary production and are predicted to show positive trends in habitat suitability indices (HSIs) across the basin, potentially resulting in increasing biomass, in the FWP as compared to the FWOP within the mid and lower basin. Higher salinity species such as brown shrimp, spotted seatrout, and oysters may benefit from increased primary production, but primarily within the lower basin during years of high river flows, where salinity is less influenced by the Project. These species are predicted to show trends of lower average basin HSIs, potentially resulting in decreasing biomass, in the FWP as compared to the FWOP. Due to Project salinity decreases in the mid-basin, higher salinity species may experience restrictions in available habitat and potentially decreased biomass over time within the mid-basin and concentrated biomass in the lower basin. Conversely, lower salinity species are expected to experience increases in available habitat and potentially increased biomass over time with the mid and lower basins.

Changes in habitat conditions may result in changes in the distribution of prey items. Specifically, changes in salinity are expected to create a southward shift in distribution of species that are sensitive to low salinity conditions. Species affected by low salinity conditions include commercially important species such as brown shrimp. Commercial fisheries are expected to adjust fishing effort and intensity as populations change in Barataria Basin. Commercially available brown shrimp populations are predicted to decrease and become concentrated in the lower portions of Barataria Basin. While fishing effort may or may not decrease in response to these changes over time, it is likely that fishing effort will become more concentrated in lower portions of Barataria Basin.

5.4 Direct and Indirect Effects on Species

This section discusses how the effect pathways discussed above affect listed species. Table 5.4-1 describes presence and magnitude of effects to each ESA listed species. Species potential presence within the action area by life stage is described in Section 4.6.

Project effects may be short, intermediate or long term in duration. Short term (or temporary) effects are typically those associated with construction or other intermittent activities where the effect duration is expected to be minutes to months in total duration. These are typically effects associated with construction or other temporary project components. Intermediate effects are predicted to occur for a specified duration that are longer than short term effects and may occur for years or decades, but are predicted to end. These effects are not permanent. Long term effects are projected to continue may exist for the duration of the project or for the foreseeable future.

Table 5.4-1. Table of Effects to ESA Species

Species	Life Stage	Salinity	Turbidity/ Suspended Sediment	Dissolved Oxygen	Water Temperature	Sediment Deposition/ Land Creation	Water Quality/ Contaminants	Nutrients	Sound/ Noise Effects	Entrainment/ Stranding	Physical Disturbance of Organisms	Shading (overwater structure)	Prey	Presence of Marsh Vegetation	Depth
Pallid Sturgeon	juveniles	-	n	n	-	-	n	-	x	x	n	n	-	-	-
	adults	-	n	n	-	-	n	-	x	x	n	n	-	-	-
Black Rail	eggs	-	-	-	-	+	-	-	x	-	x	-	x	x; +	-
	hatchlings	-	-	-	-	+	-	-	x	-	x	-	x	x; +	-
	juveniles	-	-	-	-	+	-	-	x	-	x	-	x	x; +	-
	adults	-	-	-	-	+	-	-	x	-	x	-	x	x; +	-
	breeding pairs	-	-	-	-	+	-	-	x	-	x	-	x	x; +	-
Piping Plover	juveniles	-	-	-	-	+	-	-	n	-	n	-	x	x; +	-
	adults	-	-	-	-	+	-	-	n	-	n	-	x	x; +	-
Red Knot	juveniles	-	-	-	-	+	-	-	n	-	n	-	x	x; +	-
	adults	-	-	-	-	+	-	-	N	-	n	-	x	x; +	-
West Indian Manatee	juveniles	n	n	n	x	-	n	n	n	-	n	n	-	x; +	-
	adults	n	n	n	x	-	n	n	n	-	n	n	-	x; +	-
Green Sea Turtle	juveniles	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	adults	x	-	n	n	-	-	-	-	-	-	-	n	-	-
Hawksbill Sea Turtle	juveniles	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	adults	x	-	n	n	-	-	-	-	-	-	-	n	-	-
Kemp's Ridley Sea Turtle	juveniles	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	adults	x	-	n	n	-	-	-	-	-	-	-	n	-	-
Leatherback Sea Turtle	juveniles	x	-	n	n	-	-	-	-	-	-	-	-	-	-
	adults	x	-	n	n	-	-	-	-	-	-	-	-	-	-
Loggerhead Sea Turtle	eggs	x	-	-	-	-	n	-	-	-	-	-	-	-	-
	hatchlings	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	juveniles	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	adults	x	-	n	n	-	-	-	-	-	-	-	n	-	-
	nesting adults	x	-	-	-	-	n	-	-	-	-	-	-	-	-

- = Not applicable, outside of effect range
n = No effect or negligible effect
x = Minor effect (e.g. short duration, small geographic extent).
xx = More than Minor effect (to be assessed in more detail)
+ = Potential for positive effect

5.4.1 *Project Effects on Pallid Sturgeon*

Species Tolerances Relevant to the Project

- Pallid sturgeon entrained by the Project would be assumed lost to the population.
- There are no established tolerance levels of turbidity for pallid sturgeon. It is expected that pallid sturgeon are highly tolerant of elevated levels of turbidity given their known distribution and benthic feeding patterns.
- Underwater sound levels for fish behavioral disruption and injury have been established by collaborative agreement of NOAA, USFWS, and the U.S. Federal Highway Administration (WSDOT 2008). See Table 5.4.1-1 below.

Table 5.4.1-1. Guidance on Fish Underwater Noise Thresholds

Functional Hearing Group	Noise Thresholds	
	Behavioral Disruption Threshold	Injury Threshold
Fish > 2 grams Fish < 2 grams Fish all sizes	150 dB RMS	187 dB Cumulative SEL 183 dB Cumulative SEL Peak 206 dB
SEL = sound exposure level = 1 dB re 1 μPa2 -sec RMS = For pile driving, this is the square root of the mean square of a single pile driving impulse pressure event Source: WSDOT 2018, NMFS 2018		

Project Construction

<u>**Disturbance or Injury**</u>

Construction activities in the Mississippi River, such as dredging, vessel operations, pile driving, and pier construction, have the potential to physically disturb or injure pallid sturgeon present within the action area. The loudest underwater sound that sturgeon may encounter would be generated by impact pile driving activities, which have the potential to injure fish present within 3,281 feet during impact pile driving activities. Pier construction, including pile driving activities, would be located along the western bank near RM 60.7. The Mississippi River is about one-half mile wide at the Project location, which may not allow for unobstructed passage by fish through areas of elevated noise. Fish present within the area during pile driving activities may be affected by elevated underwater sound levels. High underwater SPLs are known to injure and/or kill fish by causing barotraumas (injuries caused by pressure waves, such as hemorrhage and rupture of internal organs), as well as causing temporary stunning and alterations in behavior (Turnpenny et al. 1994, Turnpenny and Nedwell 1994, Popper 2003, Hastings and Popper 2005). Fish with swim bladders, such as sturgeon, are more susceptible to barotraumas from impulsive sounds than fish without swim bladders. Brown et al. (2013) found that for white sturgeon, juveniles are susceptible to barotrauma after initial feeding (about 9 days post hatch) due to the potential for herniation in their intestines, while their swim bladders partially inflate later in development (later than 75 days post hatch) and due to the physiology of the swim bladder in sturgeon gas transfers from the swim bladder can be released through the sturgeon's mouth. Any gas-filled structure within an animal is particularly susceptible to the effects of underwater sound (Gisiner et al. 1998). When practicable, the use of vibratory hammers, rather than impact hammers, to install in-water piles would avoid the major potential physical effects to fish from barotrauma.

Behavioral responses to elevated underwater sound in fish are not well understood. Behavioral responses may include avoidance of the area, a startle response, or delayed foraging. Mueller et al. (1998) and Knudsen et al. (1992, 1994) found that juvenile salmonids (40- to 60-millimeter length) exhibit a startle response followed by a habituation to low frequency (infrasound) noise in the 7 to 14 Hz range. Mueller et al. (1998) and Knudsen et al. (1992, 1994) also indicate that noise intensity level must be 70 dB to 80 dB above the hearing threshold at 150 Hz to obtain a behavior response. According to Feist et al. (1992) broad-band pulsed noise (e.g., impact pile driving noise) rather than continuous, pure tone noises (e.g., vibratory pile driving) are more effective at altering fish behavior. According to Olsen (1969), in order to produce a behavioral response in herring, ambient sound must be at least 24 dB less than the minimum audible field of the fish and the pile driving noise levels have to be 20 dB to 30 dB higher than ambient sound levels. Herring are similar in size to juvenile sturgeon life stages, and may serve as a model organism.

There is little evidence that increases in underwater sound from the vessel operations, dredging, or vibratory pile driving would result in adverse behavioral shifts of fish. However, it is possible that individuals exposed to elevated underwater sound levels could exhibit an avoidance response or temporary displacement from foraging activities, resulting in reduced foraging success and/or undue energy expenditure. The duration of such a response is expected to be only short-term and intermittent, correlating with brief encounters with mobile vessels or instances of pile driving (about 8-12 hours/day).

Habitat Alteration

During temporary construction activities and seasonal operation, the proposed Project has the potential to alter downstream pallid sturgeon habitat, such as scour holes, sandbars, and flow refugia, through the alteration of Mississippi River flow volumes downstream of the Project. As the Mississippi River is a dynamic system, the Project alterations to downstream habitat are not likely to be of consequence. The Project would not alter habitat within pallid sturgeon spawning areas, but may alter habitats used by larvae, juveniles, or migrating adults. Construction area isolation using a cofferdam in the Mississippi River may reduce the habitat available to pallid sturgeon by about 9.25 acres, and any fish isolated in the cofferdam area during installation may be lost.

Project Operation

Entrainment

Diversion operations would be the main sources of entrainment risk for pallid sturgeon. Pallid sturgeon that are entrained in the diversion are assumed to be lost from the Mississippi River population and are not expected to be able to return to the river. Entrainment is expected to be highest during periods of high Mississippi River flows when large amounts of water pass through the diversion (generally January–July). High volume and high velocity flow through the diversion are likely to convey any fish in the water column.

Past studies (Schultz 2013) found that small numbers of pallid sturgeon were entrained by the Bonnet Carré Spillway at RM 133 and the Davis Pond Freshwater Diversion at RM 119. Smaller diversions at RM 83.8, 81.5, 64.5, and 63.9 were also sampled; however, no pallid sturgeon were detected. The Project is located at RM 60.7, downstream of documented spawning sites and previously studied diversion locations.

Entrainment risk may be affected by water temperatures. While a total of 31 pallid sturgeon and 122 shovelnose sturgeon were collected following the 2008 and 2011 Bonnet Carré openings when water temperatures were 23-30 ˚C, only 1 sturgeon was collected following the 2016 opening (when temperatures were about 10 ˚C) despite substantial fishing effort (USACE 2017).

Pallid sturgeon also have positive rheotaxis and orient towards flow (Hoover et al. 2011). This may affect the likelihood that sturgeon would enter the diversion intake and it may also affect the distribution of sturgeon in Barataria Basin for fish that become entrained by the diversion flow. Pallid sturgeon collected at Bonnet Carré were found near the spillway structure and in depressions being dewatered after the closure of the spillway. While the MBSD has a different design and purpose, it is possible that when the MBSD transitions from peak to base flow conditions, pallid sturgeon would similarly be found near the diversion structure.

Friedenberg (2019) has developed an estimate for the entrainment risk associated with the MBSD (Appendix C). Entrainment risk is a function of the abundance of pallid sturgeon present in the action area and the likelihood of entrainment during operations. To estimate the abundance level of pallid sturgeon in the action area, 3 potential density scenarios were evaluated based on a conservative estimate of the abundance of pallid sturgeon in the system (Friedenberg et al. 2018). Because entrainment estimates are based on the predicted number of fish present per volume of water, this method characterizes the greatest potential effect from entrainment losses to the population, overestimating the effect of a given level of entrainment on the population. The scenarios evaluated include the following:

- **50% population density below New Orleans** – Consistent with the observations at Davis Pond and Caernarvon freshwater diversions, the abundance is assumed to be 50% lower south of New Orleans compared to the rest of the lower Mississippi River. Despite the lack of detections downstream of New Orleans, this level of density represents the maximum population density that is consistent with the lack of detections of pallid sturgeon during monitoring at the Caernarvaron small diversion (Schultz 2013).

- **10% population density below New Orleans** – ERDC sampling data give a mean estimate of 10% population density downstream of New Orleans relative to upstream area, with all age classes present.

- **Only juveniles below New Orleans** – The absence of sturgeon in sampling and diversion observations suggests that sturgeon vulnerable to entrainment may be rare or absent. However, juveniles less than 3 years old that are not collected with typical survey gear may be present and are expected to be present because larvae likely drift long distances (Kynard et al. 2007, Braaten et al. 2008, Braaten 2010).

Using these estimates, local and Lower Mississippi River (LMR) estimates of abundance for pallid sturgeon were generated (see Table 6.4.1-1). The population estimate is combined with the entrainment risk, which assumes that fish are evenly distributed and therefore proportional to the volume of Mississippi River water diverted. Volumetric entrainment rates were either based on USFWS derived rates (USFWS 2018) or mark-recapture rates (Schultz 2013) predicted or observed in diversions. These rates were applied to generate annual volumetric estimates (see Table 5.4.1-2).

These estimates may overestimate entrainment risk because pallid sturgeon may be at lower risk of entrainment during the winter's low water temperatures. This hypothesis is supported by the observation that pallid sturgeon are caught in deeper water during winter (DeVries et al. 2015) and that few pallids were entrained during a January opening of the Bonnet Carre in 2016 (USFWS 2018). Studies have noted reduced growth and survival of juvenile *Schaphyrynchus* at 10 and 12 °C (Kappenman et al. 2009) and reduced sustained swimming speed at 10 C (Adams et al. 2003), suggesting metabolic stress and the possibility that individuals may seek energetic refugia and reduce activity during winter. If entrainment does not occur at or below a lower temperature threshold of 10 or 12 °C then annual entrainment would be reduced by 27% to 36%.

Table 5.4.1-1. Abundance of Age 1+ Pallid Sturgeon Used to Calculate Entrainment Mortality at the Scale of the Local Population and the Lower Mississippi River

Age Structure	Pallid Sturgeon Abundance	
	Local Population	Lower Mississippi River Population
50% Density	1,954	7,177
10% Density	1,806	7,031
Juveniles Only	1,769	6,994
Source: Friedenberg et al. 2018		

Table 5.4.1-2. Predicted Mean Annual Pallid Sturgeon Entrainment through the MBSD

Age Structure	Ages Entrained	Mean Annual Entrainment Estimates	
		USFWS 2018 Capture Rate* mean (SD)	Mark-Recapture Rate** mean (SD)
50% Density	Age 1+	58.0 (19.1)	34.8 (11.5)
10% Density	Age 1+	11.6 (3.8)	7.0 (2.3)
Juveniles Only	Age 1-2	20.2 (6.7)	12.1 (4.0)
*USFWS 2018 methods; **Schultz 2013 methods SD = standard deviation Sources: Schultz 2013, Friedenberg 2019			

These projections were applied to simulations of future flows over the next 50 years to generate predicted mean total entrainment over the Project operational period (see Table 5.4.1-3). These were also compared to projections of pallid sturgeon population growth to evaluate the potential effects of removing individuals from the population due to the diversion. The simulations resulted in reduced annual population growth rates ranging from a reduction of 0.07 to 0.43 percent depending on the entrainment scenario, with juvenile only scenarios resulting in the least potential effect to population growth and 50 percent densities resulting in the greatest. Although more juvenile pallid sturgeon are entrained in the juvenile only scenario, these fish are not yet reproductive and annual mortality rates for these life stages are relatively high, meaning that losses of individual juvenile pallid sturgeon contribute less to population growth than losses of adults.

Table 5.4.1-3. Predicted Mean Total Pallid Sturgeon Entrainment through the MBSD over 50 Years

Age Structure	Mean Total Entrainment Over 50 Years Estimates	
	USFWS 2018 Capture Rate* mean (SD)	**Mark-Recapture Rate**** mean (SD)
50% Density	2,403 (292)	1,561 (186)
10% Density	515 (62)	350 (47)
Juveniles Only	1,020 (281)	647 (191)

*USFWS 2018 methods; **Schultz 2013 methods
SD = standard deviation
Sources: Schultz 2013, Friedenberg 2019

These entrainment scenarios characterize the anticipated high (50% density scenario) and low (10% density and juvenile only scenarios) expectations for pallid sturgeon effects. There is insufficient fishery data on pallid sturgeon to determine which scenario best represents the conditions expected to occur, so a conservative assumption is that the 50% density scenario represents the maximum predicted entrainment and population effects from the Project.

Several prior Biological Opinions have authorized take for pallid sturgeon. While most consultations have concerned research and recovery efforts; however, several consultations have authorized lethal take for flow diversions. Table 5.4.1-4 presents all take of pallid sturgeon authorized by Biological Opinions completed for the Lower Mississippi River.

Table 5.4.1-4. Historical Take of Pallid Sturgeon from the Mississippi River

Opinion Year	Issue	Authorized Take	Take Reported
2003[1]	BO on Natchitoches National Fish Hatchery's collection of Endangered Pallid Sturgeon from Louisiana Waters for Propagation and Research	revised	23 adult harassment (2003)
2004	Modification to revise 2003 IT estimates for BO (4-7-3-702) on Natchitoches National Fish Hatchery's Activities	120 adults/season for 5 seasons (harassment) 14 adults/season for 5 seasons (potential death)	329 (Atchafalaya) harassment (through 2010) 7 adults dead (2004)
2004	Programmatic BO Addressing Effects of the Southeast Region's Section 10(a)(1)(A) Permitting on the Pallid Sturgeon (5-years)	28 adults in captive propagation/year (death) 2,500 - 15,000 captive year-class 90 days or older (one-time loss-death)[2] 200 larval/juvenile/year sampling (death) 3 class ≥5-inches fish/year netting (death or injury) 3 fish/year external tagging (death or injury) 1 fish/year transport (death) 5 fish/year radio-tracking (death or injury)	461 (lower Mississippi River) harassment (through 2012) 1 dead (2006) 2 dead (2007) 1 dead (2009)
2005	Modifications of Programmatic BO-adding new forms of take to the 2004 revisited IT (4-7-04-734) for the 2003 BO (4-7-03-702) on Natchitoches National Fish Hatchery's Activities	14 wild fish/season (death) 15,000 hatchery-reared fish/season (potential death)	NA
2009	BO on Emergency Opening of Bonnet Carré Spillway, USACE	14 adults (harassment) 92 adults (death)	14 adult harassment unknown deaths
2010	BO on Medium White Ditch Diversion	23 adults/year (potential death)	0
2010	BO on Small Diversion at Convent/Blind River	7 adults/year (potential death)	0
2010	Taxonomic ID Study	100 adults (death)	76 adult deaths
2013	Mod-Programmatic BO	21 adults/year (potential death)	0
2013	USACE CIP	unspecified	0
2014	USACE Permits for Sand and Gravel Mining in the Lower Mississippi River	unspecified	NA
2018	BO on Bonnet Carré Spillway 2011 and 2016 Emergency Operations, USACE	2011: 20 adults (harassment) 82 adults (potential death) 2016: 26 adults (potential death) 0 larvae/juvenile (potential death)	2011: 20 adults (harassment) unknown deaths 2016: 0 adults (harassment) unknown deaths (assumed ≥1)

Opinion Year	Issue	Authorized Take	Take Reported
2020	BO on Bonnet Carré Spillway 2018 Emergency Operations, USACE	14 adults (potential death)	2 (harassment) 2 (death)
Pallid Sturgeon Total[1]		**Hatchery Related**[2]**:** 120 adults/season for 5 seasons (harassment) 28 adults/year (potential death) 2,500-15,000 year-class 90 days or older (potential death) **Taxonomic or other Permitted Study Related:** 100 adults (death) 200 larval fish/year (potential death) 3 fish/year external tagging (death or injury) 1 fish/year transport (death) 5 fish/year radio-tracking (death or injury) **Diversion Related:** 34 adults (harassment) 92 adults (death) 173 adults (potential death)	**Hatchery Related**[2]**:** 329 (Atchafalaya) harassment 7 adult deaths **Taxonomic or other Permitted Study Related:** 461 (lower Mississippi River) harassment 76 adult deaths 4 deaths <200/year larvae collected **Diversion Related:** 36 adult harassment 2 adult deaths
BO = biological opinion [1] The original estimates for the 2003 BO are not included as they were revised in 2004. [2] Hatchery propagation was terminated in Region 4 in 2005.			

Pallid sturgeon are described in detail above in Section 3.2.1, and their use of the action area is described above in Section 4.6.1.

5.4.2 Project Effects on Eastern Black Rail

Species Tolerances Relevant to the Project

- There are no established tolerance levels of disturbance from airborne sound or human proximity for Eastern black rail, though there is consensus that disturbance events, especially during the nesting season and flightless life stages, have potential negative effects on individuals and nesting success.

Project Construction

Disturbance

Project construction activities may disrupt resident Eastern black rails throughout their nesting and non-nesting seasons if black rails are present. Eastern black rails are cryptic, marsh-dependent, and not anticipated to be present in high densities near the diversion location. Construction activities will occur outside of marsh habitats and are therefore not likely to interact with the rail's preferred foraging and resting areas, and outside all nesting areas. The unlikely disturbance of non-nesting black rails may displace birds from resting or foraging activities. A common strategy to address this, and ensure that similar habitats are protected, is to provide disturbance-free areas of refuge near areas of active disturbance, thus reducing or eliminating the negative effects from disturbed areas. The proposed Project's construction activities are not anticipated to directly affect preferred resting, nesting, or foraging habitat for black rails, though these birds may be in the general area in low densities. Direct temporary disturbance would be restricted to within the limits of construction, and indirect temporary disturbance may occur in areas adjacent to the limits of construction, with nearby areas having preferred marsh habitats appropriate for black rail resting, nesting, and foraging. Black rail, potentially present in the area in very low densities, could be excluded from a small portion of available (not preferred) resting and foraging habitat within the project footprint during Project construction activities.

Project Operation

Habitat and Prey Alteration

Peak operation flows are predicted to change the water quality within the mid-basin, initially inundating marsh vegetation adjacent to the diversion and shifting brackish marsh habitats to mixed/fresh marsh habitats over the course of the first decade of the Project throughout the mid-basin. Emergent vegetation and invertebrate community biomass is anticipated to recover rapidly after initial flows, so there would only be a short period where the black rail's preferred habitat and prey would be decreased adjacent to the diversion. The temporary decrease in available quality habitat due to the Project is not anticipated to affect mobile black rail.

The Project is anticipated to add and maintain areas of marsh habitat adjacent to and near the Project diversion, preserving essential marsh habitats in comparison to the FWOP. Changes to marsh habitat vegetation and infauna communities would change the composition of available prey resources for secretive marsh birds such as black rail, but would also preserve and increase the area of available marsh habitat in the mid-basin. Long-term effects to black rail, which do not show preference between marsh types, are anticipated to be positive, with black rail benefiting from areas of marsh habitat creation and preservation.

5.4.3 Project Effects on Piping Plover and Piping Plover Critical Habitat

Species Tolerances Relevant to the Project

- There are no established tolerance levels of disturbance from airborne sound or proximity for migrating and resting shorebirds, though there is consensus that disturbances have potential negative effects on individuals.

Project Construction

Disturbance

Project construction activities may disrupt migrating or resting piping plover individuals or groups. Piping plover that occur in the action area are more likely to be present in the barrier islands region of Barataria Basin, and thus have a low likelihood of interacting with construction activities.

Project-related construction disturbance may displace birds from resting or foraging areas, potentially leading to the abandonment of these areas. Repeated and long-term disturbances during migration and resting may decrease a bird's ability to build up adequate fat stores, potentially leading to decreased individual or group fitness and population level effects (Pfister et al. 1992). However, piping plover occur infrequently during migration within the mudflats and estuarine habitats within or adjacent to the Project construction footprint (see Section 4.6.3). Providing disturbance-free areas of refuge near areas of active disturbance could ensure protected areas of similar habitats, reducing or eliminating the negative effects from disturbed areas. During this Project, temporary disturbance would be restricted to within the limits of construction; areas beyond the limits of construction with similar marsh habitats appropriate for shorebird resting and foraging would remain undisturbed. Shorebirds such as the piping plover prefer sand spits, beaches, sand flats, and mudflats associated with barrier islands or Gulf shoreline headlands. The proposed Project's construction activities are not anticipated to affect preferred resting and foraging habitat for piping plover.

Piping plover critical habitat, within the barrier islands, would not be impacted by Project construction activities.

Project Operation

Habitat and Prey Alteration

Peak operations flows are predicted to change the water quality within the mid-basin, initially inundating marsh vegetation adjacent to the diversion, and shifting brackish marsh habitats to mixed/fresh marsh habitats over the course of the first decade of the Project throughout the mid-basin. Emergent vegetation and invertebrate community biomass are anticipated to recover rapidly after initial flows and species composition would adjust to the new salinity regime, so there would only be a short period where marsh habitat and prey would be decreased. The temporary decrease in available quality marsh habitat due to the Project is not likely to affect mobile migrating piping plover because they have access to abundant sources of existing habitats.

The Project is anticipated to add and maintain areas of marsh habitat adjacent to and near the Project diversion, but would not significantly alter barrier islands island habitats along the edges of the action area. Piping plover, which are more likely to be present along the barrier islands, are not likely to experience changes in available habitat or prey due to the Project.

Piping plover critical habitat, near the Birdfoot Delta and within the barrier islands, is not anticipated to be impacted by Project operations.

5.4.4 Project Effects on Red Knot

Species Tolerances Relevant to the Project

There are no established tolerance levels of disturbance from airborne sound or proximity for migrating and resting shorebirds, though there is consensus that disturbances have potential negative effects on individuals.

Project Construction

Disturbance

Project construction activities are unlikely to disturb migrating or resting red knot individuals or groups. Red knot are not likely to be present in the marsh habitats near construction activities. Red knots prefer sand spits, beaches, sand flats, or mudflats associated with barrier islands or Gulf shoreline headlands. They are found within the action area seasonally, in low densities, and unlikely near Project construction activities. Red knot are considered rare to uncommon along the Louisiana coast, but are considered a regular visitor to Grande Isle (Fontenot and Demay 2014).

Disturbing migrating or resting shorebirds may displace birds from preferred resting or foraging areas, potentially leading to the abandonment of the disturbed areas. Repeated and long-term disturbances during migration and resting may decrease a bird's ability to build up adequate fat stores during migration, potentially leading to decreased individual or group fitness and population level effects (Pfister et al. 1992). However, providing disturbance-free areas of refuge near areas of active disturbance is a common strategy used to ensure protected areas of similar habitats, reducing or eliminating the negative effects from disturbed areas. The proposed Project's construction activities are not anticipated to affect preferred resting and foraging habitat for red knots. Temporary disturbance would be restricted to within the limits of construction, with areas to the north, south, and east with shorelines and marsh habitats that are less appropriate for shorebird resting and foraging. Red knots are not likely to be disturbed by construction activities due the lack of suitable habitat in the construction area.

Project Operation

Habitat and Prey Alteration

Initial Project operations are anticipated to alter marsh habitat within the mid-basin, especially immediately adjacent to the Project diversion. Peak operation flows are predicted to change the water quality within the mid-basin, initially inundating marsh vegetation adjacent to the diversion, and shifting brackish marsh habitats to mixed/fresh marsh habitats over the course of the first decade of the Project throughout the mid-basin. Emergent vegetation and invertebrate community biomass is anticipated to adjust to new salinity regime and recover rapidly after initial flows, so there would only be a short period where marsh habitat and prey would be decreased. The temporary decrease in available quality marsh habitat due to the Project would not affect mobile migrating red knots because they do not prefer such marsh habitat.

The Project is anticipated to add and maintain areas of marsh habitat adjacent to and near the Project diversion, preserving essential marsh habitats in comparison to the FWOP. Changes to marsh habitat vegetation and infauna communities would change the composition of available prey resources for migrating birds, but would also preserve and increase the area of available marsh habitat for bird use in the mid-basin. There are no anticipated effects to barrier island habitats where they are currently most likely to occur. Documented red knot habitat use suggests they are most likely to be present along the barrier islands and near the Birdfoot Delta, areas that are not likely to experience changes in available habitat or prey due to the Project.

5.4.5 Project Effects on West Indian Manatee

Species Tolerances Relevant to the Project

Table 5.4.5-1 lists tolerances of the West Indian Manatee relative to aquatic Project effects.

Table 5.4.5-1. Species Tolerances Relevant to the Project – West Indian Manatee

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project		
		Salinity (psu)	Water Temperature (°C)	Depth (m) Tolerance (optimal)
West Indian Manatee	juveniles adults	NL NL	>20 >20	NL (1-6) NL (2-6)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
NL = No reported tolerance limits.

Project Construction

Disturbance

West Indian manatees, which frequent Barataria Basin in low densities during warm summer months, may experience disturbance or exclusion from the small area of construction activities near the diversion outflow, within the basin side of the Project. It is also possible, but not likely, that transient manatees within the Mississippi River may travel through the action area on the river side of the Project. It is likely that water temperatures in the Mississippi River preclude Manatee presence during most periods.

Aquatic activities during Project construction (such as dredging, vessel operations, and vibratory sheet pile driving) at either the diversion inflow from the Mississippi River or the outflow in mid Barataria Basin, have the potential to physically disturb or displace West Indian manatees present within the action area. Underwater sound impacts on manatees are characterized by estimating instantaneous and cumulative exposure to sound intensity. The loudest underwater sound that manatees may encounter would be generated by impact pile driving activities. Manatees may also be exposed to underwater sound from dredging or boat activities. Outfall pile driving activities would be located along the south edge of the diversion construction. In the mid region of Barataria Basin, the construction area accounts for only a small fraction of available habitat for West Indian manatees, allowing them to pass safely around the action area. Manatees present within the area during pile driving activities may be affected by elevated underwater sound levels. When practicable, the use of vibratory hammers, rather than impact hammers, to install in-water piles would avoid the major potential physical effects to organisms from barotrauma. While impact pile driving of steel piles is proposed in the Mississippi River, planned impact pile driving in Barataria Basin is limited to installation of 12-inch timber pilings. Sirenians (manatees) have an estimated onset of temporary threshold shifts (TTS) at 187 dB re 1 μPa^2s. This level of underwater sound is predicted to occur within approximately 1 meter of timber pile driving activities.

Manatees have been found to avoid areas of elevated underwater noise, even within preferred seagrass habitats (Miksis-Olds et al. 2007). Therefore, it is likely that all construction activities that increase underwater noise, such as vessel operations, dredging, or vibratory pile driving, would result in avoidance behaviors and temporary displacement from foraging areas, resulting in reduced foraging success and undue energy expenditure. The duration of such a response is expected to be only short-term and intermittent, correlating with brief encounters with mobile vessels or instances of pile driving.

In order to avoid and minimize effects on West Indian manatees, all personnel involved with Project-related in-water work in potential manatee habitat shall be fully instructed and trained in measures for avoiding and minimizing manatee effects. In addition, CPRA would install bubble curtains around pile driving activities during work in the Barataria Basin. These conservation measures are fully described in Section 2.3.6 above.

Project Operation

Temperature

Project operations are anticipated to decrease water temperatures within the mid and lower basin during peak flows (see Section 5.3.2). However, during months where West Indian manatees may be present (May–September) temperatures are not anticipated to drop below manatee's minimum temperature tolerance of 20°C. No temperature effects are anticipated for West Indian manatees.

Habitat Alteration

The Project is anticipated to create and maintain areas of fresh and intermediate marsh habitat adjacent to and near the Project diversion, preserving land above sea level and marsh habitats in comparison to the FWOP. The same sediment contributions from the Project would also preserve and maintain areas within West Indian manatee's optimal depth range (1-6 m) in the mid-basin, adjacent to the diversion. While land loss is predicted throughout the proposed Project area due to sea level rise, increasing amounts of shallow water habitat near the diversion structure are predicted to result in increased total biomass of SAV over time. Project operations may also result in a shift to include more fresh and intermediate species of aquatic vegetation and SAV.

5.4.6 Project Effects on All Species of Sea Turtles

This section describes potential effects that may be common to all species of sea turtles in the action area. Species-specific characteristics are discussed for each species in Sections 6.4.7 through 6.4.11.

Table 5.4.6-1. Species Tolerances Relevant to the Project – All Sea Turtle Species

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project
		Salinity (psu)
All	hatchlings	> 0 psu**
	juveniles	> 0 psu**
	adults	> 0 psu**

*Showing all life stages that may potentially occur within the action area.
** Sea turtles may be unaffected by exposure to freshwater conditions for over 4 days
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Sources: Ortiz et al. 2000, Witt 2007, Valverde and Holzwart 2017

Project Construction: Upland

Sea turtles do not currently nest in the vicinity of the project construction area and habitat is likely inappropriate to support sea turtle nesting north of the barrier islands, which are about 40 kilometers south of the limits of construction (see Section 4.6).

Project Construction: Aquatic

Underwater Sound

Sea turtles are not likely to be affected by Project-related underwater noise because construction activities are restricted to a relatively small area in the mid-portion of Barataria Basin that is beyond the range where sea turtles are known to and expected to occur. However, sea turtles may move into the mid-Barataria Basin and travel through the relatively small canals and channels to the area where the diversion is proposed. If that occurs, sea turtles may be exposed to construction noise. Data on sea turtle hearing sensitivity is currently limited (Popper et al. 2012). Table 5.4.6-1 describes underwater noise thresholds identified for sea turtles. Broadly speaking, turtle hearing sensitivity seems to be greatest at frequencies between 100 and 600 Hz, with a potential hearing range of 50 to 1200 Hz across sea turtle species in general. Popper et al. (2012) suggest SPL exposure thresholds of 207 dB_{peak} or 210 dB_{SEL} would likely protect sea turtles from physical injury. Blackstock et al. (2017) proposed a behavioral effects threshold of 175 dB_{RMS} for impulsive sounds based on observed avoidance behavior during underwater airgun blasts. NOAA Greater Atlantic Regional Fisheries Office (GARFO) uses similar guidelines for sea turtle underwater noise thresholds (see Table 5.4.6-2 below). Impact pile driving (i.e., impulsive sound) associated with the project is planned to occur in the Mississippi River. Sea turtles do not occur in that portion of the Project area. Impact pile driving in Barataria Basin is limited to 12-inch-diameter timber piling. In addition, vibratory pile driving is proposed for sheet pile and H-pile associated with cofferdam construction. Areas of potential behavioral disturbance related to pile driving are limited to an area of about 804 acres where underwater noise may exceed background conditions due to vibratory or impact pile driving within the Barataria Basin (Section 5.3.8); however, although produced noise would exceed ambient conditions in a larger area, exceedance of the behavioral threshold for sea turtles would not occur related to pile driving activities. Pile driving noise levels for planned activities are shown in Table 5.4.6-3. Applicable noise thresholds are not predicted to be exceeded by pile driving construction activities in Barataria Basin based on data provided in the NOAA GARFO Acoustics Tool (NOAA 2020c).

While the use of BMPs and other sea turtle protection measures (described in Section 2.3.6) would effectively minimize the risk of exposure to injury-level effects, in the highly unlikely event that individual turtles are found within 15 feet of dredging activities, potential for behavioral impacts do exist, but their likelihood is so low that they are considered discountable. The potential range of noise for dredging activities are described in Table 5.4.6-3, with only the upper range of possible noise from dredging activities presented having the potential to exceed sea turtle behavior thresholds at 15 feet (4.6 meters). However, this distance is expected to result in no effect because other conservation measures are expected to prevent pile driving operations if sea turtles are within 50-feet of the operation.

With the exception of the behavioral threshold noted above, applicable noise thresholds for sea turtles are not predicted to be exceeded by the Project based on data provided in the NOAA GARFO Acoustics Tool (NOAA 2020c).

Table 5.4.6-2. Guidance on Marine Reptile Underwater Noise Thresholds

Functional Hearing Group	Noise Thresholds			
	dB_{peak}	Weighted dB SEL	dB_{RMS}	Citation
Sea Turtles (general)	207	210	175	1,2
Sea Turtles – Behavioral	-	-	175	3
Sea Turtles – Temporary Threshold Shift	226	189	-	3
Sea Turtles – Permanent Threshold Shift	232	204	-	3

dB = decibel
SEL = sound exposure level = 1 dB re 1 μPa2 -sec
RMS = For pile driving, this is the square root of the mean square of a single pile driving impulse pressure event.
Source: 1. Popper et al. 2014, 2. Blackstock et al. 2017, 3. NOAA GARFO 2019

Table 5.4.6-3. Predicted Underwater Noise Levels During Construction

Activity	Observed Noise Levels				Distance to Sea Turtle Weighted SEL Threshold (Temporary Threshold Shift)
	dB_{peak}	dB SEL	dB_{RMS}	Citation	
Vibratory sheet pile driving	-- @ 10 m	165	165	2, 6	NE
Vibratory H-pile driving	-- @ 10 m	160	137	1, 2, 6	NE
Impact pile driving – 12-inch timber	180 @ 10 m	160	170	2, 6	NE
Dredging	172-185* @ 1 m		172-185*	7, 8	15 feet
Dredge Vessels	175 @ 1 m		175	3, 5, 8	NE

dB = decibel
SEL = sound exposure level = 1 dB re 1 μPa2 -sec
RMS = For pile driving, this is the square root of the mean square of a single pile driving impulse pressure event.
* The specific method of dredging has not yet been determined. Therefore, dredging source levels for cutterhead dredge activities are the most conservative option and the estimated noise range identified as appropriate for the Project area habitat has been presented (172 - 185 dB re 1uPa rms) (CEDA 2011)
NE = no effect, does not exceed threshold
Citations: 1. Illingworth and Rodkin 2007; 2. WSDOT 2019, 3. de Jong et al. 2010, 4. Dickerson et al. 2001, 5. Reine et al. 2012, 6. CalTrans 2015, 7. Blue Planet Marine 2013 (as cited in Jones et al. 2015) 8. CEDA 2011

Project Construction: Vessel and In-Water Construction Interactions

Vessels supporting construction activities may encounter sea turtles in material transport routes, dredging areas, and construction areas. Vessels operating in these areas will follow NMFS Vessel Strike Avoidance Measures and Reporting for Mariners (NMFS 2008) and Sea Turtle and Smalltooth Sawfish Construction Conditions (NMFS 2006) to limit the potential for adverse interactions with sea turtles. Sea turtle protection measures are described in section 3.3.6.

These conservation measures are expected to be protective for sea turtles that may occur in the areas of vessel operations and in-water construction for MBSD.

Project Operation: Upland

Loggerhead sea turtles are the only species documented to nest within the proposed action area. Nesting for this species has occurred on Grand Isle (LDWF 2016) and could occur on other barrier islands with similar shore conditions. Upland areas of the barrier islands are not predicted to be impacted by the proposed Project; therefore, Project operations would not impact loggerhead nesting within the action area. Other species of sea turtles do not nest within the proposed action area, and would, therefore, not be using upland areas (see Section 4.6).

Project Operation: Aquatic

Temperature

Project-related changes to water temperature may affect sea turtles that occur within the action area. Operation of the project is anticipated to decrease water temperatures within the mid and lower basin during peak flows (see Section 5.3.2). During winter months, the Project is anticipated to extend the amount of time that temperatures drop below the minimum temperature tolerance for sea turtles (10°C) (Schwartz 1978) adjacent to the outfall, and immediately south of the outfall in the northern mid-basin (i.e. within about 10 kilometers of the outfall). The water quality sites where notable reductions in predicted temperature occur are at the outfall and just east of Wilkinson Canal about 6 km south of the outfall. These sites are predicted to reduce water temperatures by up to 6.5°C, with temperatures staying below 10°C during January and February of each year as a result of the project. Sites farther away from the outfall would experience minimal effects on temperature, typically seeing reductions in winter temperatures of less than 0.5°C.

Model results (Sadid et al. 2018) indicate that temperatures may be below sea turtle tolerances due to the Project in December through February in years 2020-2039, but after 2039 potentially harmful temperatures below 10°C would only occur during January and February. Cold-shock injury to sea turtles is associated with rapid onset or encounters of low temperature conditions. Potentially injurious low temperature conditions caused by the Project would occur when turtles tend to already be at wintering sites and are therefore unlikely to be within Barataria Basin or the portion of Barataria Basin during these conditions. Temperatures throughout the basin are not expected to exceed the upper range of sea turtle tolerances (33-34°C) (Valverde and Holzwart 2017) in the FWP or FWOP. Therefore, the minor seasonal restriction due to low temperatures in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect sea turtles due to their limited use of this area.

Salinity

Sea turtles use salt glands to excrete excess salts and obtain water by drinking salt water (Kooistra and Evans 1976). When sea turtles are exposed to freshwater the rate of water consumption increases causing a reduction in plasma osmolality and electrolytes (Ortiz et al.

2000). However, acute short-term exposure to freshwater does not appear to create a stress response (Ortiz et al. 2000). While prolonged exposure to freshwater would diminish turtle osmoregulatory capacity, short term (~4 days) exposures appear to be inconsequential (Ortiz et al. 2000). The Project is expected to reduce salinities with the greatest reductions occurring during periods of peak Mississippi River and MBSD flow, typically between December and July of each year (as described in Section 6.3.1). Sea turtles can tolerate low salinities for periods of time; however, it is unclear how low salinities may affect sea turtle behavior. Sea turtles may continue to use low salinity areas for foraging or predator refuge while experiencing physiological effects. Salinity changes due to the Project may concentrate sea turtle activity in the lower portion of Barataria Basin.

Aquatic Vegetation

Project effects to aquatic vegetation may affect sea turtles that occur within the action area. Sea turtles have been observed foraging on SAV around the barrier islands along jetties and other hard substrates (K. Hart, USGS Research Ecologist, pers. com. 2019). The Project is not anticipated to affect the area of remaining emergent marsh or SAV vegetation along the inside of the barrier islands in the lower basin by 2070 as compared to the FWOP (see Section 5.3.10 above). Overall, SAV is predicted to be about 2% higher throughout the Barataria Basin and Birdfoot Delta due to the project by the end of the project.

While composition of SAV in the mid-basin would shift from more saline species to more intermediate species due to the project, SAV near the barrier islands are projected to be minimally impacted. Overall, sea turtles may, but are not likely to, experience benefits from the ~2% basin-wide increase in SAV due to the project (see Figure 5.3.10-1).

Harmful Algal Blooms

Sea turtles are susceptible to brevetoxins associated with blooms of *Karenia brevis*, a dinoflagellate responsible for "Florida Red Tide" (Magaña et al. 2003). Sea turtles are exposed to the toxin by eating affected forage items or aerosolized toxins during blooms of *K. brevis*. Turtle stranding rates are especially high during red tides, and necropsies have indicated the presence of brevetoxins in dead turtles. *K. brevis* is present throughout the Gulf of Mexico; however, blooms are typically associated with temperatures between 22 and 28^{o} C in higher salinity waters along the gulf shelf (Magaña et al. 2003). At least 1 red-tide event causing the closure of shellfish harvest was observed in lower salinity waters (<24 psu) in Louisiana in 1996 (Dortch et al. 1998 as cited in Magaña et al. 2003). No blooms of *K. brevis* have occurred in Louisiana since 1996, and that bloom was associated with a special set of conditions where a tropical storm tracked westward during a bloom along the Florida panhandle (Brown et al. 2006). Although the addition of nutrients in Barataria Basin from the Mississippi River has the potential to increase algal blooms in the Basin, the Project is not anticipated to affect the timing or distribution of *K. brevis* blooms, and therefore not expected to contribute to sea turtle mortalities as a result of harmful algal blooms. The Barataria Basin portion of the action area is currently and will become fresher as a result of the Project. Furthermore, reduced water temperature as a result of the Project may inhibit or delay *K. brevis* blooms within the basin.

Fishing Interactions

High numbers of stranded Kemp's ridley and loggerhead sea turtles in the mid-1980s prompted regulations to require turtle excluder devices (TEDs) on shrimping vessels to prevent turtles caught in fishing gear from drowning. In fisheries such as skimmer trawls and push-head trawls, where TEDs have not been historically used, tow time limitations have been used as alternative conservation measure to reduce the likelihood of drowning turtles. NOAA is in the process of re-evaluating this conservation measure and may require TEDs in the future (NOAA 2019). Despite management interventions, interactions between turtles and shrimping activity continue to contribute to strandings of sea turtles (Lewison et al. 2003).

The Project is anticipated to reduce habitat suitability for brown shrimp populations; this may affect commercial fishing activity in Barataria Basin. Over time the fishing effort targeting brown shrimp is expected to decline throughout Barataria Basin; however, fishing effort may shift in location or intensity over time. While commercially harvestable populations of brown shrimp are projected to decline, basin conditions for white shrimp, another commercially harvested species, are predicted to be unchanged or improve slightly as a result of the Project. However, there may be minor decreases in overall shrimp fishing efforts as a result of the Project. The areas and timing of fishing effort may shift as a result of these changes in the shrimp populations. Fishing effort may shift towards the lower basin or offshore. These changes may create or increase interactions between fishing effort and sea turtles in these areas.

Populations of some species of sea turtles have been increasing since conservation measures were instituted in the past several decades. This increases the likelihood of turtle interactions with shrimping vessels and resulting strandings (Lewison et al. 2003). The Project may contribute to changes in fishing effort and turtle abundance that may lead to more activity in the lower portions of Barataria Basin. Fishing effort is predicted to decrease for brown shrimp and may increase for white shrimp due to the project. These changes will shift the timing of fishing effort. Furthermore, some fishers may venture further to target shrimp populations and may extend fishing to areas offshore. This may lead to increased interactions over time, and while the TEDs have a relative high effectiveness rate for releasing turtles alive, there would still be an increased number of adverse interactions and mortality events.

5.4.7 Project Effects on Green Sea Turtle

Table 5.4.7-1 lists tolerances of the green sea turtle relative to aquatic Project effects.

Table 5.4.7-1. Species Tolerances Relevant to the Project – Green Sea Turtle

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project (Tolerance (optimal))
		Water Temperature (°C)
Green Sea Turtle	juveniles adults	10-34 (22-33) 10-34 (22-33)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
Sources: Witt 2007, Valverde and Holzwart 2017

Project Construction and Operation: Upland

Green sea turtles do not nest within the proposed action area, and would, therefore, not be using upland areas (see Section 4.6).

Project Construction: Aquatic

Underwater Sound

Green sea turtles are not likely to be affected by Project underwater sound effects because sound effects are limited to about 804 acres in an area near the proposed dredging and vibratory pile installation (Section 6.3.8) for the MBSD outfall transition features. This habitat is not near SAV forage resources for green sea turtles.

Project Operation: Aquatic

Temperature

Temperatures are predicted to decrease in areas near the outfall as cooler water from the Mississippi enters Barataria Basin through the outfall. Minimum temperatures will decrease, and the duration of temperatures below green sea turtle minimum temperature tolerances of 10°C will increase by about 1 month and extend later in the season. Temperatures throughout the basin are not expected to exceed the upper range of green sea turtle tolerances (33°C) in the FWP or FWOP. This minor seasonal restriction in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect green sea turtles, and is considered negligible.

Aquatic Vegetation

Project related effects to aquatic vegetation may affect green sea turtles within the action area. Green sea turtles have been observed foraging on SAV in the vicinity of the barrier islands along jetties and other hard substrates (K. Hart, USGS Research Ecologist, pers. com. 2019). The Project is not anticipated to affect the area of remaining emergent marsh vegetation or SAV vegetation along the inside of the barrier islands in the lower basin by 2070 as compared to the FWOP (see Section 5.3.10 above). Overall, SAV is predicted to be higher throughout the Barataria Basin and Birdfoot Delta due to the Project.

While composition of SAV in the mid-basin would shift from more saline species to more intermediate species due to the project, SAV near the barrier islands are projected to be minimally impacted. Overall, green sea turtles may, but are not likely to, experience benefits from the ~2% basin-wide increase in SAV due to the project (see Figure 5.3.10-1).

Fisheries Interactions

Changes in local shrimp populations due to the Project (including a decrease in the brown shrimp population and a negligible to minor increase in the white shrimp population) may result in spatial changes to the shrimp fishery efforts in the Project area (see Section 5.3.11). If these changes result in shrimp fishers focusing on locations lower in the basin or in nearshore/offshore waters near the barrier islands, where more green sea turtles would be present, it may increase the potential for interactions between fishers and sea turtles, which is a primary threat to sea turtles. Increased interactions could increase the rate of injury and mortality to green sea turtles present in the Project area. Overall, green sea turtles are likely to be adversely affected by the project, and minor to moderate adverse impacts are possible due to the Project's impacts on increased shrimp fisheries interactions.

5.4.8 Project Effects on Hawksbill Sea Turtle

Table 5.4.8-1 lists tolerances of the hawksbill sea turtle relative to aquatic Project effects.

Table 5.4.8-1. Species Tolerances Relevant to the Project – Hawksbill Sea Turtle

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project (Tolerance (optimal))
		Water Temperature (°C)
Hawksbill Sea Turtle	juveniles adults	10-33 (10-21) 10-33 (10-21)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
Sources: Witt 2007, Valverde and Holzwart 2017

Project Construction and Operation: Upland

Hawksbill sea turtles do not nest within the proposed action area, and therefore, are not expected to use upland areas (see Section 4.6).

Project Construction: Aquatic

Underwater Noise

Hawksbill sea turtles are not likely to be affected by Project-related underwater noise, because they have not been documented inside of Barataria Basin. Hawksbill sea turtles have only been documented in nearshore waters beyond the barrier islands, outside of the action area.

Project Operation: Aquatic

Temperature

Temperatures are predicted to decrease in areas near the outfall as cooler water from the Mississippi enters Barataria Basin through the outfall. Minimum temperatures will decrease, and the duration of temperatures below hawksbill sea turtle minimum temperature tolerances of 10°C will increase by about 1 month and extend later in the season. Hawksbill sea turtles are not likely to be affected by Project-related temperature effects because they do not typically occur in the action area. Temperatures throughout the basin are not expected to exceed the upper range of Hawksbill sea turtle tolerances (33°C) in the FWP or FWOP. This minor seasonal restriction in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect Hawksbill sea turtles, and is considered negligible.

Aquatic Vegetation

Hawksbill sea turtles are not likely to be affected by Project aquatic vegetation or SAV effects, as they have only been documented outside of the barrier islands and Birdfoot Delta, a portion of the action area where MBSD Project effects are discountable.

5.4.9 Project Effects on Kemp's Ridley Sea Turtle

Table 5.4.9-1 lists tolerances of the Kemp's ridley sea turtle relative to aquatic Project effects.

Table 5.4.9-1. Species Tolerances Relevant to the Project – Kemp's Ridley Sea Turtle

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project (Tolerance (optimal))
		Water Temperature (ºC)
Kemp's ridley Sea Turtle	juveniles adults	10-34 (20-34) 10-34 (20-34)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
Sources: Witt 2007, Valverde and Holzwart 2017

Project Construction and Operation: Upland

Kemp's ridley sea turtles do not nest within the proposed action area, and therefore, would not be expected to use upland areas (see Section 4.6).

Project Construction: Aquatic

Underwater Sound

Kemp's ridley sea turtles are not likely to be adversely affected by Project underwater sound effects because sound effects are limited to approximately 804 acres in an area near the proposed dredging and vibratory pile installation (section 5.3.8) for the MBSD outfall transition features.

Project Operation: Aquatic

Temperature

Sub-adult and juvenile Kemp's ridley sea turtles have been encountered in Barataria Bay, and are abundant south of the barrier islands (Section 4.6.8). Temperatures are predicted to decrease in areas near the outfall as cooler water from the Mississippi enters Barataria Basin through the outfall. Minimum temperatures will decrease, and the duration of temperatures below Kemp's ridley sea turtle minimum temperature tolerances of 10°C will increase by about 1 month and extend later in the season. Temperatures throughout the basin are not expected to exceed the upper range of Kemp's ridley sea turtle tolerances (34°C) in modeling predictions with or without the Project. This minor seasonal restriction in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect Kemp's ridley sea turtles, and is considered negligible.

Prey Items

Prey studies suggest that Kemp's ridley sea turtles are opportunistic predators that consume a wide variety of prey items from the seabed including crabs, tunicates, mollusks, vegetation and fish. Crabs, in particular blue crabs, are the primary prey items for this species across its range (Witzell and Schmid 2005). Predicted habitat suitability was evaluated for 3 likely prey items: brown shrimp, white shrimp and blue crab. Brown shrimp habitat suitability is predicted to decline as a result of the Project; however, habitat suitability for blue crab and white shrimp is predicted to remain neutral or improve as a result of the project (LA TIG 2019). Reductions in brown shrimp habitat suitability are associated with reduced salinities during early developmental stages, while potentially improved habitat suitability for white shrimp and blue crab are due to increased quantities of wetland habitat and better food web support. Overall, these findings suggest that Kemp's ridley prey items will continue to be available, however shifts in the abundance of some prey items may cause seasonal or regional reductions in some prey items (e.g., brown shrimp) and no change or potential increases in others (e.g., blue crab).

Fisheries Interactions

Changes in local shrimp populations due to the Project (including a decrease in the brown shrimp population and a negligible to minor increase in the white shrimp population) may result in spatial changes to the shrimp fishery efforts in the Project area (see Section 5.3.11). If these changes result in shrimp fishers focusing on locations lower in the basin or in nearshore/offshore waters near the barrier islands, where more Kemp's ridley turtles would be present, it may increase the potential for interactions between fishers and sea turtles, which is a primary threat to sea turtles. Increased interactions could increase the rate of injury and mortality to Kemp's ridley sea turtles present in the Project area. Overall, Kemp's ridley sea turtles are likely to be adversely affected by the project, and minor to moderate adverse impacts are possible due to the Project's impacts on increased shrimp fisheries interactions.

5.4.10 Project Effects on Leatherback Sea Turtle

Table 5.4.10-1 lists tolerances of the leatherback sea turtle relative to aquatic Project effects.

Table 5.4.10-1. Species Tolerances Relevant to the Project – Leatherback Sea Turtle

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project (Tolerance (optimal))
		Water Temperature (°C)
Leatherback Sea Turtle	juveniles adults	25-38 (26-38) 5-38 (10-22)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
Sources: Witt 2007, Valverde and Holzwart 2017

Project Construction and Operation: Upland

Leatherback sea turtles do not nest within the proposed action area, and therefore, would not be expected to use upland areas (see Section 4.6).

Project Construction: Aquatic

Underwater Noise

Leatherback sea turtles are not likely to be affected by Project underwater sound effects because sound effects are limited to approximately 804 acres in an area near the proposed dredging and vibratory pile installation (section 6.3.8) for the MBSD outfall transition features. Underwater noise will be created only near the diversion structure, more than 40 kilometers from the southern edge of the action area. Leatherback sea turtles have only been documented outside of the barrier islands, a portion of the action area where project effects are insignificant.

Project Operation: Aquatic

Temperature

Leatherback sea turtles are not likely to be affected by Project temperature effects, as they've only been documented outside of the barrier islands, in a portion of the action area where project effects are insignificant. The minor seasonal restriction in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect leatherback sea turtles, as they have not been documented there, and is considered negligible.

5.4.11 Project Effects on Loggerhead Sea Turtle and Loggerhead Sea Turtle Critical Habitat

Table 5.4.11-1 lists tolerances of the loggerhead sea turtle relative to aquatic Project effects.

Table 5.4.11-1. Species Tolerances Relevant to the Project – Loggerhead Sea Turtle

ESA Listed Species	Life Stage*	Species Tolerances and Interactions with Potential Effects of Proposed Project (Tolerance (optimal))
		Water Temperature (ºC)
Loggerhead Sea Turtle	hatchlings juveniles adults	10-33 (20-33) 10-33 (20-33) 10-33 (20-33)

*Only showing life stages reported to occur within the action area.
Tolerance Range = Identifies the range where the organism is able to survive in natural or laboratory settings.
Optimal Range = Identifies the range where the organism is not experiencing significant stress, and where maximal growth, abundance, or activity occurs.
Sources: Witt 2007, Valverde and Holzwart 2017

Project Construction: Upland

Loggerhead sea turtles rarely nest within the proposed action area, and, therefore, are not expected to use upland areas within the construction footprint. Two records of adult female loggerhead sea turtles nesting on Grand Isle on June 29 and July 3, 2015 represent the first confirmed sea turtle nesting on the coast of Louisiana for 30 years (Louisiana Sportsman 2015). Since sea turtles typically return to their natal beaches it is possible that future nesting activity could occur near Grand Isle, far from construction activities. Upland habitats in the barrier island area are not expected to experience effects due to the Project construction.

Project Construction: Aquatic

Underwater Noise

Loggerhead sea turtles are not likely to be affected by Project underwater sound effects because sound effects are limited to approximately 804 acres in an area near the proposed dredging and vibratory pile installation (Section 5.3.8) for the MBSD outfall transition features. Turtles are expected to be at low abundances or absent from these areas as they do not represent high quality foraging habitat and are distant from documented sea turtle observations.

Project Operation: Upland

As noted above, loggerhead sea turtles rarely nest within the proposed action area, It is possible that future nesting activity could occur near Grand Isle. Upland habitats in the barrier island area are not expected to experience effects due to the Project (see Section 4.6).

Project Operation: Aquatic

Temperature

Temperatures are predicted to decrease in areas near the outfall as cooler water from the Mississippi enters Barataria Basin through the outfall. Minimum temperatures will decrease, and the duration of temperatures below loggerhead sea turtle minimum temperature tolerances of 10°C will increase by about 1 month and extend later in the season. Temperatures throughout the basin are not expected to exceed the upper range of loggerhead sea turtle tolerances (33°C) in the FWP or FWOP. This minor seasonal restriction in the upper portion of sea turtle range in Barataria Basin is not likely to negatively affect loggerhead sea turtles, and is considered negligible.

Prey Items

Prey studies suggest that loggerhead sea turtles are opportunistic predators that consume mostly bottom-dwelling invertebrates including mollusks and crabs (Valverde 2017). Predicted habitat suitability was evaluated for 3 likely prey items: brown shrimp, white shrimp and blue crab. Brown shrimp habitat suitability is predicted to decline as a result of the Project; however, habitat suitability for blue crab and white shrimp is predicted to remain neutral or improve as a result of the project (LA TIG 2019). Reductions in brown shrimp habitat suitability are associated with reduced salinities during early developmental stages, while potentially improved habitat suitability for white shrimp and blue crab are due to increased quantities of wetland habitat and better food web support. Overall, these findings suggest that loggerhead prey items will continue to be available, however shifts in the abundance of some prey items may cause seasonal or regional reductions in some prey items (e.g., brown shrimp) and no change or potential increases in others (e.g., blue crab).

Fisheries Interactions

Changes in local shrimp populations due to the Project (including a decrease in the brown shrimp population and a negligible to minor increase in the white shrimp population) may result in spatial changes to the shrimp fishery efforts in the Project area (see Section 5.3.11). If these changes result in shrimp fishers focusing on locations lower in the basin or in nearshore/offshore waters near the barrier islands, where more loggerhead sea turtles would be present, it may increase the potential for interactions between fishers and sea turtles, which is a primary threat to sea turtles. Increased interactions could increase the rate of injury and mortality to loggerhead sea turtles present in the Project area. Overall, loggerhead sea turtles are likely to be adversely affected by the project, and minor to moderate adverse impacts are possible due to the Project's impacts on increased shrimp fisheries interactions.

5.5 Effects from Interdependent and Interrelated Actions

Interrelated actions include those that are part of a larger action and depend on the larger action for justification. Interdependent actions are those with no independent utility apart from the proposed action. There are no interdependent or interrelated actions identified for this project.

5.6 Cumulative Effects

Cumulative effects, with respect to ESA, are those effects arising from local, state, tribal, or private activities that are reasonably certain to occur within the area of the federal action subject to consultation (50 CFR 402.02 Definitions). Cumulative effects under ESA do not include other federal actions occurring in the action area or projects requiring federal permits. Federal actions and project requiring federal permits unrelated to the proposed action are not considered in this section because they require separate consultation pursuant to Section 7 of the ESA. Activities in the area occurring in aquatic and wetland habitats that will require and be subject to federal permitting and will not be included in the cumulative effects analysis. Potential pathways of cumulative effects are described in the following sections.

5.6.1 Past, Present, and Ongoing Actions and Trends

The following past, present, and ongoing actions and trends were identified as impacting the Project area resources and were included in baseline of the analysis of project impacts. They are described below:

- **Levees and channelization of the Mississippi River**: These actions have caused major, adverse, permanent impacts on the Barataria Basin by altering natural sediment transport from the river into the basin, removing the source of sediment and fresh water that built and maintained wetlands and marshes. As a result, the basin is suffering from significant coastal habitat loss (USGS 2015, CPRA 2012). Without the Project, this reduced input of sediment due to Mississippi River levees would continue to cause major wetland loss in the Barataria Basin.

- **Subsidence and sea-level rise**: These ongoing trends continue to be a primary cause of major, adverse, permanent impacts on Barataria Basin wetland and land loss by increasing flooding frequency and duration, marsh vegetation break-up, and erosion (BTNEP 2010, Couvillion et al. 2017). Subsidence and sea-level rise were factored into the baseline conditions and Project alternatives over the 50-year period of analysis for all resources. The SLR value simulated for all model runs was an increase of 2.2 feet (0.7 meter) by 2070 compared to year 2020 sea levels, or 4.9 feet (1.5 meters) by year 2100.

- **Storm and hurricane events**: These ongoing major, adverse events will continue to cause loss of life, major economic damages, and outmigration of residents and businesses. They also convert wetlands to open water from erosion when large storm surges bring salt water inland (Day et al. 2007). Results of ADCIRC/SWAN modeling of storm surge and wave height elevation simulations over the 50-year analysis period are included as past and present projects in the Delft3D Basinwide Model.

- **Canals dredged in the Barataria Basin for navigation and oil and gas development**: Canals and channels in the basin provide a conduit for saltwater intrusion and obstruct the natural hydrology and sheetflow of water across and through marsh, causing marsh loss and impoundment (Cowan et al. 1988 from Boesch et al. 1994, Swenson and Turner 1987).

- **2010 DWH oil spill**: This major disaster was the direct cause of a minimum of 850 miles of shoreline oiling in coastal Louisiana, with the most widespread oiling occurring in Barataria Bay salt marshes (DWHNRDAT 2016). The consequences of the spill included major adverse impacts on aquatic resources, including marsh vegetation, intertidal biota (for example, fiddler crabs), and shoreline erosion (Zengel et al. 2015). This catastrophic event is the basis of the purpose and need of the MBSD Project which is to help restore habitat and ecosystem services injured by the DWH oil spill. The impacts of the DWH oil spill are captured in the baseline conditions of the Project area.

- **Shoreline and marsh restoration projects**: The Delft3D Basinwide Model incorporates past or recently completed restoration projects into the baseline conditions of Project-area topography, bed elevations, hydrology, water quality, and wetland conditions.

- **Rivers and diversions**: Within the Delft3D Basinwide Model, numerous rivers are applied at the model boundary. The rivers carry fresh water, sediments, and nutrients into the model domain. Additionally, the model incorporates the impacts of the following natural and man-made diversions that allow Mississippi River water to leave: the Davis Pond Freshwater Diversion (see more information about this diversion below), the Bonnet Carré Spillway, the Caernarvon Freshwater Diversion, Mardi Gras Pass, the West Point A La Hache Siphon, and various passes in the Birdfoot Delta. Ongoing operations and influences of rivers and diversions were incorporated into the Delft 3D Basinwide Model baseline conditions and 50-year projections for hydrology, hydrodynamics, water quality, vegetation/wetlands, and other resources in the Project area.

- **Davis Pond Freshwater Diversion**: As described above, ongoing operations and influences of this diversion were incorporated into the Delft 3D Basinwide Model baseline conditions and 50-year projections for the MBSD and the FWOP. This diversion operates at a minimum of 1,000 cfs flow with the capacity to divert up to 10,650 cfs of water from the Mississippi River at RM 118 ABH (approximately 15 miles upriver from

New Orleans). The diversion introduces fresh water, sediments, and nutrients into the marshes of the northern Barataria Basin (USACE 2018).

5.6.2 Reasonably Foreseeable Future Projects

The following 3 projects are reasonably certain to occur within the timeframe and general area of the proposed Project and will not involve or require federal permits or actions. These 3 projects are considered in the cumulative effects analysis and are described below.

These included the following types of projects:

- municipal;
- major industrial development; and
- recreation.

Table 5.6.2-1 lists each project considered in the cumulative effects analysis, its distance from the MBSD Project, and the resources that each would potentially impact. None of these 3 reasonably foreseeable projects assessed were incorporated into the Delft3D Basinwide Model, as they all occur upland and would not impact areas large enough to be captured in the Delft3D Basinwide Model resolution.

Table 5.6.2-1. Reasonably Foreseeable Future Projects Considered in the Cumulative Effects Analysis

Project Name/ Proponent	Project Type	Closest Distance to Project Location	Description and Status	Estimated Construction Timing	Resources with Potential Cumulative Effects
Braithwaite Methanol Plant/CCI Port Nickel LLC	Major Industrial	13.0 miles	Methanol manufacturing facility with 5,000-metric ton daily production capacity (1.8 million tons per annum) of feedstock natural gas from an unspecified connection. The schedule for the construction is unknown. Air permit received from LDEQ in December 2019.	2020 – 2023	Commercial Fisheries
Bayou Segnette State Park Improvements/ CPRA	Recreational Use	19.1 miles	Infrastructure improvements in Bayou Segnette State Park in Jefferson Parish, including upgrades to an existing boating area to improve access, upgrades to a playground to comply with ADA requirements, and repairs to road and parking areas damaged by repeated flooding.	2020	Commercial Fisheries
Pumping Capacity Improvements Phase I/ LDEQ/CPRA & Fresh Water District	Municipal	67.5 miles	Construction of a pump station on the Mississippi River at Donaldsonville in Ascension Parish with a minimum pumping capacity of 1,000 cfs alongside the existing 500-cfs pump station, thereby tripling the capacity for fresh water entering Bayou Lafourche to combat saltwater intrusion and provide fresh drinking water to over 300,000 residents in Assumption, Ascension, Lafourche, and Terrebonne Parishes.	Unavailable	Aquatic Resources, ESA Species; Commercial Fisheries, Marine Mammals

5.6.3 Potential Cumulative Effects on Each Resource

None of the foreseeable projects have the potential to contribute to cumulative effects on listed species and designated critical habitat within the Project action area; therefore, there are no anticipated cumulative effects during construction or operations of the MBSD Project. The potential projects and impacts pathways for cumulative effects are described below.

Surface Water and Coastal Processes

The foreseeable projects would not add cumulative effects on bed elevations, water levels, tides, currents, flow, and sediment transport in the Mississippi River portion of the action area during construction or operations of the MBSD Project.

Because no reasonably foreseeable projects overlap with the Project action area in the Barataria Basin, there would be no cumulative effects on hydrology and hydrodynamics of the Basin during construction or operations of the MBSD Project.

Wetland Resources

None of the foreseeable projects overlap with wetlands or the Project action area in the Barataria Basin; therefore, there are no anticipated cumulative effects to wetland resources during construction or operations of the MBSD Project.

Noise

Airborne Noise

If construction of the reasonably foreseeable projects planned in the action area were to occur at the same time as construction of the MBSD Project, concurrent construction would result in temporary increases in noise where sound from more than 1 project overlaps at nearby noise sensitive areas. As the nearest foreseeable project is over 13 miles away from the action area, there are no anticipated cumulative effects from airborne noise levels during construction or operations of the MBSD Project.

Underwater Noise

None of the foreseeable projects have the potential to contribute to underwater noise during the same construction period in the Mississippi River or Barataria Basin areas of the Project action area; therefore, there are no anticipated cumulative effects from underwater noise levels during construction or operations of the MBSD Project.

Terrestrial Wildlife and Habitat

None of the foreseeable projects have the potential to contribute to cumulative effects to terrestrial wildlife and habitat within the upland areas of the Project action area; therefore, there are no anticipated cumulative effects to terrestrial wildlife habitat during construction or operations of the MBSD Project.

Aquatic Resources

Potential impacts from the upland foreseeable project of the Pumping Capacity Improvements Phase I/LDEQ/CPRA & Fresh Water District are expected to be minor and highly localized, and would not contribute to cumulative effects to aquatic resources during construction or operations of the MBSD Project.

Commercial Fisheries

None of the foreseeable projects have the potential to contribute to cumulative effects to commercial fisheries in the Mississippi River or Barataria Basin areas of the Project action area. The 3 upland foreseeable projects are small enough in scale and spread out enough that they are not likely to create traffic disruptions, or disrupt commercial fishing activities.

Potential minor positive impacts by the Bayou Segnette State Park Improvements/CPRA are possible through increased water access as a result of the project, but the impact to commercial fisheries within the Project action area would be discountable. Potential impacts from the upland foreseeable project of the Pumping Capacity Improvements Phase I/LDEQ/CPRA & Fresh Water District are expected to be minor and highly localized, and would not contribute to cumulative effects to aquatic resources or commercial fisheries during construction or operations of the MBSD Project. Therefore, there are no anticipated cumulative effects from underwater noise levels during construction or operations of the MBSD Project.

Marine Mammals

Potential impacts from the upland foreseeable project of the Pumping Capacity Improvements Phase I/LDEQ/CPRA & Fresh Water District are expected to be minor and highly localized, and would not contribute to cumulative effects to marine mammals during construction or operations of the MBSD Project.

ESA Species

Riverine Species (Pallid Sturgeon)

Potential impacts from the upland foreseeable project of the Pumping Capacity Improvements Phase I/LDEQ/CPRA & Fresh Water District are expected to be minor and highly localized, and would not contribute to cumulative effects to riverine species in the Mississippi River during construction or operations of the MBSD Project.

Terrestrial Species (Eastern Black Rail, Piping Plover, and Red Knot)

None of the foreseeable projects have the potential to contribute to cumulative effects to terrestrial wildlife and habitat within the upland areas of the Project action area; therefore, there are no anticipated cumulative effects to terrestrial wildlife habitat during construction or operations of the MBSD Project.

Marine/Estuarine Species (West Indian Manatee, Sea Turtles)

Potential impacts from the upland foreseeable project of the Pumping Capacity Improvements Phase I/LDEQ/CPRA & Fresh Water District are expected to be minor and highly localized, and would not contribute to cumulative effects to marine or estuarine species in Barataria Basin during construction or operations of the MBSD Project.

6.0 MONITORING AND ADAPTIVE MANAGEMENT

The DWH Natural Resource Damage Assessment Trustees identified implementation of monitoring and adaptive management (MAM) as one of the programmatic goals in the DWH PDARP (DHNRDAT 2016). The MAM for the Project identifies monitoring needs to evaluate progress towards meeting restoration objectives and to inform adaptive management. This includes describing the key performance measures associated with each objective that the LA TIG would use to assess progress toward meeting the restoration objectives as described in the Restoration Plan (RP).

Monitoring would include a combination of baseline (pre-operations) and Project (post-operations) monitoring efforts. These monitoring efforts would facilitate evaluations of trends over time as well as pre- and post-Project effects. The locations, types of data collected, and frequency of post-construction data collection would be reviewed and refined during the Project lifespan to improve operations (e.g., sediment capture from the river) and sediment retention in the basin. Data would be collected and evaluated by CPRA, cooperating State and federal agencies, as well as non-governmental organizations. Data collection would be organized around Project objectives including the following:

(1) Deliver fresh water, sediment, and nutrients to Barataria Basin through a large-scale sediment diversion from the Mississippi River;

(2) Reconnect and re-establish sustainable deltaic processes between the Mississippi River and the Barataria Basin; and

(3) Create, restore, and sustain wetlands and other deltaic habitats and associated ecosystem services.

Additional monitoring may focus on status and trends or compliance with regulatory requirements.

Project-level adaptive management focuses on identifying Project uncertainties and, where feasible, reducing those uncertainties through Project design, research, or monitoring to inform management actions. Modeling is essential to this adaptive management approach as it provides the expectations that justify plan implementation. This is especially important in Louisiana due to the constantly changing baseline (TWI 2013). The adaptive management actions would be identified based on the monitoring data and associated assessments.

7.0 EFFECT DETERMINATIONS

Table 7.0-1, below, summarizes the effects determinations for the ESA listed species in Table 3.1-1 above.

Table 7.0-1. ESA Species Effect Determinations

Listed Species	Status	Effects Determination		Justification
		Species	Critical Habitat	
ESA Listed Fish				
Pallid Sturgeon (*Scaphirhynchus albus*)	E	LAA	NA	▪ Pallid sturgeon within the action area are most likely larval or juvenile. ▪ Most pallid sturgeon lost to entrainment are anticipated to be larval stages. ▪ Low rates of loss of pallid sturgeon larvae and subadults due to entrainment by the Project are not considered to have population-level effects for the species.
ESA Listed Birds				
Eastern Black Rail (*Laterallus jamaicensis jamaicensis*)	PT	NLAA	NA	▪ Eastern black rail are not likely to but may occur in the action area year-round. ▪ Temporary short-term disturbance or displacement from foraging and resting areas during Project construction is possible. ▪ Project operations may have positive long-term effects on marsh habitat in the mid-basin.
Piping plover (*Charadrius melodus*) - Atlantic Coast, Great Lakes, and Northern Great Plains population	T	NLAA	NE	▪ Piping plover are not likely to but may occur in the southernmost portions of the action area. ▪ Critical habitat would not be affected by Project construction activities or operation. ▪ Potential habitat for piping plovers, including sand spits, beaches, sand flts and muflats associated with barrier islands, are not expected to be affected by the project. ▪ Piping plover do not nest within the action area, however may use area during annual migration and wintering periods.
Red knot (*Calidris canutus rufa*)	T	NLAA	NA	▪ Red knot are not likely to but may occur within the action area. ▪ Potential habitat for red knot, including sand spits, beaches, sand flts and muflats associated with barrier islands, are not expected to be affected by the project. ▪ Red knot do not nest within the action area, but may be temporary visitors to the action area during annual migration and wintering periods.
ESA Listed Marine Mammals				
West Indian manatee (*Trichechus manatus*)	T	NLAA	NA	▪ West Indian manatee are occasional or seasonal visitors to the action area. ▪ Construction activities may temporarily disturb or displace manatees within the action area. ▪ BMPs will limit or avoid negative interactions with project construction activities
ESA Listed Sea Turtles				
Green sea turtle (*Chelonia mydas*) - North Atlantic DPS	T	LAA	NA	▪ Limited occurrence in action area ▪ Increased negative interactions with commercial shrimp fishing due to the potential spatial shift in shrimp fishing effort due to the Project.

Listed Species	Status	Effects Determination		Justification
		Species	Critical Habitat	
Hawksbill sea turtle (*Eretmochelys imbricata*)	E	NLAA	NA	▪ Hawksbill sea turtles are not anticipated to occur within the action area. They have been documented in the Gulf of Mexico south of the action area.
Kemp's ridley sea turtle (*Lepidochelys kempii*)	E	LAA	NA	▪ Potential for interactions with dredging or vessel operations during construction. ▪ Potential habitat and prey species within Barataria Basin may be affected by Project operations. ▪ Increased negative interactions with commercial shrimp fishing due to the potential spatial shift in shrimp fishing effort due to the Project.
Leatherback sea turtle (*Dermochelys coriacea*)	E	NLAA	NA	▪ Leatherback sea turtles are not anticipated to occur within the action area. They have been documented in the Gulf of Mexico south of the action area.
Loggerhead sea turtle (*Caretta caretta*), - Northwest Atlantic DPS	T	LAA	NE	▪ Critical habitat would not be affected by Project construction activities or operation. ▪ Occurrence primarily near barrier island with limited effects to prey resources ▪ Increased negative interactions with commercial shrimp fishing due to the potential spatial shift in shrimp fishing effort due to the Project.
Sources: NMFS 2018a, USFWS 2018a Abbreviations: DPS = Distinct Population Segment; E = Endangered; T = Threatened; PT = Proposed Threatened NA = not applicable; LAA = likely to adversely affect; NLAA = may affect, not likely to adversely affect; NE = no effect				

7.1 Pallid Sturgeon

The Project would result in both short- and long-term alterations of pallid sturgeon habitat in the action area. Short-term alterations would occur due to construction of the intake structure and associated temporary and permanent in-water structures in the Mississippi River at RM 60.7. Construction activities would result in conditions that may directly and indirectly affect pallid sturgeon, such as increased turbidity due to substrate disturbance and increased underwater noise due to piling and cofferdam installation.

Other potential stressors to pallid sturgeon from construction activities—including pollutants, benthic disturbance, and physical debris from construction activities—are expected to be insignificant due to construction BMPs.

Operation of the diversion is expected to divert between 330 billion and 1.8 trillion cubic feet of water from the Mississippi River per year. This represents between approximately 3.1% and 7.2% of the annual Mississippi River flow. Pallid sturgeon may become entrained in that flow and diverted into Barataria Basin. Once diverted into the Barataria Basin it is presumed they would be unable to access the Mississippi River and would become functionally segregated from the listed population.

Therefore, the Project *may affect* pallid sturgeon for the following reasons:

- Pallid sturgeon habitat may be temporarily isolated by the cofferdam installed for construction of the intake structure.
- Pile driving and cofferdam installation would increase underwater sound levels.
- Diversion operations would cause fish to become entrained in diverted water and fish entering Barataria Basin would become functionally segregated from the ESA listed population.

The Project is *likely to adversely affect* pallid sturgeon for the following reasons:

- Pallid sturgeon may be present near pile driving activities and experience behavioral avoidance or injury.
- Pallid sturgeon are likely to occur in the action area during diversion operations and may be entrained at a rate similar to the total fraction of Mississippi River water diverted through the MBSD.

7.2 Eastern Black Rail

The Project would result in both short-term and long-term alterations of Eastern black rail habitat within the action area. Short-term alterations would occur due to 2 factors: (1) exclusion from and alteration of available habitat within the limits of construction during Project construction activities; and (2) habitat degradation during initial diversion operations, which are predicted to temporarily inundate vegetation and smother invertebrates immediately adjacent to the diversion during the initial deliveries of sediment from Mississippi River flows.

Long-term alterations of Eastern black rail habitat within the action area would occur as the predominantly brackish marsh would transition to fresh/intermediate marsh within the mid-basin. Black rail utilize both of these marsh types and are not anticipated to be negatively affected by this habitat alteration.

Other potential stressors to Eastern black rails include disturbance events during temporary construction activities. These temporary construction activities may have the following effect: disturbance or displacements of individuals outside of the limits of construction but still within the action area.

Therefore, the Project *may affect* Eastern black rail for the following reasons:

- Construction activities may temporarily disturb or displace Eastern black rail present in marsh habitats near construction activities.

The Project is *not likely to adversely affect* Eastern black rail for the following reasons:

- Eastern black rail within the action area would be present in low densities, mobile, and only exposed to temporary effects associated with Project construction and initial operation.

7.3 Piping Plover and Piping Plover Designated Critical Habitat

Suitable habitat for piping plovers predominantly includes the sand spits, beaches, sand flats, and mudflats associated with barrier islands and Gulf shoreline headlands. These areas are not expected to be affected by Project construction or operations.

The Project would not result in either short-term or long-term alterations of piping plover habitat, and no alterations to piping plover designated critical habitat would occur within the action area.

The Project is *not likely to adversely affect* piping plover for the following reasons:

- Piping plover within the action area would be mobile, and are not likely to occur in the construction area or near Project construction activities.

- Project operations are not likely to change the coastal processes that will continue to influence barrier island morphology.

The Project is anticipated to have *no effect* on piping plover critical habitat for the following reason:

- No effects to beaches, sand spits, sand flats, or mudflats or land building/maintenance are anticipated along the barrier islands or Gulf shoreline headlands as a result of the Project.

7.4 Red Knot

Suitable habitat for red knots predominantly includes sand spits, beaches, sand flats, and mudflats associated with barrier islands and Gulf shoreline headlands. These habitats are not expected to be affected by Project construction or operations.

The Project would not result either short-term or long-term alterations of red knot habitat within the action area.

The Project is *not likely to adversely affect* red knot for the following reasons:

- Red knot within the action area would be mobile, and are not likely to occur in the construction area or near Project construction activities.

- Project operations are not likely to change the coastal processes that will continue to influence barrier island morphology

7.5 West Indian Manatee

The Project would result in short-term and long-term alterations of West Indian manatee habitat within the action area. Short-term alterations would occur due to 2 factors: (1) exclusion from and alteration of available habitat during Project construction activities; and (2) habitat degradation during initial diversion operations, which are predicted to temporarily increase turbidity and inundate vegetation immediately adjacent to the diversion during the initial deliveries of fresh water and sediment from Mississippi River flows.

Long-term alterations of West Indian manatee habitat within the action area would occur as the predominantly brackish marsh would transition to fresh/intermediate marsh within the mid-basin. Manatee utilize both of these marsh types and are not anticipated to be negatively affected by this habitat alteration. The Project would confer large areas of habitat benefits to manatee by adding and preserving marsh habitat and shallow waters within the mid-basin.

Other potential stressors to manatee include disturbance events during temporary construction activities, which may displace resting or foraging individuals near the diversion outfall.

Therefore, the Project *may affect* West Indian manatee for the following reasons:

- Construction activities may temporarily disturb or displace manatees present in marsh habitats at the diversion location, near construction activities.
- Project operations are predicted to reduce water temperatures in Barataria Basin, with the greatest reductions in water temperature occurring during the winter and early spring months and near the outfall site.

The Project is *not likely to adversely affect* West Indian manatee for the following reasons:

- West Indian manatee protection measures identified in Section 3.3.6 are expected to prevent adverse interactions between construction activities and manatees.
- West Indian manatee within the action area during summer months would be mobile, and only exposed to temporary effects associated with Project construction and initial operation.

7.6 Green Sea Turtle

The Project would result in long-term alterations of sea turtle habitat within the action area. Long-term alterations of sea turtle habitat would occur due to minor increases in SAV within the mid-basin and extended periods below sea turtle temperature thresholds during high-flow winter months within the mid-basin and adjacent to the Project diversion.

Other potential stressors to sea turtles include disturbance events during temporary construction activities, which may displace resting or foraging individuals near the diversion outfall. Additionally, spatial shifts in shrimp fishery effort due to the Project may more frequently overlap with sea turtle distributions in the basin, increasing their risk of direct injury or mortality.

Based on tagging and capture data, green sea turtles are likely present in low numbers. Green sea turtles are expected to be present near major passes connecting Barataria Basin to Gulf of Mexico. Based on their occurrence and the potential effects of the project they may be adversely affected by the Project.

Therefore, the Project *may affect* green sea turtle for the following reasons:

- In the unlikely event green sea turtles are present near the diversion during construction activities, they may be disturbed or displaced during foraging activities.
- Sea turtles present within the lower basin or adjacent to the diversion during winter months may be exposed to temperatures below their temperature threshold.
- Salinity changes in Barataria Basin may concentrate sea turtles in the lower portion of Barataria Basin.

The Project is *likely to adversely affect* green sea turtle for the following reasons:

- Commercial fishing effort and sea turtle distribution may co-occur in portions of the Lower Barataria Basin.

7.7 Hawksbill Sea Turtle

The Project would result in long-term alterations of sea turtle habitat within the action area. Long-term alterations of sea turtle habitat would occur due to minor increases in SAV within the mid-basin and extended periods of areas below sea turtle temperature thresholds during high-flow winter months within the lower basin and adjacent to the Project diversion.

Other potential stressors to sea turtles include disturbance events during temporary construction activities, which may displace resting or foraging individuals near the diversion outfall.

However, based on tagging and capture data, hawksbill sea turtles are not likely to occur within the action area and are unlikely to be affected by the Project.

Therefore, the Project *may affect* hawksbill sea turtle for the following reasons:

- In the unlikely event sea turtles are present near the diversion during construction activities, they may be disturbed or displaced during foraging activities.
- Sea turtles present within the lower basin or adjacent to the diversion during winter months may be exposed to temperatures below their temperature threshold.
- Salinity changes in Barataria Basin may concentrate sea turtles in the lower portion of Barataria Basin.

The Project is *not likely to adversely affect* hawksbill sea turtle for the following reason:

- Hawksbill sea turtles have only been documented outside of the barrier islands, a portion of the action area where project effects are insignificant.

7.8 Kemp's Ridley Sea Turtle

The Project would result in long-term alterations of sea turtle habitat within the action area. Long-term alterations of sea turtle habitat would occur due to minor increases in SAV within the mid-basin and extended periods of areas below sea turtle temperature thresholds during high-flow winter months within the lower basin and adjacent to the Project diversion.

Other potential stressors to sea turtles include disturbance events during temporary construction activities, which may displace resting or foraging individuals near the diversion outfall. Additionally, spatial shifts in shrimp fishery effort due to the Project may more frequently overlap with sea turtle distributions in the basin, increasing their risk of direct injury or mortality.

However, based on tagging and capture data, Kemp's ridley sea turtles are not likely to occur in the mid-basin near the diversion, nor within the lower basin during winter months.

Therefore, the Project *may affect* Kemp's ridley sea turtle for the following reasons:

- In the unlikely event sea turtles are present near the diversion during construction activities, they may be disturbed or displaced during foraging activities.
- Kemp's ridley sea turtles may be affected by dredging or vessel operations supporting construction activities.
- Sea turtles present within the lower basin or adjacent to the diversion during winter months may be exposed to temperatures below their temperature threshold.
- Kemp's ridley sea turtles may be affected by changes in distribution or abundance of prey.
- Salinity changes in Barataria Basin may concentrate sea turtles in the lower portion of Barataria Basin.

And is *likely to adversely affect* Kemp's ridley sea turtles for the following reasons:

- Sea turtles may experience shifts in the timing, abundance or distribution of prey items as a result of the project.
- Commercial fishing effort and sea turtle distribution may co-occur in portions of the Lower Barataria Basin.

7.9 Leatherback Sea Turtle

Leatherback turtles are the most pelagic of sea turtle species with the potential to occur within the action area. Though it is possible transients may visit the barrier islands or basin, these areas fall outside of the leatherback's core habitat. The Project would result in very minor long-term alterations of available optimal leatherback sea turtle habitat within the action area. Long-term alterations of leatherback sea turtle habitat would occur due to the extension of time each year when areas below leatherback sea turtle optimal temperature thresholds would occur within the lower basin, and adjacent to the Project diversion (during high-flow winter months).

Other potential stressors to sea turtles include disturbance events during temporary construction activities, which may displace resting or foraging individuals (if present) near the diversion outfall.

However, based on tagging and capture data, leatherback sea turtles are not likely to occur within the action area and are unlikely to be affected by the Project.

Therefore, the Project *may affect* leatherback sea turtle for the following reasons:

- In the unlikely event sea turtles are present near the diversion during construction activities, they may be disturbed or displaced during foraging activities.
- Sea turtles present within the lower basin or adjacent to the diversion during winter months may be exposed to temperatures below their temperature threshold.
- Salinity changes in Barataria Basin may concentrate sea turtles in the lower portion of Barataria Basin.

The Project is *not likely to adversely affect* leatherback sea turtle for the following reasons:

- Leatherback sea turtles have only been documented outside of the barrier islands, a portion of the action area where project effects are insignificant.

7.10 Loggerhead Sea Turtle and Loggerhead Sea Turtle Designated Critical Habitat

The Project would result in long-term alterations of sea turtle habitat within the action area. Long-term alterations of sea turtle habitat would occur due to minor increases in SAV within the mid-basin and extended periods of time when areas of the lower basin and adjacent to the Project diversion are below sea turtle temperature thresholds during high-flow winter months.

Other potential stressors to sea turtles include disturbance events during temporary construction activities, which may displace resting or foraging individuals near the diversion outfall. Additionally, spatial shifts in shrimp fishery effort due to the Project may more frequently overlap with sea turtle distributions in the basin, increasing their risk of direct injury or mortality.

Loggerhead sea turtles may occur throughout Barataria Basin, however limited tagging and capture data indicates that loggerhead sea turtles may have limited occurrence in the mid-basin near the diversion, or within the lower basin during winter months.

Therefore, the Project *may affect* loggerhead sea turtle for the following reasons:

- Sea turtles present near the diversion during construction activities may be disrupted or displaced during foraging activities.
- Sea turtles present within the lower basin or adjacent to the diversion during winter months may be exposed to temperatures below their temperature threshold.

- Salinity changes in Barataria Basin may concentrate sea turtles in the lower portion of Barataria Basin.

The Project is *likely to adversely affect* loggerhead sea turtle for the following reasons:

- Commercial fishing effort and sea turtle distribution may co-occur in portions of the Lower Barataria Basin.

Upland nesting areas for loggerheads in the barrier islands are not anticipated to experience Project effects. The Project is anticipated to have *no effect* on loggerhead sea turtle critical habitat for the following reasons:

- Project impacts are not anticipated to extend into designated critical habitat sargassum habitats outside of the barrier islands and on the edges of the Birdfoot Delta.

8.0 REFERENCES

Adams, S. R., J. J. Hoover, and K. J. Killgore. 1999. Swimming endurance of juvenile pallid sturgeon, Scaphirhynchus albus. Copeia:802-807.

Addison, L. 2012. Use of southeastern North Carolina inlets by migrating and wintering piping plovers (Charadrius melodus). Audubon North Carolina, Wilmington, North Carolina. Poster presentation at January 2012 Atlantic Coast piping plover and least tern workshop in Shepherdstown, West Virginia.

Ambrose, R.F. and T.W. Anderson. 1990. Influence of an artificial reef on the surrounding infaunal community. Marine Biology 107:41-52.

Baker, A. J., P. M. Gonzalez, T. Piersma, L. J. Niles, I. de Lima Serrano do Nascimento, P. W. Atkinson, N. A. Clark, C. D. T. Minton, M. K. Peck, and G. Aarts. 2004. Rapid population decline in red knots: fitness consequences of decreased refueling rates and late arrival in Delaware Bay. Proceedings of the Royal Society B: Biological Sciences 271:875-882.

Benedetti, M.M. 2003. Controls on overbank deposition in the Upper Mississippi River. Geomorphology, v. 56, issue 3-4, p. 271-290.

Blackstock, S.A., J.O. Fayton, P.H. Hulton, T.E. Moll, K. Jenkins, S. Kotecki, E. Henderson, V. Bowman, S. Rider and C. Martin. 2018. Quantifying Acoustic Impacts on Marine Mammals and Sea Turtles: Methods and Analytical Approach for Phase III Training and Testing. NUWC-NPT Technical Report August 2018. Naval Undersea Warfare Center Division Newport, Rhode Island.

Blevins, D. 2011. Water-Quality Requirements, Tolerances, and Preferences of Pallid Sturgeon (Scalphirhynchus albus) in the Lower Missouri River. U.S. Geologic Survey. Scientific Investigations Report 2011-5186.

Blue Planet Marine. 2013. Final Report: July 2013 Survey – Monitoring Aquatic Ambient Noise and the Associated Pressure Impacts in Port Curtis and Port Alma. CA130043., Report Reference BPM-NSW-13-GPC-Report-CA130043-Aquatic Noise Monitoring-v1.3.

Boesch, D.F., Josselyn, M.N., Mehta, A.J., Morris, J.T., Nuttle, W.K., Simenstad, C.A. and D.J. Swift. 1994. Scientific assessment of coastal wetland loss, restoration and management in Louisiana. Journal of Coastal Research, pp.i-103.

Bonin, P. 2015. Threatened loggerhead turtles discovered nesting on Grand Isle in 2015. Louisiana Sportsman.

Braaten, P. J., D. B. Fuller, L. D. Holte, R. D. Lott, W. Viste, T. F. Brandt, and R. G. Legare. 2008. Drift dynamics of larval pallid sturgeon and shovelnose sturgeon in a natural side channel of the Upper Missouri River, Montana. North American Journal of Fish Management 28:808-826.

Braaten, P.J., D.B. Fuller, R.D. Lott, M.P. Ruggles, and R.J. Holm. 2010. Spatial Distribution of Drifting Pallid Sturgeon Larvae in the Missouri River Inferred from Two Net Designs and Multiple Sampling Locations. North American Journal of Fisheries Management, volume 30, p. 1062-1074.

Bramblett, R.G. 1996. Habitats and movements of pallid and shovelnose sturgeon in the Yellowstone and Missouri Rivers, Montana and North Dakota. Thesis submitted to Montana State University. Bozeman, Montana.

Bramblett, R.G and R White. 2001. Habitat Use and Movements of Pallid and Shovelnose Sturgeon in the Yellowstone and Missouri Rivers in Montana and North Dakota, Transactions of the American Fisheries Society, 130:6, 1006-1025, DOI: 10.1577/1548-8659(2001)130<1006:HUAMOP>2.0.CO;2

Brault, S. 2007. Population viability analysis for the New England population of the piping plover (Charadrius melodus). Report 5.3.2-4. Prepared for Cape Wind Associates, L.L.C., Boston, Massachusetts.

Brooks, M. T., Coles, V. J., Hood, R. R., & Gower, J. F. (2018). Factors controlling the seasonal distribution of pelagic Sargassum. Marine Ecology Progress Series, 599, 1-18.

Brown, R.S., K.V. Cook, B.D. Pflugrath, L.L. Rozeboom, R.C. Johnson, J.G. McLellan, T.J. Linley, Y. Gao, L.J. Baumgartner, F.E. Dowell, E.A. Miller, and T.A.White. 2013. Vulnerability of larval and juvenile white sturgeon to barotrauma: can they handle the pressure? Journal of Conservation Physiology 1:1-9.

BTNEP (Barataria Terrebonne National Estuary Program). 2010. Shedding Light on Our Estuary & Our Economy: Environmental Indicators in the Barataria-Terrebonne Estuary System 2010. Thibodaux, LA. 36 pp.

Caillouet, C.W., B.J. Gallaway, and N.F. Putman. 2016. Kemp’s Ridley Sea Turtle Saga and Setback: Novel Analyses of Cumulative Hatchlings Released and Time-Lagged Annual Nests in Tamaulipas, Mexico. Chelonian Conservation and Biology 15(1): 115–131.

Caillouet, C.W., S.W. Raborn, D.J. Shaver, N.F. Putman, B.J. Gallaway, and K.L. Mansfield. 2018. Did Declining Carrying Capacity for the Kemp’s Ridley Sea Turtle Population Within the Gulf of Mexico Contribute to the Nesting Setback in 2010-2017? Chelonian Conservation and Biology 17(1): 123–133.

Calvert A., A. D. Amirault, F. Shaffer, R. Elliot, A. Hanson, J. McKnight, and P. Taylor. 2006. Population Assessment of an Endangered Shorebird: the Piping Plover (Charadrius melodus melodus) in Eastern Canada. Avian Conservation and Ecology.

Carle, M. V., Sasser, C. E., & Roberts, H. H. (2015). Accretion and vegetation community change in the Wax Lake Delta following the historic 2011 Mississippi River flood. Journal of Coastal Research, 31(3), 569-587.

Carlson, D. M., Pflieger, W. L., Trial, L. and Haverland, P. S. 1985. Distribution, biology, and hybridization of Scaphirhynchus albus and S. platorynchus in the Missouri and Mississippi rivers. Environmental Biology of Fishes, 14: 51–59.

Carlson, D. M. and Pflieger, W. L. 1981. Abundance and life history of the lake, pallid, and shovelnose sturgeons in Missouri, Final Report to the Missouri Department of Conservation, Endangered Species Project SE-1hyphen; 6 Jefferson City.

Chu, S.H., Zhang, Q.S., Tang, Y.Z., Zhang, S.B., Lu, Z.C. and Yu, Y.Q., 2012. High tolerance to fluctuating salinity allows *Sargassum thunbergii* germlings to survive and grow in artificial habitat of full immersion in intertidal zone. Journal of Experimental Marine Biology and Ecology, 412, pp.66-71.

CPRA (Coastal Protection and Restoration Authority for Louisiana). 2007. Integrated Ecosystem Restoration and Hurricane Protection: Louisiana's 2007 Comprehensive Master Plan for a Sustainable Coast. 140 pp.

CPRA. 2009. Sediment Testing of Dredging Material Proposed for the Mississippi River Sediment Delivery System-Bayou Dupont (BA-39). Project report.

CPRA. 2012. Louisiana's Comprehensive master plan for a sustainable coast. 190p.

CPRA. 2017. Louisiana's Comprehensive master plan for a sustainable coast. Effective June 2, 2017. 184p.

CPRA. 2017. State of Louisiana Coastal Protection and Restoration Authority: Mid-Barataria Sediment Diversion (MBSD) Project State Project No. BA-153. Engineering and Design Scope of Services.

Cohen, J. B., S. M. Karpanty, D. H. Catlin, J. D. Fraser, and R. A. Fischer. 2008. Winter ecology of piping plovers at Oregon Inlet, North Carolina. Waterbirds 31:472-479.

Constant, G. C., W.E. Kelso, D.A. Rutherford, C.F. Bryan. 1997. Habitat, movement, and reproductive status of pallid sturgeon (Scaphirhynchus albus) in the Mississippi and Atchafalaya rivers. Report prepared for U.S Army Engineer New Orleans District, 78 pp.

Couvillion, B.R.; Barras, J.A.; Steyer, G.D.; Sleavin, William; Fischer, Michelle; Beck, Holly; Trahan, Nadine; Griffin, Brad; and Heckman, David, 2011, Land area change in coastal Louisiana from 1932 to 2010: U.S. Geological Survey Scientific Investigations Map 3164, scale 1:265,000, 12 p. pamphlet.

Couvillion, B.R., Beck, Holly, Schoolmaster, Donald, and Fischer, Michelle, 2017, Land area change in coastal Louisiana 1932 to 2016: U.S. Geological Survey Scientific Investigations Map 3381, 16 p. pamphlet, https://doi.org/10.3133/sim3381.

Couvillion, B.R., H. Beck, D. Schoolmaster, and M. Fischer. 2017. Land area change in coastal Louisiana 1932 to 2016: U.S. Geological Survey Scientific Investigations Map 3381, 16 p. pamphlet. Available at: https://doi.org/10.3133/sim3381.

Cowan, J.H., Turner, R.E. and Cahoon, D.R., 1988. Marsh management plans in practice: do they work in coastal Louisiana, USA?. Environmental Management, 12(1), pp.37-53.

Day, J.W., Boesch, D.F., Clairain, E.J., Kemp, G.P., Laska, S.B., Mitsch, W.J., Orth, K., Mashriqui, H., Reed, D.J., Shabman, L. and C.A. Simenstad. 2007. Restoration of the Mississippi Delta: lessons from hurricanes Katrina and Rita. science, 315(5819), pp.1679-1684.

De Graaf, R. M., Tilghman, N. G., and Anderson, S. H. (1985). Foraging guilds of North American birds. Environmental Management, 9(6), 493-536.

de Jong, C.A.F., Ainslie, M.A., Dreschler J., Jansen, H.W.J., Heemskerk, H.J.M., Groen, W.H.M. 2010. Underwater noise of trailing suction hopper dredgers at Maasvlakte 2: analysis of source levels and background noise. Report TNO-DV 2010 C335, Netherlands Organization for Applied Scientific Research (TNO), The Hague.

DeLonay, A.J., R.B. Jacobson, D.M. Papoulias, D.G. Simpkins, M.L. Wildhaber, J.M. Reuter, T.W. Bonnot, K.A. Chojnacki, G.E. Mestl, and M.J. Mac. 2009. Ecological requirements for pallid sturgeon reproduction and recruitment in the Lower Missouri River—A research synthesis 2005–08: U.S. Geological Survey Scientific Investigations Report 2009–5201, 59 p.

DeLonay, A.J., D.M. Papoulias, M.L. Wildhaber, G.E. Mest, D.W. Everitt, and K.A. Chojnacki. 2007. Movement, habitat use, and reproductive behavior of shovelnose sturgeon and pallid sturgeon in the Lower Missouri River, in Korschgen, C.E., ed., Factors affecting the reproduction, recruitment, habitat, and population dynamics of Pallid Sturgeon and Shovelnose Sturgeon in the Missouri River: U.S. Geological Survey Open-File Report 2007–1262, p. 23–102.

Deltares. 2014. Delft 3D Flexible Mesh Suite. https://www.deltares.nl/en/software/delft3d-flexible-mesh-suite/

DeMarco, K., Couvillion, B., Brown, S., & La Peyre, M. (2018). Submerged aquatic vegetation mapping in coastal Louisiana through development of a spatial likelihood occurrence (SLOO) model. Aquatic Botany, 151, 87-97.

DeMay, R., D. LeBlanc, D. Norman, D. O'Malley, T. O'Malley, E. Johnson, N. Waters, and B. Hutchinson. 2015. Piping Plover 2015 Survey of the Caminada Headland, Louisiana: May 6, 2015. Prepared by the Barataria-Terrebonne National Estuary Program for the Coastal Protection and Restoration Authority. 24 p.

DeVries, R. J., D. A. Hann, and H. L. Schramm. 2015. Increasing capture efficiency of pallid sturgeon Scaphirhynchus albus (Forbes and Richardson, 1905) and the reliability of catch rate estimates. Journal of Applied Ichthyology 31:603-608.

DHNRDAT (Deepwater Horizon Natural Resource Damage Assessment Trustees). 2016. Deepwater Horizon oil spill: Final Programmatic Damage Assessment and Restoration Plan and Final Programmatic Environmental Impact Statement. Available: http://www.gulfspillrestoration.noaa.gov/restoration-planning/gulf-plan. Viewed: April 11, 2019.

Dickerson, C., Reine, K.J., Clarke, D.G. and Engler, R.M., 2001. Characterization of underwater sounds produced by bucket dredging operations (No. ERDC-TN-DOER-E14). Engineer Research and Development Center, Vicksburg, MS.

Discovery of Sound in the Sea [DOSITS] 2017 (underwater noise)

Dortch, Q., Moncreiff, C.A., Mendenhall, W.M., Parsons, M.L., Franks, J.S., Hemphill, K.W., 1998. Spread of Gymnodinium breve into the northern Gulf of Mexico. In: Reguera, B., Blanco, J., Fernandez, M.L., Wyatt, T. (Eds.), Harmful Algae. Xunta de Galicia and Intergovernmental Oceanographic Commission of UNESCO, Vigo, Spain, pp. 143–144.

Drake, K.R., J.E. Thompson, K.L. Drake, and C. Zonick. 2001. Movements, habitat use, and survival of non-breeding Piping Plovers. Condor 103(2):259-267.

Eddleman, William R., R. E. Flores and M. Legare. 1994. Black Rail (Laterallus jamaicensis), version 2.0. In The Birds of North America (P. G. Rodewald, editor). Cornell Lab of Ornithology, Ithaca, New York, USA. Available at: https://doi.org/10.2173/bna.123.

Elliot-Smith, E. and S. Haig. 2004. Winter Distribution and abundance of Snowy Plovers in eastern North America and the West Indies. Wader Study Group Bulletin, volume 104.

Elliott-Smith, E., M. Bidwell, A.E. Holland, and S.M. Haig. 2015. Data from the 2011 International Piping Plover Census. U.S. Geological Survey Data Series 922, 296 p., http://dx.doi.org/10.3133/ds922.

Feist, F.E. 1992. Potential impacts of pile driving on juvenile pink (Oncorhyncus gorbuscha) and chub (O. keta) salmon behavior and distribution. MS Thesis, University of Washington.

Foley, A.M., K.E. Singel, P.H. Dutton, T.M. Summers, A.E. Redlow and J. Lessman. 2007. Characteristics of a Green Turtle (Chelonia mydas) Assemblage in Northwestern Florida Determined During a Hypothermic Stunning Event. Gulf of Mexico Science 25 (2). Available at: https://aquila.usm.edu/goms/vol25/iss2/4. Accessed January 2021.

Fonseca, M. and S.S. Bell. 1998. Influence of Physical Setting on Seagrass Landscapes Near Beaufort, North Carolina, USA. Marine Ecology Progress Series, 171: 109-121.

Fontenot, B. and R DeMay. 2014. Louisiana Shorebirds. A Barataria Terrebonne National Estuary Program Publication. Available at: https://btnep.org/wp-content/uploads/2017/07/Final_Proof_161641_ShorebirdsLR.pdf. Accessed December, 2017.

French, W.E., B. D. S. Graeb, S.R. Chipps, K.N Bertrand, T.M. Selch, and R.A. Klumb. 2010. Vulnerability of age-0 pallid sturgeon Scaphirhynchus albus to fish predation. Journal of Applied Ichthyology.

Friedenberg, N.A., J.J. Hoover, K. Boysen, and K.J. Killgore. 2018. Estimating abundance without recapture of marked pallid sturgeon in the Mississippi River. Conservation Biology, volume 32, p. 457-465.

Friedenberg, N.A. and J. Siegrist. 2019. Computation of pallid sturgeon entrainment and population level risk. November 11, 2019.

Garvey, J.E., E.J. Heist, R.C. Brooks, D.P. Herzog, R. A. Hrabik, K.J. Killgore, J. Hoover, and C. Murphy. 2009. Current status of the pallid sturgeon in the Middle Mississippi River: habitat, movement, and demographics. Saint Louis District, US Army Corps of Engineers.

Gerrity, P.C., C.S. Guy, and W.M Gardner. 2006. Juvenile pallid sturgeon are piscivorous: a call for conserving native cyprinids. Transactions of the American Fisheries Society 135:604-609.

Gill, J.A., K. Norris, P. M. Potts, T. G. Gunnarsson, P. W. Atkinson, and W. J. Sutherland. 2001. The buffer effect and large-scale population regulation in migratory birds. Nature 412:436-438.

Gisiner, R. C. 1998. Workshop on the Effects of Anthropogenic Noise in the Marine Environment. Marine Mammal Science Program, Office of Naval Research, 1998.

Gratto-Trevor, C., D. Amirault-Langlais, D. Catlin, F. Cuthbert, J. Fraser, S. Maddock, E. Roche, and F. Shaffer. 2012. Connectivity in piping plovers: do breeding populations have distinct winter distributions? Journal of Wildlife Management 76:348-355.

Grohs, K.L., R.A. Klumb, S.R. Chipps, and G.A. Wanner. 2009. Ontogenetic patterns in prey use by pallid sturgeon in the Missouri River, South Dakota and Nebraska. Journal of Applied Ichthyology.

Hastings, M.C. and A.N. Popper. 2005. Effects of Sound on Fish. California Department of Transportation Contract 43A0139, Task Order 1.

Haig, S.M. 1992. Piping Plover in The Birds of North America, No. 2 (A. Poole, P. Stettenheim and F. Gill, editors). Philadelphia: The academy of Natural Sciences; Washington DC: The American Ornithologists' Union.

Harrington, B.A. 2008. Coastal inlets as strategic habitat for shorebirds in the southeastern United States. DOER technical notes collection. ERDC TN-DOERE25. U.S. Army Engineer Research and Development Center; Vicksburg, Mississippi.

Hartfield, P., N.M. Kuntz, and H.L. Schramm, Jr. 2013. Observations on the identification of larval and juvenile Scaphirhynchus spp. in the Lower Mississippi River. Southeastern Naturalist 12(2):251–266.

Hawkes, L. A., A. C. Broderick, M. H. Godfrey, and B. J. Godley. 2007. Investigating the potential impacts of climate change on a marine turtle population. Global Change Biology 13:1-10.

Hays, G. C., A. C. Broderick, F. Glen, B. J. Godley, J. D. R. Houghton, and J. D. Metcalfe. 2002. Water temperature and internesting intervals for loggerhead (Caretta caretta) and green (Chelonia mydas) sea turtles. Journal of Thermal Biology 27(5):429-432.

Herrala, J.R., P.T. Kroboth, N. M. Kuntz and H.L. Schramm Jr. 2014. Habitat Use and Selection by Adult Pallid Sturgeon in the Lower Mississippi River, Transactions of the American Fisheries Society, 143:1, 153-163, DOI: 10.1080/00028487.2013.830987

Hesse L.W. and G.E. Mestl. 1993. An Alternative Hydrograph for the Missouri River Based on the Precontrol Condition, North American Journal of Fisheries Management, 13:2, 360-366, DOI: 10.1577/1548-8675(1993)013<0360:AAHFTM>2.3.CO;2

Hillmann, E.R., K. DeMarco, and M. K. LaPeyre. 2016. Establishing a baseline of estuarine submerged aquatic vegetation resources across salinity zones within coastal areas of the northern Gulf of Mexico. Journal of the Southeastern Association of Fish and Wildlife Agencies 3: 25-32.

Hillmann, E., K. DeMarco, and M. La Peyre. 2017. Submerged aquatic vegetation and environmental data along a salinity gradient in Barataria Bay, Louisiana (2015): U.S. Geological Survey Data Release. Available at: https://doi.org/10.5066/F7M61HG4.

Hoover, J.J., J. Collins, K.A. Boysen, A.W. Katzenmeyer, and K.J. Killgore. 2011. Critical swimming speeds of adult shovelnose sturgeon in rectilinear and boundary-layer flow. Journal of Applied Ichthyology, volume 27.

Hoover J.J., S.G. George, and K.J. Killgore. 2007. Diet of shovelnose sturgeon and pallid sturgeon in the free-flowing Mississippi River. Journal of Applied Ichthyology, Volume 23, Issue 4.

Hurley, K. L., R.J. Sheehan, R.C. Heidinger, and P.S. Wills. 2004. Habitat use by Middle Mississippi River pallid sturgeon. Trans. Am. Fish. Soc. 133, 1033–1041.

Illingworth, R. and R. Rodkin. 2007. Compendium of pile driving sound data. Prepared for the California Department of Transportation, Sacramento, CA.

Jones, D., Marten, K., and K. Harris. 2015. Underwater Sound from Dredging Activities: Establishing source levels and modelling the propagation of underwater sound. Proceedings from the EDA Dredging Days 2015: Innovative dredging solutions for ports, Rotterdam, the Netherlands.

Kallemeyn, L.1983. Status of the pallid sturgeon. American Fisheries Society 8, 3–9.

Kappenman, K. M., W. C. Fraser, M. Toner, J. Dean, and M. A. H. Webb. 2009. Effect of temperature on growth, condition, and survival of juvenile shovelnose sturgeon. Transactions of the American Fisheries Society 138:927-937.

Kilgore KJ, Hoover JJ, George SG, Lewis BR, Murphy CE, Lancaster WE. 2007. Distribution, relative abundance and movements of pallid sturgeon in the free-flowing Mississippi River. Journal of Applied Ichthyology 23:476–483.

Knudsen, F.R, P.S. Enger, and O. Sand. 1992. Awareness reactions and avoidance responses to sound juvenile Atlantic salmon, Salmo salar L. Journal of Fish Biology, volume 40, p. 523-534.

Koch, E.W. 2001. Beyond light: physical, geological, and geochemical parameters as possible submersed aquatic vegetation habitat requirements. Estuaries, 24(1), pp.1-17.

Koch B., R.C. Brooks, A. Oliver, D. Herzog, J.E. Garvey, R. Hrabik, R. Colombo, Q. Phelps and T. Spier. 2012. Habitat Selection and Movement of Naturally Occurring Pallid Sturgeon in the Mississippi River, Transactions of the American Fisheries Society, 141:1, 112-120, DOI: 10.1080/00028487.2011.652008

Kocmoud, A.R., H.H. Wang, W.E. Grant, and B.J. Gallaway. 2019. Population dynamics of the endangered Kemp's ridley sea turtle following the 2010 oil spill in the Gulf of Mexico: Simulation of potential cause-effect relationships. Ecological Modeling 392: 159-178.

Krentz, S. 2004. Scaphirhynchus albus. Missouri River Fish & Wildlife Management Assistance. The IUCN Red List of Threatened Species 2004: e.T19940A9111329.

Kroboth, P. K., J. R. Herrala, N. M. Kuntz, D. Hann, J. Porter, T. Young, and H. L. Schramm. 2013. Pallid sturgeon habitat use, movement, and growth and mortality rate estimates in the Lower Mississippi River, Report for 2007-2013. Report submitted to Jeff Quinn, Arkansas Game and Fish Commission, Little Rock, AR.

Kynard, B., M. Breece, Atcheson M. and M. Kieffer. Status of Shortnose Sturgeon in the Potomac River. Part 1: Field Studies. U.S. Fish and Wildlife Service.

Larson, M. A., M. R. Ryan, and R. K. Murphy. 2002. Population viability of piping plovers: effects of predator exclusion. Journal of Wildlife Management 66:361-371.

Lauritsen, A.M., P.M. Dixon, D. Cacela, B. Brost, R. Hardy, S.L. MacPherson, A. Meylan, B.P. Wallace, and B. Witherington. 2017. Impact of the Deepwater Horizon oil spill on loggerhead turtle Caretta caretta nest densities in northwest Florida. Endangered Species Research 33: 83-97.

Magaña, H. A., C. Contreras., and T.A. Villareal. (2003). A historical assessment of Karenia brevis in the western Gulf of Mexico. Harmful Algae, 2(3), 163-171.

McGowan R. and M. Ryan. 2009. A quantitative framework to evaluate incidental take and endangered species population viability. Biological Conservation, Volume 142, Issue 12. Pages 3128-3136.

Melvin, S.M. and J.P. Gibbs. 1996. Viability analysis for the Atlantic Coast population of piping plovers. Pages 175-186 in Piping plover (Charadrius melodus), Atlantic Coast population, revised recovery plan. U.S. Fish and Wildlife Service; Hadley, Massachusetts.

Miksis-Olds, J.L., Donaghay, P.L., Miller, J.H., Tyack, P.L. and Nystuen, J.A., 2007. Noise level correlates with manatee use of foraging habitats. The Journal of the Acoustical Society of America, 121(5): 3011-3020.

Modde, T. and J.C. Schmulbach, 1977. Food and feeding behavior of the shovelnose sturgeon, Scaphirhynchus platorynches, in the unchannelized Missouri River, South Dakota. Trans. Am. Fish. Soc. 106(6):602-608.

Morrison, R. I. G. and K. A. Hobson. 2004. Use of body stores in shorebirds after arrival on high-arctic breeding grounds. Auk 121:333-344.

Mueller, R.P., D.A. Neitzel, W.V. Mavros, and T.J. Carlson. 1998. Evaluation of low and high frequency sound for enhancing fish screening facilities to protect outmigrating salmonids. Project No. 86-118. U.S. Department of Energy, Bonneville Power Administration.

NMFS (National Marine Fisheries Service). 2016. Framework Biological Opinion on Deepwater Horizon Oil Spill, Final Programmatic Damage Assessment and Restoration Plan and Final Programmatic Environmental Impact Statement (SER-2015-17459). 414p.

(NMFS. 2018a. ESA effects determinations.

NMFS. 2018b. 2018 Revisions to: Technical Guidance for Assessing the Effects of Anthropogenic Sound on Marine Mammal Hearing (Version 2.0): Underwater Thresholds for Onset of Permanent and Temporary Threshold Shifts. U.S. Dept. of Commer., NOAA. NOAA Technical Memorandum NMFS-OPR-59, 167 p.

NMFS and USFWS 2013a. Hawksbill Sea Turtle (Eretmochelys imbricata) – 5-Year Review: Summary and Evaluation. National Marine Fisheries Service Office of Protected Resources and U.S. Fish and Wildlife Service. June 2013.

NMFS and USFWS 2013b. Leatherback Sea Turtle (*Dermochelys coriacea*) – 5-Year Review: Summary and Evaluation. National Marine Fisheries Service Office of Protected Resources and U.S. Fish and Wildlife Service. June 2013.

NMFS and USFWS. 2020. Endangered Species Act status review of the leatherback turtle (*Dermochelys coriacea*). Report to the National Marine Fisheries Service Office of Protected Resources and U.S. Fish and Wildlife Service.

NOAA. 2019. Sea Turtle Stranding Network: Sea Turtle Stranding by Zone Reports. https://grunt.sefsc.noaa.gov/stssnrep/SeaTurtleReportI.do?action=reportquery

NOAA. 2020a. NOAA Species Directory: Green Turtle. https://www.fisheries.noaa.gov/species/green-turtle

NOAA 2020b. NOAA Species Directory: Leatherback Turtle. https://www.fisheries.noaa.gov/species/leatherback-turtle

National Oceanographic and Atmospheric Administration (NOAA) Greater Atlantic Regional Fisheries Office (GARFO). 2020c. GARFO Acoustics Tool: Analyzing the effects of pile driving on ESA-listed species in the Greater Atlantic Region. Updated September 14, 2020.

NatureServe. 2017. NatureServe Explorer: An online encyclopedia of life [web application]. Version 7.1. NatureServe, Arlington, Virginia. Available at: http://explorer.natureserve.org. Accessed January, 2018.

Nicholls, J.L. and G.A. Baldassarre. 1990a. Habitat selection and interspecific associations of piping plovers along the Atlantic and Gulf Coasts of the United States. M.S. Thesis. Auburn University; Auburn, Alabama.Nicholls and Baldassarre 1990b

Nicholls, J.L. and G.A. Baldassarre. 1990b. Habitat associations of piping plover wintering in the United States. Wilson Bulletin 102(4):581-590.

Nicholls, J.L. 1989. Distribution and other ecological aspects of Piping Plovers (Charadrius melodus) wintering along the Atlantic and Gulf coasts. MS Thesis. Auburn University, Auburn, Alabama. 164 pp.

Noel, B.L., C.R. Chandler, and B. Winn. 2007. Seasonal abundance of nonbreeding piping plovers on a Georgia barrier island. Journal of Field Ornithology 78:420-427.

Noel, B., and Chandler, C. (2008). Spatial Distribution and Site Fidelity of Non-Breeding Piping Plovers on the Georgia Coast. Waterbirds: The International Journal of Waterbird Biology, 31(2), 241-251. Pike, D. A., R. L. Antworth, and J. C. Stiner. 2006. Earlier nesting contributes to shorter nesting seasons for the loggerhead seaturtle, Caretta caretta. Journal of Herpetology 40(1):91-94.

Olsen, K. 1969. Directional responses in herring for sound and noise stimuli. International Council for Exploration of the Sea (B 20), 1-8.

Ortiz, R.M., Patterson, R.M., Wade, C.E. and F.M. Byers. 2000. Effects of acute fresh water exposure on water flux rates and osmotic responses in Kemp's ridley sea turtles (*Lepidochelys kempi*). Comparative Biochemistry and Physiology Part A: Molecular & Integrative Physiology, 127(1), pp.81-87.

Paola, C. et al. 2011. Natural Processes in Delta Restoration: Application to the Mississippi delta. Annual Review. Marine Science. 3: 67-91.

Plissner, J.H. and S.M. Haig. 2000. Viability of piping plover Charadrius melodus metapopulations. Biological Conservation 92:163-173.Powell, J.A. and G.B. Rathbun. 1984. Distribution and abundance of manatees along the northern coast of the Gulf of Mexico. Northeast Gulf Science 7(1): 1-28.

Pfister, C., B.A. Harringon, and M. Lavine. 1992. The impact of human disturbance on shorebirds at a migration staging area. Biological Conservation, volume 60, p. 115-126.

Popper, A.N. 2003. Effects of anthropogenic sound on fishes. Fisheries, volume 28, p. 24–31.

Popper, A.N. and A.D. Hawkins. 2012 The effects of noise on aquatic life. Springer Science+Business Media, LLC, New York.

Popper, A.N., A.D. Hawkins, R.R. Fay, D. Mann, S. Bartol, T. Carlson, S. Coombs, W.T. Ellison, R. Gentry, M.B. Halvorsen. and S. Løkkeborg. 2014. Sound Exposure Guidelines for Fishes and Sea Turtles: A Technical Report prepared by ANSI-Accredited Standards Committee S3/SC1 and registered with ANSI. 10.1007/978-3-319-06659-2.

Putman, N.F., F.A. Abreu-Grobois, I. Iturbe-Darkistade, E.M. Putman, P.M. Richards, and P. Verley. 2015. Deepwater Horizon oil spill impacts on sea turtles could span the Atlantic. Biology Letters 11: 20150596.

Reid, J.P., G.B. Rathbun, and J.R. Wilcox. 1991. Distribution patterns of individually identifiable West Indian manatees (Trichechus manatus) in Florida. Marine Mammal Science7:180-190.

Reine, K.J., Clarke, D.G., and C. Dickerson. 2012. Characterization of underwater sounds produced by a backhoe dredge excavating rock and gravel. DOER technical notes collection ERDC TN-DOER-E36, US Army Engineer Research and Development Center, Vicksburg, MS. Available at http://el.erdc.usace.army.mil/elpubs/pdf/doere36.pdf

Reyff, J.A. 2006. Russian River Bridge at Geyserville: Underwater sound measurement data for driving permanent 48-inch CISS piles. Illingworth and Rodkin, Inc., Petaluma, California

Richards PM, S.P. Epperly, S.S. Heppell, R.T. King, C.R. Sasso, F. Moncada, G. Nodarse, D.J. Shaver, Y. Medina, J. Zurita. 2011. Sea turtle population estimates incorporating uncertainty: a new approach applied to western north Atlantic loggerheads Caretta caretta. Endang Species Res 15:151–158

Roche, E. A., J. B. Cohen, D. H. Catlin, D. L. Amirault-Langlais, F. J. Cuthbert, C. L. Gratto-Trevor, J, Felio, and J. D. Fraser. 2010. Range-wide piping plover survival: correlated patterns and temporal declines. Journal of Wildlife Management 74:1784-1791.

Ryan, M.R., B.G. Root, and P.M. Mayer. 1993. Status of Piping Plovers in the Great Plains of North America: A Demographic Simulation Model. Conservation Biology, Vol. 7, Issue 3.

Sadid, K., Messina, F., Jung, H., Yuill, B., & Meselhe, E. (2018). TO51: Basinwide Model Version 3 - Basinwide Model for Mid-Breton Sediment Diversion Modeling. Baton Rouge, LA: The Water Institute of the Gulf. Funded by the Coastal Protection and Restoration Authority under Task Orders 51.

Schramm, H., P. Hartfield, and D. Hann. 2017. Observations on Trawl Sampling for Age-0 Scaphirhynchus spp. in the Lower Mississippi River. In: Schramm, Jr., H., Abundance, Growth, Mortality, and Habitat Use of Pallid Sturgeon and Shovelnose Sturgeon in the Lower Mississippi River. Report to U.S. Fish and Wildlife Service, Jackson, MS.

Schultz, D. L. 2013. Fish entrainment by freshwater diversions of the lower Mississippi River in ERDC Environmental Laboratory, ed. Entrainment studies of pallid sturgeon associated with water diversions in the lower Mississippi River, New Orleans District, U.S. Army Corps of Engineers.Turnpenny, A.W.H and J.R. Nedwell. 1994. The effects on marine fish, diving mammals and birds of underwater sound generated by seismic surveys. FARL Report Reference: FCR 089/94, October 1994.

Seminoff, J.A., Allen, C.D., Balazs, G.H., Dutton, P.H., Eguchi, T., Haas, H., Hargrove, S.A., Jensen, M., Klemm, D.L., Lauritsen, A.M. and S.L. MacPherson. 2015. Status review of the green turtle (Chelonia mydas) under the Endangered Species Act. NOAA Technical Memorandum, NOAA-NMFS-SWFSC-539. 571pp.

Shaver, D.J., K.M. Hart, I. Fujisaki, C. Rubio, A.R. Sartain, J. Peña, P.M. Burchfield, D. Gomez Gamez, and J. Ortiz. 2013. Foraging area fidelity for Kemp's ridleys in the Gulf of Mexico. Ecology and Evolution 3(7): 2002-2012.

Stucker, J. H., F. J. Cuthbert, B. Winn, B. L. Noel, S. B. Maddock, P. R. Leary, J. Cordes, and L. C. Wemmer. 2010. Distribution of non-breeding Great Lakes piping plovers (Charadrius melodus) along Atlantic and Gulf of Mexico coastlines: ten years of band sightings. Waterbirds: 33:22-32.

Swarzenski, C.M., and S.M. Perrien. 2015. Discharge, suspended sediment, and salinity in the Gulf Intracoastal Waterway and adjacent surface waters in south-central Louisiana, 1997–2008: U.S. Geological Survey Scientific Investigations Report 2015–5132, 21 p., http://dx.doi.org/10.3133/sir20155132.

Swenson, E.M. and R.E. Turner. 1987. Spoil banks: effects on a coastal marsh water-level regime. Estuarine, Coastal and Shelf Science, 24(5), pp.599-609.

TWI (The Water Institute). 2013. Adaprive Management Framework for Coastal Louisiana. Produced for and Funded by CPRA. October 25, 2013. 28 pp.

TWI. 2015. Basin wide Model Development for the Louisiana Coastal Area Mississippi River hydrodynamic and Delta Management Study (TO27). 322p.

Turnpenny, A.W.H., R.M.H Seaby, J.R. Nedwell, and K. Needham. 1994. Underwater sound pressures measured during seismic testing at Redhorn Quay. Client Report to BP Exploration, FCR 118/94.

USEPA (U.S. Environmental Protection Agency). 1974. Information on Levels of Environmental Noise Requisite to Protect Public Health and Welfade with an Adequate Margin of Safety. Prepared by the USEPA Office of Noise Abatement and Control. 242 pp.

USEPA. 2010. Nutrients in Estuaries. A Summary Report of the National Estuarine Experts Workgroup 2005-2007. Contracts 68-C-02-091 and EP-C-07-025.

USACE (U.S. Army Corps of Engineers). 2017. Biological Assessment: Bonnet Carré Spillway 2011 and 2016 emergency operations, New Orleans District.Valverde, R.A. and K. R. Holzwart. 2017. Chapter 11: Sea Turtles of the Gulf of Mexico. Habitats and Biota of the Gulf of Mexico: Before the Deepwater Horizon Oil Spill. Springer-Verlag. New York.

USACE. 2018. Davis Pond Freshwater Diversion. Available at: http://www.mvn.usace.army.mil/About/Projects/Davis-Pond-Freshwater-Diversion/. Accessed December 2018.

USACE. 2019. Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtant Structures. Engineer Pamphlet (EP) 1110-2-18. May 1, 2019.

USFWS (U.S. Fish and Wildlife Service). 1985. Determination of endangered and threatened status for the piping plover. Federal Register 50: 50726-50734.

USFWS. 1996. Piping plover (Charadrius melodus) Atlantic Coast population revised recovery plan. Hadley, MA. 246 pp. Available online at http://www.fws.gov/northeast/pipingplover/recovery.html

USFWS 2009 (U.S. Fish and Wildlife Service). Biological Opinion: Small Diversion at Convent/Blind River Project, Louisiana Coastal Area. 94p.

USFWS. 2010. Biological Opinion: Medium Diversion at White Ditch located in Plaquemines Parish, Louisiana, and its effects on the endangered pallid sturgeon (Scaphirhynchus albus) per section 7 of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 et seq.). 64p.

USFWS. 2013a. Draft Revised Recovery Plan for the Pallid Sturgeon (Scaphirhynchus albus). Prepared by G. R. Jordan. United Fish and Wildlife Service.

USFWS. 2013b. Habitat management plan for Delta and Breton National Wildlife Refuges: Plaquemines and St. Bernard parishes, Louisiana.

USFWS. 2014. Revised Recovery Plan for the Pallid Sturgeon (Scaphirhynchus albus). U.S. Fish and Wildlife Service, Denver, Colorado. 115 pp.

USFWS 2015 Piping Plover Recovery Plan V2 - U.S. Fish and Wildlife Service. 2015. Recovery Plan for the Northern Great Plains piping plover (Charadrius melodus) in two volumes. Volume I: Draft breeding recovery plan for the Northern Great Plains piping plover (Charadrius melodus) and Volume II: Draft revised recovery plan for the wintering range of the Northern Great Plains piping plover (Charadrius melodus) and Comprehensive conservation strategy for the piping plover (Charadrius melodus) in its coastal migration and wintering range in the continental United States. Denver, Colorado. 166 pp.

USFWS. 2017a. ECOS Species Profile for Piping Plover (Charadrius melodus). Available at: https://ecos.fws.gov/ecp0/profile/speciesProfile?spcode=B079#status. Accessed December, 2017.

USFWS. 2018a. Biological Opinion: Bonnet Carré Spillway 2011 and 2016 emergency operations. FWS Log #: 04EL1000-2018-F-0590, Louisiana Ecological Services Office. Lafayette, LA.

USFWS. 2018b. Hawksbill Sea Turtle (Eretmochelys imbricata). Available at: https://www.fws.gov/northflorida/seaturtles/turtle%20factsheets/hawksbill-sea-turtle.htm. Accessed February, 2018.

USFWS. 2018c. USFWS Map/Data for Eastern Black Rail SSA; Current Range of the Eastern Black Rail (2011 to present); May 2018. https://www.fws.gov/southeast/wildlife/birds/eastern-black-rail/#current-range-section.

USFWS. 2018d. Endangered, Threatened, and Candidate Species of Louisiana. Available at: https://www.fws.gov/southeast/pdf/fact-sheet/louisiana-ecological-services-field-office-t-and-e-species.pdf. Accessed November, 2018.

USFWS. 2020a. Biological Opinion: Bonnet Carré Spillway 2018 emergency operations. FWS Log #: 04EL1000-2020-F-1639, Louisiana Ecological Services Office. Lafayette, LA.

USFWS. 2020b. Draft Fish and Wildlife Coordination Act Report on the Mid-Barataria Sediment Diversion Project. Submitted to USACE and CPRA. Prepared by USFWS Ecological Services, Lafayette, LA. August 2020.

USGS (U.S. Geological Survey). 2008. A Conceptual Life-History Model for Pallid and Shovelnose Sturgeon. Circular 1315.

USGS. 2015. Louisiana coastal wetlands: A resource at risk. U.S. Geological Survey, Coastal & Marine Geology Program. Available at: http://pubs.usgs.gov/fs/la-wetlands/. Accessed May 2019.

USGS. 2018. National Water Information System: Web Interface. Available at: https://waterdata.usgs.gov/nwis.

Valverde, R.A. and Holzwart, K.R. 2017. Sea turtles of the Gulf of Mexico. In Habitats and Biota of the Gulf of Mexico: Before the Deepwater Horizon Oil Spill (pp. 1189-1351). Springer, New York, NY.

Wallace B., M. Rissing, D. Cacela, L. Garrison, T. McDonald, B. Schroeder, D. McLamb, B. Witherington, and B. Stacy. 2015. Estimating degree of oiling of sea turtles and surface habitat during the Deepwater Horizon oil spill: implications for injury quantification. (ST_TR.02). DWH Sea Turtles NRDA Technical Working Group Report.

Wanner, G. 2006. Evaluation of a Gastric Lavage Method on Juvenile Pallid Sturgeon. American Fisheries Society, Vol. 26, Issue 3.

Weishampel, J. F., D. A. Bagley, and L. M. Ehrhart. 2004. Earlier nesting by loggerhead sea turtles following sea surface warming. Global Change Biology 10:1424-1427.

Wemmer, L. C., U. Ozesmi, and F. J. Cuthbert. 2001. A habitat-based population model for the Great Lakes population of the piping plover (Charadrius melodus). Biological Conservation 99:169-181

Wissel, Björn, Arian Gaçe, and Brian Fry. "Tracing river influences on phytoplankton dynamics in two Louisiana estuaries." Ecology 86.10 (2005): 2751-2762.

Witt, M.J. 2007. The spatial ecology of marine turtles. University of Exeter. Doctor of Philosophy in Biological Sciences thesis.

WSDOT (Washington Department of Transportation). 2019. Construction Noise Impact Assessment. BA Manual.

WSDOT. 2008. Agreement in Principle for Interim Criteria for Injury to Fish from Pile Driving Activities. Dated June 12, 2008. 3 pp.

Zengel, S., Bernik, B.M., Rutherford, N., Nixon, Z. and J. Michel. 2015. Heavily oiled salt marsh following the Deepwater Horizon oil spill, ecological comparisons of shoreline cleanup treatments and recovery. PloS one, 10(7), p.e0132324.

Appendix A
Endangered Species Act Species Lists

Louisiana

Threatened and Endangered Species and Critical Habitats Under NOAA Fisheries Jurisdiction

Species	Listing Status	Recovery Plan	Critical Habitat
Green sea turtle	Threatened - North and South Atlantic Distinct Population Segment (81 FR 20057; April 6, 2016)	October 1991	63 FR 46693; September 2, 1998
Kemp's ridley sea turtle	Endangered (35 FR 18319; December 2, 1970)	September 2011	None
Leatherback sea turtle	Endangered (35 FR 8491; June 2, 1970)	April 1992	44 FR 17710; March 23, 1979
Loggerhead sea turtle	Threatened - Northwest Atlantic Ocean Distinct Population Segment (76 FR 58868; September 22, 2011)	December 2008	79 FR 39856; July 10, 2014
Hawksbill sea turtle	Endangered (35 FR 8491; June 2, 1970)	December 1993	63 FR 46693; September 2, 1998
Gulf sturgeon	Threatened (56 FR 49653; September 30, 1991)	September 1995	68 FR 13370; March 19, 2003
Oceanic whitetip shark	Threatened (83 FR 4153; January 30, 2018)	2018 Recovery Outline	None
Giant manta ray	Threatened (83 FR 2916; January 22, 2018)	None	None
Fin whale	Endangered (35 FR 18319/ December 2, 1970)	August 2010	None
Sperm whale	Endangered (35 FR 18319; December 2, 1970)	December 2010	None
Sei whale	Endangered (35 FR 12222/ December 2, 1970)	December 2011	None

Species	Listing Status	Recovery Plan	Critical Habitat
Gulf of Mexico Bryde's whale	Endangered (81 FR 88639; December 8, 2016)	None	None

Last updated by Southeast Regional Office on September 11, 2019

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Louisiana Ecological Services Field Office
200 Dulles Drive
Lafayette, LA 70506
Phone: (337) 291-3100 Fax: (337) 291-3139

In Reply Refer To: December 17, 2019
Consultation Code: 04EL1000-2020-SLI-0299
Event Code: 04EL1000-2020-E-00696
Project Name: Mid-Barataria Sediment Diversion

Subject: List of threatened and endangered species that may occur in your proposed project location, and/or may be affected by your proposed project

To Whom It May Concern:

The enclosed species list identifies threatened, endangered and candidate species, as well as designated and proposed critical habitat that may occur within the boundary of your proposed project and may be affected by your proposed project. The Fish and Wildlife Service (Service) is providing this list under section 7 (c) of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 *et seq*.). Changes in this species list may occur due to new information from updated surveys, changes in species habitat, new listed species and other factors. Because of these possible changes, feel free to contact our office (337/291-3126) for more information or assistance regarding impacts to federally listed species. The Service recommends visiting the ECOS-IPaC site or the Louisiana Ecological Services website (www.fws.gov/lafayette) at regular intervals during project planning and implementation for updated species lists and information. An updated list may be requested through the ECOS-IPaC system by completing the same process used to receive the enclosed list.

The purpose of the Act is to provide a means whereby threatened and endangered species and the habitats upon which they depend may be conserved. Under sections 7(a)(1) and 7(a)(2) of the Act and its implementing regulations (50 CFR 402 *et seq*.), Federal agencies are required to utilize their authorities to carry out programs for the conservation of Federal trust resources and to determine whether projects may affect Federally listed species and/or designated critical habitat.

A Biological Assessment is required for construction projects (or other undertakings having similar physical impacts) that are major Federal actions significantly affecting the quality of the human environment as defined in the National Environmental Policy Act (42 U.S.C. 4332(2)(c)). For projects other than major construction activities, the Service suggests that a biological evaluation similar to a Biological Assessment be prepared to determine whether the project may affect listed or proposed species and/or designated or proposed critical habitat. Recommended contents of a Biological Assessment are described at 50 CFR 402.12.

If a Federal agency determines, based on the Biological Assessment or biological evaluation, that listed species and/or designated critical habitat may be affected (e.g. adverse, beneficial, insignificant or discountable) by the proposed project, the agency is required to consult with the Service pursuant to 50 CFR 402. In addition, the Service recommends that candidate species and proposed critical habitat be addressed within the consultation. More information on the regulations and procedures for section 7 consultation, including the role of permit or license applicants, can be found in the "Endangered Species Consultation Handbook" at http://www.fws.gov/endangered/esa-library/pdf/TOC-GLOS.PDF or by contacting our office at the number above.

Bald eagles have recovered and were removed from the List of Endangered and Threatened Species as of August 8, 2007. Although no longer listed, please be aware that bald eagles are protected under the Bald and Golden Eagle Protection Act (BGEPA) (16 U.S.C. 668 *et seq.*). The Service developed the National Bald Eagle Management (NBEM) Guidelines to provide landowners, land managers, and others with information and recommendations to minimize potential project impacts to bald eagles, particularly where such impacts may constitute "disturbance," which is prohibited by the BGEPA. A copy of the NBEM Guidelines is available at: http://www.fws.gov/southeast/es/baldeagle/NationalBaldEagleManagementGuidelines.pdf. Those guidelines recommend: (1) maintaining a specified distance between the activity and the nest (buffer area); (2) maintaining natural areas (preferably forested) between the activity and nest trees (landscape buffers); and (3) avoiding certain activities during the breeding season. On-site personnel should be informed of the possible presence of nesting bald eagles within the project boundary, and should identify, avoid, and immediately report any such nests to this office. If a bald eagle nest occurs or is discovered within or adjacent to the proposed project area, then an evaluation must be performed to determine whether the project is likely to disturb nesting bald eagles. That evaluation may be conducted on-line at: http://www.fws.gov/southeast/es/baldeagle. Following completion of the evaluation, that website will provide a determination of whether additional consultation is necessary. The Division of Migratory Birds for the Southeast Region of the Service (phone: 404/679-7051, e-mail: SEmigratorybirds@fws.gov) has the lead role in conducting any necessary consultation. Should you need further assistance interpreting the guidelines or performing an on-line project evaluation, please contact this office.

Guidance for minimizing impacts to migratory birds for projects including communications towers (e.g. cellular, digital television, radio and emergency broadcast) can be found at: http://www.fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/towers.htm ; http://www.towerkill.com; and http://fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/comtow.html.

Activities that involve State-designated scenic streams and/or wetlands are regulated by the Louisiana Department of Wildlife and Fisheries and the U.S. Army Corps of Engineers, respectively. We, therefore, recommend that you contact those agencies to determine their interest in proposed projects in these areas.

Activities that would be located within a National Wildlife Refuge are regulated by the refuge staff. We, therefore, recommend that you contact them to determine their interest in proposed projects in these areas.

Additional information on Federal trust species in Louisiana can be obtained from the Louisiana Ecological Services website at: www.fws.gov/lafayette or by calling 337/291-3100.

We appreciate your concern for threatened and endangered species. The Service encourages Federal agencies to include conservation of threatened and endangered species into their project planning to further the purposes of the Act. Please include the Consultation Tracking Number in the header of this letter with any request for consultation or correspondence about your project that you submit to our office.

Attachment(s):

- Official Species List

Official Species List

This list is provided pursuant to Section 7 of the Endangered Species Act, and fulfills the requirement for Federal agencies to "request of the Secretary of the Interior information whether any species which is listed or proposed to be listed may be present in the area of a proposed action".

This species list is provided by:

Louisiana Ecological Services Field Office
200 Dulles Drive
Lafayette, LA 70506
(337) 291-3100

Project Summary

Consultation Code: 04EL1000-2020-SLI-0299

Event Code: 04EL1000-2020-E-00696

Project Name: Mid-Barataria Sediment Diversion

Project Type: ** OTHER **

Project Description: The Proposed Action generally consists of the placement of a sediment diversion through a portion of the federal Mississippi River and Tributaries (MR&T) Project mainline levee on the right descending bank of the Mississippi River (River) at approximately River Mile 60.7 and through the future New Orleans to Venice (NOV) Hurricane Protection Levee, extending into the mid-Barataria Basin in Plaquemines Parish, Louisiana.

Project Location:

Approximate location of the project can be viewed in Google Maps: https://www.google.com/maps/place/29.493020959500058N90.011898939992925W

Counties: Ascension, LA | Assumption, LA | Jefferson, LA | Lafourche, LA | Orleans, LA | Plaquemines, LA | St. Bernard, LA | St. Charles, LA | St. James, LA | St. John the Baptist, LA

Endangered Species Act Species

There is a total of 8 threatened, endangered, or candidate species on this species list.

Species on this list should be considered in an effects analysis for your project and could include species that exist in another geographic area. For example, certain fish may appear on the species list because a project could affect downstream species.

IPaC does not display listed species or critical habitats under the sole jurisdiction of NOAA Fisheries[1], as USFWS does not have the authority to speak on behalf of NOAA and the Department of Commerce.

See the "Critical habitats" section below for those critical habitats that lie wholly or partially within your project area under this office's jurisdiction. Please contact the designated FWS office if you have questions.

---

1. NOAA Fisheries, also known as the National Marine Fisheries Service (NMFS), is an office of the National Oceanic and Atmospheric Administration within the Department of Commerce.

Mammals

NAME	STATUS
West Indian Manatee *Trichechus manatus* There is **final** critical habitat for this species. Your location is outside the critical habitat. ***This species is also protected by the Marine Mammal Protection Act, and may have additional consultation requirements.*** Species profile: https://ecos.fws.gov/ecp/species/4469	Threatened

Birds

NAME	STATUS
Piping Plover *Charadrius melodus* Population: [Atlantic Coast and Northern Great Plains populations] - Wherever found, except those areas where listed as endangered. There is **final** critical habitat for this species. Your location overlaps the critical habitat. Species profile: https://ecos.fws.gov/ecp/species/6039	Threatened
Red Knot *Calidris canutus rufa* No critical habitat has been designated for this species. Species profile: https://ecos.fws.gov/ecp/species/1864	Threatened

Reptiles

NAME	STATUS
Hawksbill Sea Turtle *Eretmochelys imbricata* There is **final** critical habitat for this species. Your location is outside the critical habitat. Species profile: https://ecos.fws.gov/ecp/species/3656	Endangered
Kemp's Ridley Sea Turtle *Lepidochelys kempii* There is **proposed** critical habitat for this species. The location of the critical habitat is not available. Species profile: https://ecos.fws.gov/ecp/species/5523	Endangered
Leatherback Sea Turtle *Dermochelys coriacea* There is **final** critical habitat for this species. Your location is outside the critical habitat. Species profile: https://ecos.fws.gov/ecp/species/1493	Endangered
Loggerhead Sea Turtle *Caretta caretta* Population: Northwest Atlantic Ocean DPS There is **final** critical habitat for this species. Your location is outside the critical habitat. Species profile: https://ecos.fws.gov/ecp/species/1110	Threatened

Fishes

NAME	STATUS
Pallid Sturgeon *Scaphirhynchus albus* No critical habitat has been designated for this species. Species profile: https://ecos.fws.gov/ecp/species/7162	Endangered

Critical habitats

There is 1 critical habitat wholly or partially within your project area under this office's jurisdiction.

NAME	STATUS
Piping Plover *Charadrius melodus* https://ecos.fws.gov/ecp/species/6039#crithab	Final

Appendix B

Basis of Design Report

Rev 1

STATE OF LOUISIANA
COASTAL PROTECTION AND RESTORATION AUTHORITY

MID-BARATARIA SEDIMENT DIVERSION (MBSD) PROJECT
STATE PROJECT No. BA-153
LaGOV NO. 4400010386

Preparation of Engineering and Design
BASIS OF DESIGN REPORT

For

Prepared By:
AECOM Technical Services
7389 Florida Blvd.
Suite 300
Baton Rouge, LA 70806

October 12, 2018

Rev	Date	Description
0	09/07/2018	Draft Submittal
1	10/12/2018	Final Submittal Addressing QRF comments

Rev 1

Prepared in Association With

DISCLAIMER: The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. All product names and trademarks cited are the property of their respective owners. The findings of this report are preliminary, not for construction, and not to be construed as official directives unless so designated by other authorized documents.

DESTROY THIS REPORT WHEN NO LONGER NEEDED. DO NOT RETURN IT TO THE ORIGINATOR.

Bruce R. Lelong, PE Lic #29393

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

1. EXECUTIVE SUMMARY

The Coastal Protection and Restoration Authority (CPRA) has located the Mid-Barataria Sediment Diversion (MBSD) on the West Bank of the Mississippi River (MR) 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 diversion will reconnect the MR 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. The diversion intake is sited at a point bar to facilitate the capture of sand.

The Engineering and Design (E&D) of the MBSD Project is organized into two major phases. Phase 1 is the Basis of Design (BOD) Phase, which comprises alternatives analyses of major diversion components and major appurtenant features, conceptual (i.e., 15%-level) E&D and Class 5 and Class 3 cost estimates in support thereof. Major project features' Design Criteria were established during the BOD Phase. The numerical modeling and E&D performed during BOD Phase were based on available existing data and current conditions, not future conditions-see **Section 3.8** for these definitions. Phase 2 comprises detailed E&D and cost estimates of the selected component and appurtenance alternatives, development of the construction contract documents (plans and specifications); development of the Operations, Maintenance, Repairs, Replacement and Rehabilitation (OMRR&R) Plan, and the preparation and submittals of the Section 408 Permissions Application and Sections 404/10/CUP Joint Permit Application (JPA), along with supporting documents and reports; and the associated regulatory reviews. Phase 2 is divided into 30%-, 60%-, 90%-, and 100%-Level Phases.

Conceptual engineering performed during the BOD phase was initiated using existing data, studies, and reports. Additional data collection was begun during the BOD Phase, including additional geotechnical borings and lab data and additional topographic and geotechnical surveys, but this new data will not be used until Phase 2 design. Existing data has been deemed sufficient for executing the conceptual designs and performing alternatives analyses and selections.

The Basis of Design Report (BODR) and its appendices summarize and document the major project design criteria, the conceptual-level engineering and designs and evaluations of the engineering alternatives, which the Design Team (DT) is scoped to perform, and the conclusions and recommendations arising from the work completed during the BOD Phase. The fundamental goal of the BOD Phase is to develop what its title implies-a Basis of Design-, not the detailed engineering and designs. These will be performed during the subsequent task orders comprising the overall Engineering and Design Phase of the MBSD Project.

All alternatives studied in the BOD phase are documented herein, with the BOD drawings, studies, and reports included as Appendices. In performing the conceptual designs, the DT produced a Design Criteria Document (DCD) which serves to document the design criteria specific to the MBSD project. The DCD was developed with input from CPRA and the U.S. Army Corps of Engineers (USACE), and it is intended to be a living document which will be periodically updated as the design process progresses. The DCD is in **Appendix U**. Of particular note are the following other sections of the BODR:

- The numerical modeling efforts are summarized in **Section 8**; with modeling efforts presented in detail in **Appendix H**.

- Conceptual geotechnical engineering efforts are summarized in **Section 9**; with supporting geotechnical analyses presented in **Appendix G**.
- Conceptual structural designs of the project's hydraulic structural alternatives are summarized in **Section 10**; with supporting structural computations presented in **Appendix J**.
- Cost estimates are summarized in **Section 23**, with detailed back-up in **Appendix F**.

During the BOD Phase, the major project design criteria are established, and major diversion component alternatives are selected. For the three major components of the diversion system, i.e., the headworks, conveyance channel, and outfall, the diversion's invert elevation and basic geometry are selected, along with structure types and typical cross-sections. Major decisions and recommendations involving ancillary features are also made: the height of the riverine and hurricane flood protection, the modifications to the existing interior/site drainage necessitated by the diversion's construction, the alignment and basic geometrics of the new Hwy 23 and New Orleans Gulf Coast (NOGC) railroad bridges crossing the diversion, the features comprising the support facilities and associated site work, the disposition of the utilities/infrastructure crossing the diversion, and the use of excavated earthen materials unsuitable for levee construction and construction fill. These aspects of the Project will be refined and further developed during the detailed engineering and design phase of the Project, which comprises the remainder of the E&D.

CPRA established seven goals for BA-153 MBSD Project. The BOD Phase was organized, scoped and executed to establish a design basis in accordance with the project goals, in conjunction with the land-building modeling and other environmental modeling and engineering and science for the Project EIS:

Table 1-1: Conformance with Project Goals

Goal No.	Goal Description	BOD Phase E&D Role in Achieving Project Goals
1	Reconnection of the MR to the Barataria Basin	Diversion layouts and numerical hydraulic modeling were performed to establish the basis of design of a gravity-driven, controlled conveyance system that delivers sediment-laden, fresh water flows from the MR to Barataria Basin.
2	Establishment of conditions to allow the development of a delta area open to tidal exchanges	This goal will be partially achieved by E&D, starting with the BOD Phase, and partially through the modeling, engineering and science for the Project EIS. The BOD Phase E&D results demonstrate the system can deliver 75,000 cfs of diversion flow with a favorable sediment to water ratio (SWR), and by conceptually designing a conveyance channel that maintains velocities sufficient to keep sediment in suspension. The Basin's land-building management is addressed by the Basin-wide and Outfall Management modeling being performed by The Water Institute of the Gulf (TWIG), and by the development of the Environmental Impact Statement (EIS) by the EIS Team. (Documentation of the TWIG modeling and EIS development is not included in the BODR.)

Table 1-1: Conformance with Project Goals (Continued)

Goal No.	Goal Description	BOD Phase E&D Role in Achieving Project Goals
3	Development of the initial basis of design using 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(s) of the diversion structure	BOD Phase numerical hydraulic modeling of alternatives demonstrates the alternatives being considered achieve 75,000 cfs of diversion flow for current conditions (see Section 3.2-Key Definitions for definition of current conditions). Engineering and design for future conditions will be performed during the 30% Phase.
4	Maintenance of the current level of riverine and hurricane flood risk reduction	Investigations and designs of the MR cofferdam alternative, tie-in flood protection, and Hurricane/Guide Levees follow Project Design Criteria, which were established to provide current level or better flood risk reduction.
5	Development of designs of the major diversion components and appurtenances to maximize sediment capture, maximize flow efficiency, and allow for operations adaptability based on monitoring data collected during project operation, while minimizing OMRR&R	This goal is generally the fundamental consideration used to develop the decision matrices to select alternatives under consideration. The strategies and processes to use monitoring data to be collected during operations in order to allow for operations adaptability will be addressed in Phase 2.
6	Conformance to state and federal design criteria and environmental compliance requirements as required to achieve project regulatory approval	Conformance to state and federal design criteria initially is documented during BOD Phase with the Project Design Criteria, which is reviewed by CPRA and USACE. In subsequent phases, the Section 408 review process will confirm and document that the design conforms to state and federal design criteria by project milestone reviews and Section 408 and JPA reviews by regulatory agencies and stakeholders. The NEPA process (not part of the E&D scope) will confirm that the project conforms to environmental requirements.
7	Development of an operational plan for the diversion structure	The Operational Plan is not part of the BOD Phase, other than to establish an initial range of diversion flows over which alternatives will be evaluated. The Operational Plan will be addressed in detail in Phase 2 of E&D.

In conformance with Goal No. 5, the BOD Phase was structured around an alternatives and evaluation screening process with two decision-points, alternatives-selection workshops during the BOD Phase. A third workshop will be held during the 30% Phase, during which the enlargement of the intake type selected during BOD Phase will be confirmed. The structure to the alternatives screening process is shown in **Figure 1-1**.

The two BOD Phase workshops are summarized in greater detail in **Section 7**. At Workshop No. 1, potential alternatives to be conceptually engineered and evaluated were identified, ranked and selected using decision matrices with qualitative scoring criteria. The diversion components for which potential alternatives were as follows: the River Intake-structure type and invert elevation, the Conveyance Channel-channel invert and invert profile, Back Gate vs. No Back Gate at Outfall but with parallel, dual-purpose Hurricane/Guide Levees, and Interior Drainage System modification alternatives to accommodate the diversion's disruption of existing drainage patterns.

The Intake Alternatives chosen at Workshop No. 1 to be conceptually designed and numerically modeled during BOD Phase were four structures types: open channel type, U-Frame, U-Frame with interior walls, and a submerged culvert. Three invert elevation alternatives also were selected: -20, -40, and -50, for a total of eight intake alternatives. **(Note: All elevations referred to in this report reference NAVD88 unless specifically noted otherwise.)** EL -40 was selected for evaluation because that is the elevation selected and engineered during previous designs. EL -50 was selected for evaluation because that is the deepest that workshop participants judged could be constructed within the construction budget and with acceptable risk, and EL -20 was selected for evaluation because that is the shallowest elevation participants judged could capture sufficient sand. All four structure types were chosen to be evaluated for EL -40, again because it was the invert elevation used in previous design, while two structure types were chosen to be modeled each at EL -50 and EL -20. The best performing open type as determined by modeling of EL -40 alternatives and the submerged culvert type were selected for EL -50. The best two performing open configuration types were chosen to be evaluated at EL -20.

The Conveyance Channel Alternatives chosen to be evaluated were two Channel predominant invert elevations (EL -20 and EL -25) and three invert profiles for each predominant invert elevation, for a total of six alternatives.

It also was decided at Workshop No. 1 that the Back Gate versus No Back Gate with Hurricane/Guide Levee alternatives comparison considered only these two alternatives. The Interior Drainage System Modification Alternatives analysis was decided to include a comparison of the 2014 Base Design's proposed drainage pump station versus an inverted siphon system.

It was decided for other appurtenant project features that 15%-level engineering would proceed but changes to these design concepts compared to the 2014 Designs would not be done through a decision matrix scoring process: Diversion Gate type selection, whether there is a requirement and need for an onsite, dedicated crane at the Diversion Gate Structure for emergency situations, river and channel armoring systems, Outfall Transition Feature geometric design, selection of the alignment of the proposed railroad bridge over the diversion, Hwy 23 Bridge layout and alignment; selection of secondary site features, and the identification of beneficial uses of excavated earthen materials unsuitable for levee construction or use as fill material. The 15% E&D for those items is documented in the BODR main body and appendices.

Referring to **Figure 1-1**, numerical modeling and conceptual engineering progressed after Workshop No. 1, and the results and conclusions were used as a basis of ranking at Workshop No. 2. First numerical modeling of the Intake alternatives was performed using FLOW-3D hydrodynamic modeling with particle tracking with 1,000,000 cfs of MR flow, current conditions, and steady state. Energy losses and SWRs were computed. Numerical modeling of the Conveyance Channel was performed using Delft3D and Coastal Modeling System (CMS) modeling. Back Gate modeling also used Delft3D and CMS. Civil layouts, geotechnical analyses and designs, and structural designs of the major diversion component alternatives and related features progressed; quantity take-offs were performed and Class 5 cost estimates were prepared for the alternatives under consideration. E&D of the other features not topics of the second alternatives workshop continued. Prior to Workshop No. 2, decision matrices with evaluation criteria were developed. The engineering and designs are documented in this BODR.

At Workshop No. 2, the results of the modeling, designs, and estimated life cycle costs for these alternatives were used to score and rank them in decision matrices with a combination of quantitative scoring criteria and semi-quantitative scoring criteria. The following selections were made:

1) Intake Alternative-Open Channel with Invert at EL -40 appears preferable but would be confirmed with additional H&H modeling for medium and low operating flows;
2) Conveyance Channel with invert at EL -25, with a constant, flat invert to the end of the Channel, and beyond sloping upwards to prevailing mud bottom in the Basin in the Outfall Transition Feature;
3) Hurricane Flood Protection-elimination of the Back Gate Alternative and selection of the Hurricane/Guide Levee Alternative chosen to provide hurricane storm damage risk reduction. This decision will be evaluated by the USACE as part of the Section 408 Permissions review. CPRA and the DT will perform a risk analysis according to USACE policies and procedures, comparing the risk of Hurricane/Guide Levees to the risk to the federal NOV-5a Levee without a diversion crossing its alignment. The CPRA will submit a risk assessment report documenting the analysis. The Hurricane/Guide Levee alternative will not be objectionable provided the analysis demonstrates incorporation of the Hurricane/Guide Levees into the NOV-5a line of protection does not increase the risk to the system;
4) Interior Drainage-an Inverted Siphon near the Timber Canal crossing beneath the Conveyance Channel.

Regarding the intake alternatives, it was decided at Workshop No. 2 that the selection of the Intake Alternative would be formally made after additional numerical modeling that considered medium and low MR flows as snapshot assessments of the selected intake alternative's performance to gage how the alternatives under consideration perform corresponding across the range of operational flows. FLOW-3D modeling, hydrodynamic only, no particle tracking, was done at low flows. Delft3D modeling was done for each alternative at high, medium, and low flows. When the post-Workshop No. 2 H&H modeling had been completed, the Intake Selection Matrix categories and their weightings were finalized and the alternatives were scored. The Open Channel with Invert EL -40 is the preferred selection. This is summarized in **Section 7**.

H&H conceptual design of modifications to the site (interior) drainage network performed during the BOD Phase consisted of designing a siphon bank to drain the upriver polder to Wilkinson Pump Station, located in the downriver polder at the Wilkinson Canal. The siphon's required capacity was assumed to need to match the established capacity for the 2014-proposed pumping station, which was part of the 2014 30% design scope. The pump station's purpose as well as the siphon's is to drain the upriver polder. Conceptual civil and structural layouts, designs, and associated geotechnical analyses were performed to develop drawings and prepare quantity take-offs and cost estimates. These were compared to the 2014 30% designs and cost estimate, with unit prices updated. Based on that comparison, the siphon alternative was selected at Workshop No. 2 based on anticipated cost savings, and the pump station was eliminated from further consideration. The use of the 2014 pump station design's required capacity as the basis for conceptually designing the inverted siphon alternative was done for the purposes of alternative selection. The detailed design of the siphon will be based on the computed required capacity determined from the area-wide HEC-RAS modeling.

After Workshop No. 2, the following decisions and recommendations were made apart from the Workshop-based alternatives evaluation process:

1) The Diversion Gate type should be a tainter gate. See **Appendix O**.
2) There is no USACE specific requirement that a dedicated on-site crane be installed at the Headworks (HW). CPRA should develop a specific operational strategy for emergency and planned maintenance situations, at which times a crane will be mobilized to the site. For example, putting in place an emergency contract so that a crane will be available and mobilized quickly to the site. See **Appendix P**.
3) The River Intake segment between the MRL and the Diversion Gate Structure should be a U-frame structure type without interior walls, except beneath the R/R Bridge and directly in front of the Diversion Gate Structure. This means that the U-Frame portion of the Intake will have a concrete floor, not riprap armoring.
4) The recommended river armoring system is riprap. See Section **10.4.6**.
5) The final selection of conveyance channel armoring system will be made during the 30% Phase after further design progression.
6) The railroad bridge alignment will be over the Intake U-Frame in line with the existing track. The low chord of the bridge will be at EL 16.4 or higher, and will not be a flood-proof bridge. See **Section 14**.
7) The Hwy 23 Bridge will be constructed along the current alignment of Hwy 23. See **Section 13**.
8) The Outfall Transition Feature will be approximately 1,500 feet long. The analysis and hydraulic design are discussed further in the Executive Summary and summarized in **Section 8**.
9) Proposed secondary site features and facilities are described in Sections 16 and 20. CPRA provided owner information about needs and preferences in mid-August, 2018. Based on this guidance, layout drawings are being developed and included in the update to this report. See **Sections 16 and 20**.
10) Beneficially used earthen materials unsuitable for levee construction and use as construction fill will be used to reconstruct a ridge on the north side of Wilkinson Canal in the Basin, and to fill a designated area near Bayou Dupont on the north side of the Outfall Transition Feature to construct wetlands. This is discussed further later in the Executive Summary and summarized in **Section 24**.

Subsequent to the selection of the Intake type and invert elevation, BOD Phase numerical modeling efforts concluded with starting the numerical modeling for future conditions to evaluate the sizes of the major diversion features (see **Section 8.9**) and concurrent investigations of geometric optimizations of the selected Intake Alternative (Open Channel with Invert EL -40) based on modeling of current conditions (see **Section 8.5.7)**. The three-component diversion system with the selected alternative was incorporated into the TWIG OMBA model along with the land-building topography and bathymetry from the Basin-wide model at Year 50 to create the FTN OMBA model. The FTN OMBA model has higher mesh resolution in the 3 diversion components. Tailwater conditions were derived from the Basin-wide model's offshore boundary. The model was run using a one-year MR hydrograph. The modeling results showed the system does not produce 75,000 cfs of flow at 1,000,000 cfs in MR. Therefore, the need to upsize the diversion system for future conditions to achieve 75,000 cfs of flow with 1,000,000 cfs of river flow has been established. The initial future conditions modeling is documented in this BODR. The initial future conditions modeling is the extent of modeling and E&D included in the BODR.

Four HW optimization simulations for the selected intake type and invert elevation were completed for the Open Channel, Invert EL -40 Intake, using FLOW-3D to model current conditions. Delft3D results are not included in the report; however they will be included in an update to this report. The objective of the optimization testing was to determine if the head loss through the system could be decreased by modifying the intake geometry without decreasing the SWR. The optimizations modeled were combinations of widening the River Intake geometry by increasing the flare angle of the training walls, removing the interior divider walls of the U-Frame segment of the Intake between the MRL and the Diversion Gate except beneath the railroad bridge and immediately in front of the Diversion Gates, an upwardly sloping Intake invert to the Diversion Gate with its sill set at the invert of the Conveyance Channel, and installing riprap armoring within the geometric U-Frame segment in lieu of a structural concrete U-Frame segment. The optimizations did not include improvements to the HW Discharge Transition geometry to improve hydraulic performance. Transition alternative geometries will be modeled after the BODR. Based on the computed energy losses and SWR performance of the four alternatives, Optimization 1a is recommended, but the results need to be confirmed by the Delft3d modeling results. Optimization 1a has a wider River Intake, an Invert at EL -40 through the entire Intake and Diversion Gate Structure, and U-Frame divider walls removed, except under the railroad bridge and immediately in front of the Diversion Gates. Optimization 1a reduces energy losses by 42% for river operational high flow and by 49% for river operational low flow. The Overall Sand SWR ratio, as computed by FLOW-3D, decreases by 7% compared to the base geometry; the overall decrease is driven by the decrease in SWR for the 250 **μ** grain size, which decreases by 18%. However, SWRs for grain sizes 125 **μ** and smaller are essentially the same. Base geometry is the geometry modeled for the screening of Intake Alternative Types and invert elevations. See **Section 8.5.7** for further discussion.

Hydraulic modeling will continue beyond the BOD Phase into the beginning of the 30% design phase, and this BODR will be updated to include optimizations of the three major diversions components and the additional findings and results. During the 30% Phase, a study will be performed to evaluate HW upsizing, coupled with assessing the benefits of maintenance dredging in the Basin to manage tailwater elevations in order to achieve target flows. The alternatives will be evaluated using life cycle cost estimates and the alternative chosen will be the upsized intake that will progress to final engineering. The upsized diversion system will be modeled using a 50-Year hydrograph to confirm target flows are achieved, to compute a cumulative SLR, and as the basis to modify dimensions to improve performance. Shoaling and scour near the Point Bar will be evaluated to assess Point Bar Stability and potential impacts to the MRL. The size and performance of the diversion's major components-HW, Conveyance Channel, and Outfall Transition Feature-based on this modeling, will be confirmed at Workshop No. 3 to be scheduled during the 30% Phase.

Two scaled physical models were designed in BOD Phase. A flume test was then performed and the results and findings summarized in a Flume Test Report included in **Appendix H**. The models' construction is ongoing. Testing will occur in the 30% Design Phase. Comparisons to the numerical modeling results are anticipated to occur during both 30% and 60% Design Phases.

Sections 11 and 10 of the BODR present two alternative levels respectively for hurricane and riverine storm/flood damage risk reduction. The hurricane flood protection components, i.e., Hurricane/Guide Levees and T-Walls, were conceptually designed for the 50-Year level, projected 50 years into the future, EL 15.6, storm surge coming from the Basin. Designs prorated for a lower level of protection, EL 12.1, which is approximately a 25-Year level of protection, 25 years into the future. The DT estimates that there will be a sufficient quantity of excavated earthen materials for levee construction to construct either alternative without having to important levee fill. The cost to construct the Hurricane/Guide Levees and T-walls to EL 15.6 is approximately $18 million more than constructing these levees to EL 12.1. The DT recommends that the Guide Levees and T-walls serving as hurricane protection be constructed to EL 15.6. See **Section 11** for further discussion.

The HW components in the line of MR protection were conceptually designed to the currently authorized MRL elevation at the project site, EL 16.4 and prorated for EL 20.1, which corresponds to the 50-Year level of hurricane protection, projected 50 years into the future, storm surge coming from the river. The MRL at this location is currently not authorized as hurricane flood protection. Incremental construction costs are presented in the Cost Estimates in **Appendix F**. The DT estimates that constructing to EL 20.1 will cost approximately $3.5million more than constructing to EL 16.4. The DT recommends that the riverine protection be constructed to EL 20.1. See **Section 10** for further discussion.

After Workshop No. 2, the designs of the interior drainage modifications did not advance because of lack of access to the Wilkinson Pump Station, which is needed to obtain intake basin water level data during a rain event to calibrate the HEC-GeoRAS model for existing (pre-project) conditions. This work will recommence after access to the station is obtained, likely to be in Phase 2. The BOD Phase H&H work is summarized in **Section 8**. The conceptual structural design of the siphon system is presented in **Section 10**.

It was decided during the BOD Phase, that utility relocation dispositions will be established during the 30% Phase. It is anticipated that buried utilities crossing the conveyance channel along Hwy 23 will be relocated to be mounted on the proposed Hwy 23 Bridge over the conveyance channel. It is also anticipated that the Shell 20" Nairn crude pipeline will need to be relocated beneath the Outfall Transition Feature by directional drilling prior to construction. Utilities and their respective points of contact are listed in **Section 19**.

The proposed auxiliary structures and site features comprising "Secondary Site Features" are described in **Section 20**.

The BODR includes design concepts for the beneficial placement of excavated and dredged materials unsuitable for use for levee construction and construction fill. The unsuitable material will be the top feet of the HW and Conveyance Channel excavations and the dredged material from the Outfall Transition Feature's construction. The unsuitable material will be used to construct a ridge along the north side of Wilkinson Canal in the Basin, which will reduce the siltation within the canal from diversion operation. The other area where earthen material will be placed and wetlands constructed is in the Basin on the north side of the Outfall Transition Area. Designs will follow CPRA guidelines. See **Section 24,** which also lists the estimated quantity of unsuitable material.

Near the end of the BOD Phase, a Class 3 construction cost estimate was prepared for the overall project, reflecting the components selected at Workshop No. 2. Escalated to the mid-point of construction, the estimated cost inclusive of contingencies is $984.2 million. The escalation factor used is 15%. Contingency percentages selected vary by feature from 25% to 40%, but typically are 30%. The Class 3 estimate does not include enlarging the River Intake for future conditions. If it is determined to be needed based on the results of numerical modeling, the estimated cost to upsize the River Intake will be included in the BODR Update.

The current estimate includes the hurricane flood protection features constructed to design grade of EL 15.6, inclusive of construction overbuild of levees, and EL 16.4 for the headworks (HW) components forming part of the MRL line of riverine flood protection. If the hurricane flood protection features are constructed to EL 12.1, the construction cost decrease by $17.9 million. If the riverine flood protection features are constructed to EL 20.1, the construction cost increases by $3.0 million. These estimated incremental costs include contingency. See **Section 23** for further information.

Table 1-2: Selected Alternative Components-Estimated Construction Cost Summary

SELECTIVE ALTERNATIVE COMPONENT SUMMARY		
ID	Alternative Feature Description	Total Cost with Contingency
1	Open Cut U-Frame Intake, No Interior Walls, Top of Wall El 16.4	$245,010,743
2	Gated Diversion Structure, Top of Wall 16.4	$61,031,245
3	Transition and Wingwalls @ EL -40 to EL -25, Top of Wall EL 15.6	$53,244,418
4	Railroad Bridge (Low Chord at EL 16.4)	$44,388,923
5	Hwy 23 Roadway and Bridge (300' wide channel)	$53,249,158
6	Channel and Levee (TOL EL 15.6 to NOV, EL 11.5 to Back Levee, 300' Wide Channel at EL -25.0)	$258,752,377
7	Interior Drainage	$28,073,199
8	Secondary Site Features	$4,574,804
9	Utility Relocations	$32,955,000
10	Temporary Construction Features	$30,215,510
11	Beneficial Use Material	$997,500
12	Allowance for Flooding of the Cofferdam During a Hurricane	$2,000,000
13	***Construction Subtotal***	***$814,492,876***
14	Mobilization and Demobilization (3%)	$24,434,786
15	Misc. Insurance Hurricane & Builder's Risk Ins. (1%)	$8,389,277
16	Payment and Performance Bond (1%)	$8,473,169
17	Subtotal (Sept 2018 estimate including contingencies)	$855,790,108
18	15% escalation (Escalation Cost to mid-point construction Dec 2023):	$128,368,516
19	***Total Cost with Escalation and Contingencies:***	***$984,158,625***

Table 1-2: Selected Alternative Components-Estimated Construction Cost Summary (Continued)

OPTIONS TO THE SELECTED ALTERNATIVE		
ID	Add/Deducts	Total Cost with Contingency
1	Increase MRL Structures Height from EL 16.4 to EL 20.1	$3,040,089
1.1	Intake Structure to EL 20.1	+ $1,094,054
1.2	Gated Structure to EL 20.1	+ $1,428,034
1.3	MRL Wall to EL 20.1	+ $518,001
2	Decrease Channel Levees Height from 15.6 to 12.1	-$17,941,689
2.1	Transition Walls to EL 12.1	-$6,805,174
2.2	Top of Levee at EL 12.1	-$7,054,645
2.3	Hwy 23 T-walls at EL 12.1	-$2,040,935
2.4	Siphon T-walls at EL 12.1	-$2,040,935

In conclusion, the following noteworthy actions should be taken during 30% Design Phase to fully establish the Basis of Design.

1. Confirm the DT-recommended Intake optimizations as described previously in the Executive Summary considering the Delft3D modeling results. Note that geometry will be further refined during 30% Design Phase.
2. Select the magnitude of upsizing of the HW to meet Project Goals for future conditions, considering associated maintenance dredging needed in the Basin to manage future tailwater elevations.
3. Perform the assessment of point bar stability, and river scouring and shoaling, after the HW have been upsized for future conditions, based on upcoming modeling.
4. Assess water quality in the Conveyance Channel under maintenance flows.
5. Confirm the design grade of the HW MR flood protection components, which the DT recommends be set at EL 20.1. Note that final design grade may be based on wave overtopping, which will be assessed by near-field storm surge modeling of the Conveyance Channel.
6. Confirm the design grade of the hurricane flood protection features (protecting against storm surge from the Basin). Note that the decision may be influenced by the upcoming Risk Assessment being performed as a USACE requirement to approve the use of the Guide Levees to provide hurricane flood protection.
7. Finalize selection of the Channel armoring system type.
8. Confirm the low chord elevation of the railroad bridge, particularly if the decision is made to design the HW components tying into the MRL line of protection to EL 20.1.
9. Finalize the size of the inverted siphon(s) after the HEC-GeoRAS interior drainage model is calibrated.
10. Confirm the layout and sizes of the secondary project features to reflect CPRA guidance received in mid-August, 2018. This is expected confirmed after the DT submits layout drawings during the beginning of 30% Design Phase.

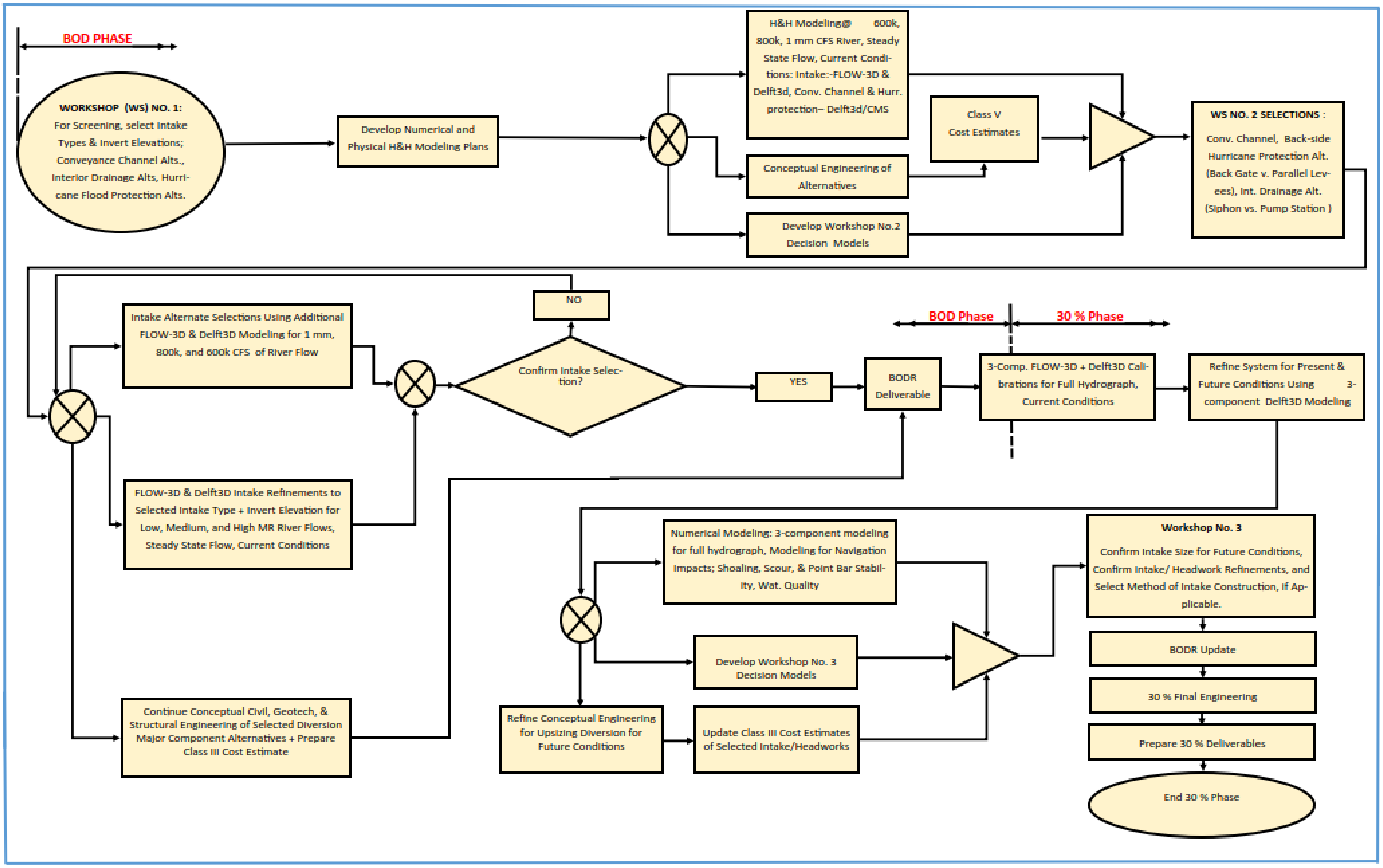


Figure 1-1: Flowchart of Alternatives Screening Process

2. PROJECT LOCATION

The project is located on the West Bank of the MR, in Plaquemines Parish, Louisiana, south of the Phillips 66 Refinery, near the town of Ironton, as shown in Figure 2-1. The proposed diversion intake is located at Mississippi River Mile (RM) 60.7 Above the Head of Passes (AHP) and intersects the MRL at Station 1109+58. The proposed diversion channel extends in a southwest direction where it will bisect the New Orleans to Venice (NOV) back levee, Reach NOV-NF-W-05c.

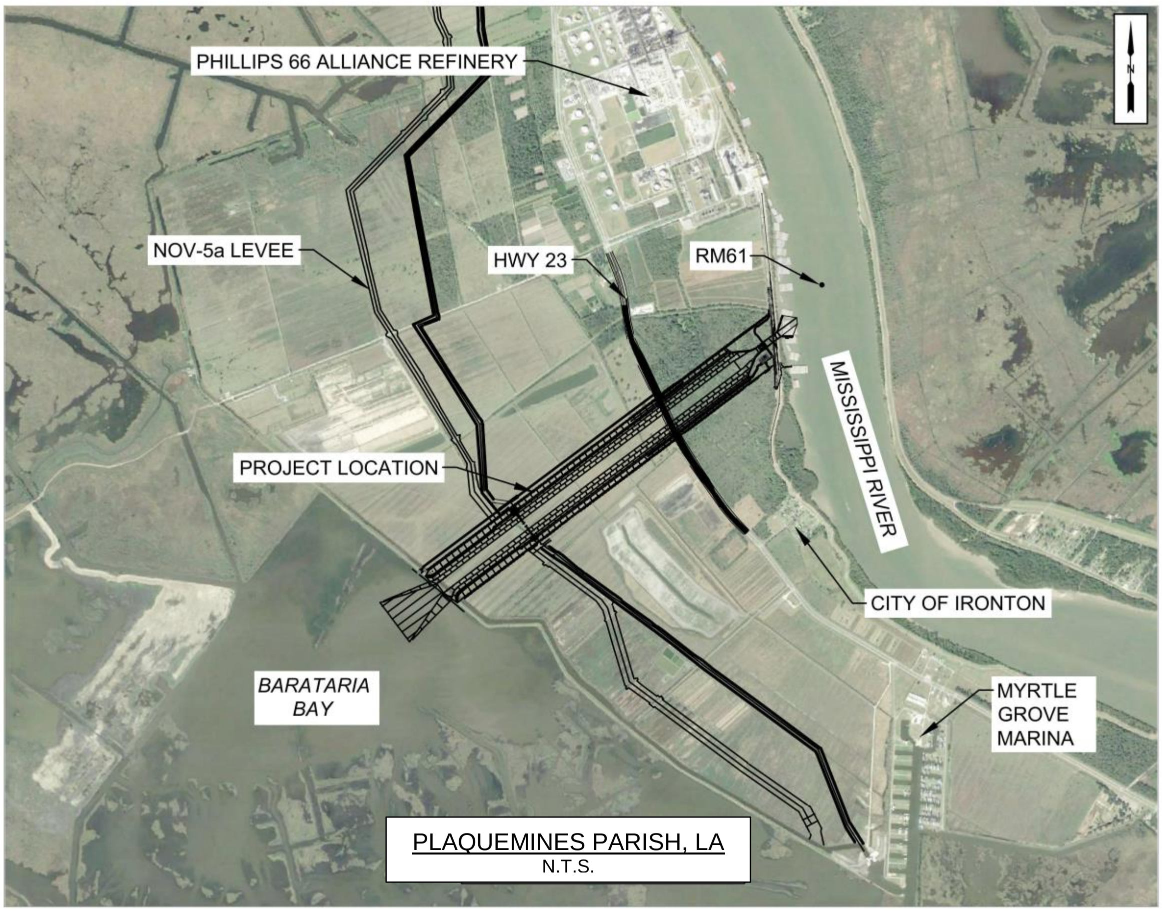


Figure 2-1: MBSD Project Location Map

Note: Other maps and drawings showing the project features to scale are included in **Appendix D**.

3. GENERAL

3.1 Project Goals and Description

The MBSD Project is one of two projects which comprise CPRA's Mississippi River Mid-Basin Sediment Diversion Program. 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
- Use, as an initial basis of design, 75,000 cubic feet per second (cfs) flow through the Conveyance Channel from the 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 NOV levee
- Design the Intake Structure, control structure, channel, and appurtenances to maximize sediment capture, 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 sediment conveyance system is divided into three sections; intake, conveyance, and discharge. The intake consists of an Intake Structure, gated diversion and transition. 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. Project components not directly related to sediment conveyance include: Hwy 23 Bridge and Roadway Realignment, Railroad Relocation (conceptual only for BOD Phase), Interim Flood Protection measures, Interior Drainage including a Siphon, Utility Relocations, and Secondary Project Features such as support buildings and a boat ramp. An overall site map is included in Appendix D, Selected Alternative Plans, Drawing #G-009.

3.2 Purpose and Goals of the Basis of Design Phase and Report

CPRA is executing the E&D services for the MBSD project in two phases: a BOD Phase, in which the DT performs 15%-level alternatives analyses, and Phase 2, which will include detailed E&D of the diversion, permitting support, and coordination with the Construction Manager at Risk (CMAR). The BODR documents the work performed in the BOD Phase.

The BOD Phase comprises 15% level-of-completion of alternatives analyses of major diversion system components to identify and select alternatives that improve sediment capture and transport capabilities, improve hydraulic performance and produce life cycle cost savings relative to previous 30%-level designs completed in 2014. Initial Project Design Criteria were established in accordance with project goals. Hydraulic modeling and E&D of the selected major diversion components to finalize their sizes and geometry will continue after the BOD Phase. The BODR serves to document the modeling and conceptual designs of the alternatives under consideration, and the screening process by which certain conceptual alternatives were selected. Alternatives evaluated included River Intake configurations and their invert elevations, Conveyance Channel inverts and profiles, HW structural components, levee and floodwalls, railroad and highway bridge alignments, and Interior Drainage modifications. Selection matrices were developed to evaluate River Intake alternatives, Conveyance Channel alternatives, and Interior Drainage modification alternatives. A matrix-based screening process for the Intake alternatives was developed to compare by scoring system established for their respective sediment capture and transport performance, hydraulic performance, cost, adaptability, and risk. The screening process for the Conveyance Channel compared hydraulic performance and cost. The other component alternatives were selected through individual studies, which are discussed in this BODR, but were not scored using evaluation matrices.

Hydraulic modeling of the major component alternatives has been completed for existing boundary conditions, and the results of those models are discussed in this BODR. During BOD Phase, evaluation of alternatives for hydraulic performance 50 years after the start of diversion operation, i.e., for future conditions, was limited to identifying the estimated future net potential energy head differential between the Mississippi River and Barataria Basin, as computed by TWIG's Basin-Wide Model, and comparing this future available net head to the computed head losses in the diversion system for each alternative. Systemic head loss exceeding the estimated, future available net head was considered a fatal flaw, and eliminated one of the Intake Types (submerged culvert alternative) from consideration.

At the end of the BOD Phase, an initial assessment was made of the capability of the diversion system, as sized for current conditions, to meet project goals in the future. The selected, major diversion components were modeled in a three-component model (i.e., HW, Conveyance Channel and Outfall Transition Feature) for Year 49 conditions, as imported from TWIG's Basin-Wide Model, and the results indicate that the selected components sized only for current conditions will not deliver 75,000 cfs of diversion flow with 1 million cfs of flow in the Mississippi River. The evaluation determined that the Conveyance Channel is properly sized, but the River Intake needs to be widened to some extent, currently undetermined. The next step is to perform more extensive hydraulic modeling for future boundary conditions, during which the Intake will be incrementally enlarged. This will be done also in conjunction with modifying the Basin's built land topography, as imported from TWIG's model, to include dredged distributary channels that lower tailwater elevations. The upsizing will be established through a life cycle cost comparison of various combinations of upsizing and future dredging to select an intake size that can be constructed within budget and whose associated Basin maintenance dredging is manageable for CPRA. The selection will be made through a final screening process.

The upcoming modeling and cost comparisons for future conditions will be documented in a forthcoming update to this BODR, which will occur during the 30% Phase of E&D. Starting in 30%, hydraulic modeling will include modeling for various operational scenarios in support of developing an operational plan. The detailed E&D of the Diversion components and associated features will be documented in a Design Documentation Report (DDR) during Phase 2 of the Project.

3.3 Report Structure

The BODR documents the engineering alternatives analysis performed during the BOD Phase of the MBSD project. The report narratives describe design processes and methods, summarize documents included in the appendices, and recommend selected alternatives. The appendices contains documents such as technical memos, reports, modeling plans and other documents previously submitted as deliverables during the BOD Phase, and conceptual drawings depicting both the selected alternatives and the eliminated alternatives. Specific appendices are referenced within the report sections as appropriate.

3.4 Design and Service Life

As directed by CPRA, the design life for the MBSD is forecasted to be 50 years, with a service life of 100 years. Ultimately, hydraulic modeling will be performed with future conveyance boundary conditions established at 50 years from project completion (2074).

3.5 Previous Reports and Studies

In 2014, CPRA contracted with a design consultant who completed a 30% BOD (herein referred to as the 2014 Base Design), which included reports and preliminary drawings. Although these deliverables were labeled as a 30% level of completion, CPRA recognized that not all of the designs were actually completed to a 30% level. CPRA tasked the DT with reviewing and verifying the feasibility of the BOD, and developing new concepts to save costs or add value to the project.

In addition to the 2014 Base Design Report, other previous reports and studies are referenced throughout the BODR, such as studies performed by TWIG, USACE, or CPRA. These publications are also documented as references in the reports and studies prepared for the MBSD project, which are included in the Appendices.

3.6 Existing Data

While efforts to obtain current data are ongoing throughout the BOD Phase, existing data was used to initiate conceptual designs and perform alternatives screening. Examples of existing data include the following:

- Results from previous geotechnical borings and testing
- Hydraulic models performed by TWIG and CPRA
- Aerial imagery
- LIDAR data
- 2013 River Bathymetry
- 2017 USACE Revetment Surveys
- TWIG's Mississippi River Sediment Data at the MBSD site

For further information regarding specific data used in the geotechnical analyses and hydraulic modeling, refer to **Appendix G** and **Appendix H**.

3.7 Site Conditions

The MBSD will span from the Mississippi River to Barataria Bay, intersecting the MRL, a railroad crossing, Hwy 23, an existing back levee, drainage ditches, and two drainage canals, the Timber Canal and Back Levee Canal. A river barge fleeting area, with mooring monopoles, is located along the river's right, descending bank, where the Intake Structure will be located. The barge fleeting area extends both upriver and downriver of the Intake location. The existing MRL crown elevation is approximately EL 15.5, but is authorized to EL 16.4. An existing railroad track, operated by NOGC Railway, is located at grade, on the protected side of the MRL, and terminates just south of the MBSD site. The wooded fastlands reach stretches from the MRL to Hwy 23, which is an at-grade 4-lane divided highway running in a north-south direction.

As part of the Plaquemines Port & Harbor Terminal District, a liquid petroleum products tank farm and marine export terminal are being considered for siting on the upriver side of the MBSD, immediately adjacent to the project. The project does not currently have necessary permits for construction and operations. The JPA has been filed. Numerical hydraulic modeling and conceptual designs have been performed independent and does not include any potential site development at the proposed terminal location.

Between Hwy 23 and the back levee, the existing site is mostly comprised of borrow pits and drainage ditches. The downstream end of the proposed Conveyance Channel will intersect the existing back levee, for which USACE is planning to construct improvements. This project, titled NOV-NF-W-05a.1 LaReussite to Myrtle Grove, originally called for the enlargement of the existing levee near its existing alignment. However, USACE is currently considering shifting this alignment close towards Hwy 23. After USACE announces their final alignment decision, the MBSD Conveyance Channel Levees will be designed to tie-in to the chosen levee alignment.

3.8 Key Definitions

Current Conditions-Current Conditions are present Mississippi River water surface elevations near the diversion intake (headwater) and water surface elevations in the Barataria Basin (tailwater). In the numerical models, these water surface elevations are specified as boundary conditions obtained from the results of the basin-wide Delft3D Model run PR15 1.5m SLR results completed by TWIG. Sediment boundary conditions are also obtained from the same TWIG model. The Mississippi River bed morphology is taken from the TWIG Outfall Model Barataria (OMBA) model and the bathymetry from USACE 2013 multibeam and USACE 2017 revetment surveys.

Future Conditions-Future Conditions include Mississippi River headwater elevations and Barataria Basin tail water elevations anticipated to occur during Diversion operational scenarios being used in the hydraulic design of the three components of the Diversion, i.e., the HW, Conveyance Channel, and the Outfall Transition Feature. The future water surface elevations are those anticipated to occur 50 years after commencement of Diversion operation. Future conditions account for the predicted amounts of SLR, land-building, and the regional soil subsidence in the Barataria Basin at Year-50. The rates of SLR and regional soil subsidence used for modeling of future conditions are specified by CPRA. Similar to the Current Conditions, these conditions are obtained from Year-50 of the TWIG basin-wide Delft3D model run PR15 1.5m.

Headworks-The Diversion HW comprise the River Intake/Inlet, the Diversion Gate Structure, the Diversion Gate Structure's discharge transition to the full trapezoidal Conveyance Channel, and MRL flood protection tie-in features. Ancillary features of the HW include access to the Diversion Gate Structure, localized scour protection armoring, toe sheeting, localized under-seepage reduction features, ground improvement/strengthening, and integral piers/bents for the railroad bridge crossing the Intake.

Outfall Transition Feature-The Outfall Transition Feature, aka, Outfall Transition Area, Outfall Apron or Outfall Ramp, is the pre-dredged flared geometric transition from the end of the fully trapezoidal Conveyance Channel where it crosses the existing non-federal NOV Levee alignment, and upwardly slopes from the invert of the Conveyance Channel, to prevailing mud bottom grade in Barataria Basin. As the feature becomes shallower it becomes wider. The dimensions of the Outfall Transition Feature are determined by discharge conveyance efficiency and sedimentation considerations.

4. SURVEY DATUM AND INFORMATION

4.1 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.

4.2 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) and were also used in the 2013 survey.

4.3 Project Surveys and Imagery

Survey data being obtained during the BOD Phase (will not be used for design until Phase 2) 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

Detailed information is provided in the Survey Report in **Appendix K.**

5. PROJECT DESIGN CRITERIA

The DT developed an initial MBSD DCD which will serve as a record of the design criteria used during the design process. This initial document, which is included in **Appendix U**, will be updated by the DT as the design work progresses. During the BOD Phase, design criteria have been developed for the following disciplines and components:

- Geotechnical Engineering of Major Diversion Components
- Hydraulic Structural Engineering of Major Diversion Components
- Hydraulics and Hydrology, including Site Drainage
- Hwy 23 Bridge and Approaches
- Railroad Bridge
- MRL, Conveyance Channel, Outfall, and Channel Armoring

Design criteria for the Marine Structures, Mechanical, Electrical, Instrumentation & Controls, Architecture, and Secondary Site Features will be developed after the BOD Phase is completed, and the DCD will be updated accordingly.

6. PROJECT DESIGN GRADES

The DCD (**Appendix U**) identified the project design grades for various current and future design years for flood protection features. Table 6-1 summarizes the design grades under consideration for the MBSD project and used during the BOD Phase.

Table 6-1: Design Grades for Project Features

MBSD Reach	Design Grade (NAVD88 2009.55)
MRL – Riverine Design Grade	EL 16.4
MRL – Hurricane Design Grade	EL 20.1
HW Structures tying into MRL line of riverine flood protection	EL 16.4 (or EL 20.1)
HW Discharge Transition Walls and Conveyance Channel T-Walls	EL 15.6 (or EL 12.1)
Conveyance Hurricane/Guide Levee Design Grade	EL 15.6 (or EL 12.1)
Conveyance (only) Guide Levee Design Grade @ Outfall	EL 11.6 *

Notes:

1. EL 16.4 is the authorized grade for the MRL at the project site.
2. EL 20.1 is the hurricane design grade for a 50-Year return period event, projected 50 years into the future, storm surge from the Mississippi River side.
3. EL 15.6 is the hurricane design grade for a 50-Year return period event, projected 50 years into the future, storm surge from the Basin.
4. EL 12.1 is the hurricane design grade for a 25-Year return period event, projected approximately 25 years into the future, storm surge from the Basin.
5. "Headworks structures tying into the MRL line of riverine flood protection are: Intake U-Frame walls, tie-in T-Walls to MRL embankment, Diversion Gate Structure and Steel Gates.
6. There is no structural superiority for hardened flood protection structures.

*EL 11.6 provided in 2014 Base Design for retainment of operational diversion flows plus freeboard. Hydraulic modeling is ongoing to verify water surface elevations in Conveyance Channel, which will determine final conveyance levee design grade.

7. SUMMARY OF ALTERNATIVES SCREENING

7.1 General

The BOD Phase was structured around an alternatives and evaluation screening process with two decision-point, alternatives-selection workshops during the BOD Phase. A third workshop will be held during the 30% Phase, during which the enlargement of the intake type selected during BOD Phase will be confirmed.

The two BOD Phase workshops were held with the DT and CPRA Project Management Team (PMT), and observed by National Fish Wildlife Foundation (NFWF), first to identify and then to collaboratively evaluate and screen the alternatives for major project components. The workshops were structured in a similar manner. Prior to each workshop the DT, with input from the CPRA and PMT, developed a group of alternatives for evaluation and decision models in order to facilitate the scoring and ranking of the alternatives. The decision models included a group of scoring criteria with weighted factors assigned specifically to each component/feature. At each workshop, the DT and PMT collectively scored each feature alternative, ranked and selected the feature alternatives. Prior to each workshop held during BOD Phase, decision matrices were prepared collaboratively by the DT and CPRA. Each matrix included evaluation categories with corresponding rating scales and was assigned importance/weighting factors. At the workshops, the alternatives were evaluated and scored, and the matrices populated. Sensitivity analyses were performed by varying rankings and importance factors to evaluate bias. Selections were made at the workshops with one exception: selection of the River Intake structure type and invert elevation alternative. It was decided at the second workshop that the decision matrix and rankings should be considered preliminary pending results of further numerical H&H modeling. Based on the modeling completed after the second workshop, the matrix was finalized and scored. The final version of the matrix is presented in this summary. Meeting minutes of the two workshop document in greater detail than Section 7 the evaluation and scoring process, and are included in **Appendix R**. It was decided at the first workshop that for certain project features that 15%-level engineering would proceed but changes to these design concepts compared to the 2014 Designs would not be done through a decision matrix scoring process. This is described in **Section 7.5**.

The structure to the alternatives screening process is shown graphically in **Figure 1-1**. The alternatives screening followed this sequence:

- At the first workshop, potential alternatives to be conceptually engineered and evaluated were identified, ranked and selected using decision matrices with qualitative scoring criteria.
- Numerical H&H modeling and conceptual-level civil, geotechnical, and structural engineering and design were then performed for the alternatives selected at the first workshop for investigation. Numerical modeling was performed according to a CPRA-approved numerical modeling work plan (see **Appendix H.7**). Conceptual engineering performed during the BOD phase was initiated using existing data, studies, and reports. Existing data has been deemed sufficient for executing the conceptual designs and performing alternatives analyses and selections. Additional data collection was begun during the BOD Phase, including additional geotechnical borings and lab data and additional topographic and geotechnical surveys, but this new data will not be used until Phase 2 design. The numerical modeling is summarized in **Section 8** and presented in greater detail in **Appendix H**. The geotechnical engineering is summarized in **Section 9** and supporting analyses are presented in **Appendix G**. Civil engineering

used in the screening of the alternatives is summarized in **Sections 11 and 12**. Structural engineering of the major diversion components is presented in Section 10 with supporting computations in **Appendix J**. H&H and civil and structural engineering for the Interior Drainage alternatives comparisons are presented in **Section 8.11** and supporting computations presented in **Appendix I**.

- Class 5 life cycle cost comparisons were developed using the modeling and conceptual designs.
- A second workshop was held at which the alternatives were evaluated and scored. The engineering and designs of the discarded alternatives were stopped. E&D of the selected alternatives continued to further refine costs.
- BOD Phase numerical H&H modeling of the intake alternatives was completed.
- The intake alternatives selection matrix was revised and scored.

7.2 Workshop No. 1

Alternatives Workshop No. 1 occurred at the beginning of the BOD Phase, on December 7, 2017. The goal of this workshop was to identify major component/design feature alternatives to evaluate through numerical hydraulic modeling and conceptual E&D during the BOD Phase. The features selected for evaluation were: the Intake, Conveyance Channel, Back Gate, and two non-conveyance features: drainage of interior polder upriver of diversion, railroad bridge alignment. It was decided that establishing the railroad bridge alignment would be done outside of the screening process, by proposing alternatives and reaching agreement with the NOGC Railway through a series of coordination meetings.

7.2.1 Alternatives Evaluated

As shown in Table 7-1, the workshop alternatives were organized according to project components or design features. The DT clearly identified alternatives, including advantages and disadvantages for each where applicable, and presented schematics and figures to assist the workshop group in visualizing the differences among alternatives. A complete copy of the presentation is provided with the workshop meeting minutes included in **Appendix R**.

Table 7-1: Alternatives Presented at Workshop No. 1

COMPONENTS	WORKSHOP No. 1 ALTERNATIVES PRESENTED
Intake Structure Type	U-Frame with Interior Walls
	U-Frame without Interior Walls
	Open Channel
	Submerged Culvert
	Longer Intake with Gate Structure at Hwy 23
Open Channel Variations	Training Walls and Armoring
	Training Walls, Armoring and Turbulent River Side Structures
Sill Elevations	EL -40
	EL -20
	EL -50
	EL -50 with adjustable weir
	Varies EL -50 to EL -35 across intake length
Intake Angle with River	Straight Alignment with Conveyance Channel
	15 Degree Angle
	30 Degree Angle
Intake Construction	Conventional, Soil Founded
	Conventional, Pile Founded
	In-wet, in Conveyance Channel
	In-wet, Off-Site River Location
Diversion Gate Types	Tainter Gate
	Vertical Lift Gate
	Tainter Gate with Variable Weir
Diversion Gate Location	450 ft, P/S of MRL C/L
	250 ft, P/S of MRL C/L
	200 ft, F/S of MRL C/L
	800 ft, MRL C/L
	At Hwy 23
Channel Profile	Flat after transition to back structure
	Vary slope from transition to back structure
	Vary slope from transition through Outfall
Channel Linings	Riprap below water, ACB to top of levee
	ACB full width of channel section
	USACE ACM along channel bottom, up slope, and ACB to levee toe
	USACE ACM along channel bottom, up slope, and turf from channel bank to levee crown
	Geoweb, geocells or marine mattress

Table 7-1 Alternatives Presented at Workshop No. 1 (continued)

COMPONENTS	WORKSHOP No. 1 ALTERNATIVES PRESENTED
Transition Type	150 ft Concrete U-Frame
	Concrete Trapezoidal Flume
	Concrete Retaining Wall with Concrete Lined Channel
Back Structure Replacement with Hurricane Levees	Back Structure at Existing Back Levee, 50-Year Stage
	Back Structure at Realigned NOV Levee, 100-Year Stage
	Guide Levees as Hurricane Levees
	Back Structure at Existing Back Levee for Sediment Dispersal
NOV Levee	USACE to keep NOV Levee at current location
	USACE Realignment of NOV Levee, existing NOV Levee maintained
Interior Drainage	Pump Station to drain north polder
	Siphon in lieu of pump station; existing NOV levee alignment
	Siphon in lieu of pump station; NOV levee realigned
Railroad Bridge	RR alignment turns west, crossing MBSD at Hwy 23
	Maintain RR on MRL alignment with flood proof bridge crossing MBSD
Hwy 23 Bridge	Low chord clears levee crown plus 15 feet or maintenance road plus 16.5 feet
	Reduce low chord by including floodwall and underpass road crossing
	Reduce low chord by including floodwall and at-grade crossing
MRL Penetration and Interim Protection	DT provides concept design; CMAR performs detail design
	DT provides concept and final design with CMAR input
Pile Type Comparison	Large diameter pipe piles
	Prestressed concrete piles
	H-piles

7.2.2 Decision Models

Decision models were developed as a tool to evaluate some of the workshop alternatives, with the objective of advancing some alternatives to the BOD Phase while eliminating others. The DT proposed selection criteria to best capture the advantages and/or disadvantages of the alternatives in relation to achieving project goals. A scoring system was defined with numerical values between 1 and 4, with a score of 1 indicating the most favorable, and a score of 4 indicating the least favorable. Definitions for the selection criteria and a detailed explanation of the scoring system are included in the workshop meeting minutes in **Appendix R**. The workshop team also collaboratively assigned a weighted factor percentage to each selection criteria, as shown in Table 7-2. The same weighted factors were applied to all criteria/alternatives evaluated in this workshop.

Table 7-2: Selection Criteria for Workshop No. 1

Criteria	Weighted Factor
Design Complexity	10%
Adaptability	25%
Constructability	15%
Environmental Impact	10%
Operations & Maintenance	15%
Sediment Transport/Land Building Potential	25%

This decision model was applied to the screening process for the Intake Structure type, the Diversion Gate location, and the transition type alternatives. For those components, the workshop group collaboratively worked through the decision model, until a consensus was reached on assigning a score of 1-4 to the criteria for each alternative. Assigned scores measured how well each alternative achieved the project goals. The decision models for the Intake Structure type, Diversion Gate location, and transition type are shown in Figures 7-1, 7-2 and 7-3. By summing the products of the score and weighted factor percentages across an alternative, the result was a numerical score which served to rank the alternatives, with the lowest numerical score representing the highest ranked alternative. In some cases, a sensitivity analysis was performed with the decision model to determine whether adjusting the weighted factor percentage for a particular criterion affected the ranking outcome.

Results of the Intake Structure type decision model (Figure 7-1) showed the open channel with the highest composite ranking. Due to a low composite ranking and a decision model sensitivity analysis, the gated structure at Hwy 23 was eliminated for consideration as an Intake Structure alternative. The other alternatives were selected for further consideration in the BOD Phase.

		Criteria							
	Importance	1	2	3	4	5	6	7	Composite Ranking*
		Design Complexity	Adaptability	Constructability	Environment Impacts	Operability / Maintenace	Land Building Potential		
	Factor %	10.0%	25.0%	15.0%	10.0%	15.0%	25.0%		100.0%
Alternative 1a	U-Frame - Bays - Soil Founded (2014 Design Report)	3	3	3	2	3	2		2.65
Alternative 1b	U-Frame w/o Interior Walls	3	3	3	2	3	2		2.65
Alternative 1c	Open Channel	2	2	2	1	2	3		2.15
Alternative 1d	Tunnel	3	4	4	2	4	1		2.95
Alternative 1e	Gated Structure at Hwy 23 w/longer intake (4400 vs 1100)	3	3	4	2	4	2		2.95

Figure 7-1: Decision Model for Intake Structure Type

Results of the Diversion Gate location decision model (Figure 7-2) showed the Hwy 23 gated structure and 200 feet flood side of the MRL gate alternatives with the lowest composite rankings. These two alternatives were eliminated from further study. The other alternatives were advanced for evaluation in the BOD Phase.

	Importance	Criteria 1	2	3	4	5	6	7	Composite Ranking*
		Design Complexity	Adaptability	Constructability	Environment Impacts	Operability / Maintenace	Sediment Transport/Land Building Potential		
	Factor %	10.0%	25.0%	15.0%	10.0%	15.0%	25.0%		100.0%
Alternative 7a	450ft to the P/S of MRL C/L – 2014 Report	2.5	2	3	2	2	2		2.2
Alternative 7b	250ft to the P/S of MRL C/L	3	2	2.5	2	2	2		2.2
Alternative 7c	200ft to the F/S of MRL C/L	3	2	3	2.5	3	2		2.5
Alternative 7d	750 from CL inland	2	2	2	2	2	2		2.0
Alternative 7e	gated structure at Hwy 23 (4300 from CL)	3	3	2.5	2.5	2.5	2		2.6

Figure 7-2: Decision Model for Diversion Gate Location

Results of the transition type decision model (Figure 7-3) showed the stepped sheet pile wall alternative having the lowest composite ranking. This alternative was eliminated from further study, and the other two alternatives were advanced for evaluation in the BOD Phase.

	Importance	Criteria 1	2	3	4	5	6	7	Composite Ranking*
		Design Complexity	Adaptability	Constructability	Environment Impacts	Operability / Maintenace	Land Building Potential		
	Factor %	10.0%	25.0%	15.0%	10.0%	15.0%	25.0%		100.0%
Alternative 11a	150ft – Stepped Sheet PileWall	3	3	2.5	1	3	2		2.5
Alternative 11b	Concrete Retaining Wall w/ all concrete floor	2	2	2	1	2	2		1.9
Alternative 11c	Concrete Retaining Wall w/ Riprap lined Channel	2	2	1.5	1	2	2		1.8

Figure 7-3: Decision Model for Transition Type

Some of the component alternatives and design features discussed during Workshop No. 1 were not conducive to being evaluated through the decision model process. The alternatives for the open channel variations, sill elevations, intake angles, intake construction methods, and the Diversion Gate types were discussed among the workshop group members, and through an open dialogue the advantages and disadvantages were openly discussed, although not numerically ranked. A summary of the all selected alternatives is shown in Table 7-3.

For a complete discussion of the decision models and selection process, refer to the workshop meeting minutes in **Appendix R**.

7.2.3 Alternatives Workshop No. 1 Selections

As evidenced through the alternative evaluation process detailed in the meeting minutes, any alternatives that were determined to contain fatal flaws were removed from consideration. If an alternative was determined through the group discussion to be neither economically nor physically feasible, or could not achieve a project goal, it was classified as having a fatal flaw.

Based on the results of the decision models or discussions during the workshop, remaining alternatives were selected to advance for further consideration in the BOD Phase. A list of these selected alternatives is provided in Table 7-3.

Table 7-3: Selected Alternatives - Workshop No. 1

COMPONENTS	WORKSHOP No. 1 SELECTED ALTERNATIVES
Intake Structure Type	U-Frame with Interior Walls
	U-Frame without Interior Walls
	Open Channel
	Submerged Culvert
Open Channel Variations	Training Walls and Armoring
	Training Walls, Armoring and Turbulent River Side Structures
Sill Elevations	EL -40
	EL -20
	EL -50
*Intake Angle with River	Straight Alignment with Conveyance Channel
Intake Construction	Conventional, Pile Founded
	In-wet, in Conveyance Channel
Diversion Gate Types	Tainter Gate
	Vertical Lift Gate
Diversion Gate Location	450 ft, P/S of MRL C/L
	250 ft, P/S of MRL C/L
	200 ft, F/S of MRL C/L
	800 ft, MRL C/L
Channel Profile	Flat after transition to back structure
	Vary slope from transition to back structure
	Vary slope from transition through Outfall
Channel Linings	Riprap below water, ACB to top of levee
	ACB full width of channel section
	USACE ACM along channel bottom, up slope, and ACB to levee toe
	Geoweb, geocells or marine mattress
Transition Type	Concrete Trapezoidal Flume
	Concrete Retaining Wall with Concrete Lined Channel
Back Structure Replacement with Hurricane Levees	Back Structure at Existing Back Levee, 50-Year Stage
	Back Structure at Realigned NOV Levee, 100-Year Stage
	Guide Levees as Hurricane Levees
	Back Structure at Existing Back Levee for Sediment Dispersal
NOV Levee	USACE to keep NOV Levee at current location
	USACE Realignment of NOV Levee, existing NOV Levee maintained

Table 7-3: Selected Alternatives - Workshop No. 1 (Continued)

COMPONENTS	WORKSHOP No. 1 SELECTED ALTERNATIVES
Interior Drainage	Pump Station to drain north polder
	Siphon in lieu of pump station; existing NOV Levee alignment
	Siphon in lieu of pump station; NOV Levee realigned
Railroad Bridge	RR alignment turns west, crossing MBSD at Hwy 23
	Maintain RR on MRL alignment with flood proof bridge crossing MBSD
Hwy 23 Bridge	Low chord clears levee crown plus 15 feet or maintenance road plus 16.5 feet
	Reduce low chord by including floodwall and underpass road crossing
	Reduce low chord by including floodwall and at-grade crossing
MRL Penetration and Interim Protection	DT provides concept design; CMAR performs detail design
	DT provides concept and final design with CMAR input
Pile Type Comparison	Large diameter pipe piles
	Prestressed concrete piles
	H-piles

*Note: The angled intake alternatives were considered as part of the optimization process for the selected intake alternative. Because the selected alternative is the open channel, for which numerical modeling revealed that the size and geometry of the sediment capture zone is the relevant characteristic, the variation in intake geometry entailed modifying the angles of the training walls rather than rotating the overall orientation of the intake. This is further described in Section 8.5.7.

7.3 Workshop No. 2

Alternatives Workshop No. 2 was held June 7, 2018, with DT and PMT members in attendance. The goal of this workshop was to select design alternatives for major project components using numerical modeling results, civil layouts and conceptual geotechnical and structural designs. Design features included Interior Drainage Alternatives, Conveyance Channel Profiles, Back Gate Structure/Hurricane Grade Levees, and Intake Structure Type/Invert Elevations.

7.3.1 Alternatives Evaluated

As shown in Table 7-4, Workshop No. 2 alternatives were organized according to project components. Based on results from Workshop No. 1 and subsequent engineering analysis, the DT clearly identified alternatives including advantages and disadvantages for each where applicable, and presented schematics and figures to assist the workshop group in visualizing the differences among alternatives. The presentation is provided with the workshop meeting minutes included in **Appendix R**.

Table 7-4: Alternatives Presented at Workshop No. 2

COMPONENTS	WORKSHOP No. 2 ALTERNATIVES PRESENTED
Interior Drainage	Pump Station
	Siphon
Conveyance Channel Profile	Channel invert at constant EL -25
	Constant sloping invert from EL -25 at transition to EL -15 at NOV Levee and EL-9.5 at Outfall
	Constant invert at EL -25 to NOV Levee, then slope to EL -12 at Outfall
	Channel invert at constant EL -20
	Constant sloping invert from EL -20 at transition to EL -12 at NOV Levee and EL -7 at Outfall
	Constant invert at EL -20 to NOV Levee, then slope to EL -10 at Outfall
Back Gate Structure vs. Hurricane Grade Levees	Back Gate Structure with Guide Levees
	Hurricane Grade Guide Levees with No Back Structure
Intake Structures	U-Frame with Interior Walls @ EL -40
	U-Frame without Interior Walls @ EL -40
	Open Channel @ EL -40
	Submerged Culvert @ EL -40
	Open Channel @ EL -50
	Submerged Culvert @ EL -50
	Open Channel @ EL -20
	U-Frame without Interior Walls @ EL -20

7.3.2 Decision Models

Decision models were developed as a tool to evaluate, rank, and select the workshop alternatives, with the objective of advancing the selected alternatives to the 15% BOD level. The DT proposed selection criteria and a scoring system somewhat similar to those used in Workshop No. 1. However, the evaluation criteria and weighted factor percentages for Workshop No. 2 were specifically tailored to each project component. Quantitative results of numerical H&H modeling; conceptual-level civil, geotechnical engineering; and Class 5 cost comparisons were used to rank the alternatives.

The decision models were presented as interactive spreadsheets, which enabled the group to perform an interactive screening process and perform sensitivity analyses, if appropriate, by adjusting the weighted factors. Definitions for the Workshop No. 2 selection criteria, detailed explanations of the scoring systems, and explanations of sensitivity analyses performed are included in the workshop meeting minutes in **Appendix R**.

Figures 7-4 through 7-6 present the final versions of the decision models, including weighted criteria and ultimate ranking of alternatives. For the purposes of the Workshop No. 2 selection process, the hydraulic models used to develop decision model criteria for the Intake Structure were based on existing conditions for a river flow of 1,000,000 cfs and channel flow of 75,000 cfs.

Interior Drainage	Importance	Criteria				Composite Ranking
		1	2	3	4	
		Construction Cost	Operability/ Maintenance Cost	Resilience	Additional Real Estate	
	Factor %	40%	35%	20%	5%	100%
Alternative 1	Pump Station	5	5	2	3	4.3
Alternative 2	Siphon	1	1	3	1	1.4

Figure 7-4: Workshop No. 2 Decision Model for Interior Drainage

Conveyance Channel Profile	Importance	Criteria		Composite Ranking
		1	2	
		Energy Loss	Construction Cost	
	Factor %	60%	40%	100%
Alternative 1	Channel Invert at Constant EL -25	1	1	1.0
Alternative 2	Constant sloping invert from EL -25.0 at transition to EL -15.0 at NOV levee and EL -9.5 at outfall	5	2	3.8
Alternative 3	Constant invert at EL -25 to NOV levee, then slope to EL -12 at the outfall	2	2	2.0
Alternative 4	Channel Invert at Constant EL -20	1	1	1.0
Alternative 5	Constant sloping invert from EL -20 at transition to EL -12 at NOV levee and EL -7 at outfall	4	5	4.4
Alternative 6	Constant invert at EL -20 to NOV levee, then slope to EL -10 at the outfall	3	2	2.6

Figure 7-5: Workshop No. 2 Decision Model for Conveyance Channel Profile

Back Gate vs. Hurricane Levees	Importance		Criteria				Composite Ranking
		1	2	3	4	5	
		Energy Loss	Hydraulic - Benefits of Sediment Distribution	Impacts to Siltation Levels	Construction Cost	Routine Operability/ Maintenance Cost	
	Factor %	25%	25%	10%	30%	10%	100%
Alternative 1	Back Gate w/ Guide Levees	5	1	1	5	5	3.6
Alternative 2	No Back Gate, Hurricane Levees	4	1	5	1	1	2.2

Figure 7-6: Workshop No. 2 Decision Model for Back Gate Structure vs. Hurricane Grade Levees

7.3.3 Alternatives Workshop No. 2 Selections

Based on the results of the decision models and/or discussions during the workshop and after the initial hydraulic modeling was completed, the siphon, channel profile EL -25, and Hurricane Grade Levees with no back structure were selected as the preferred alternatives for each feature respectively. A preliminary decision model was performed on the intake structure types at the workshop. This exercise resulted in the elimination of the submerged culvert alternatives, due to the calculated head loss values. Discussion of this screening process is included in **Appendix R**.

The selected alternatives for Workshop No. 2 are summarized in Table 7-5.

Table 7-5: Selected Alternatives - Workshop No. 2

COMPONENTS	WORKSHOP No. 2 ALTERNATIVES SELECTED
Interior Drainage	Siphon
Conveyance Channel Profile	Channel invert at constant EL -25
Back Gate Structure vs. Hurricane Grade Levees	Hurricane Grade Levees with No Back Structure
Intake Structure	Submerged Culvert Alternatives Eliminated

7.4 Additional Screening for Intake Alternatives

After Workshop No. 2, at the July Monthly Technical Meeting, the DT performed additional screening on the intake alternatives using Delft3D with three operational river stages: 1,000,000; 800,000; and 600,000 cfs; and additional FLOW-3D modeling for river stage at 600,000 cfs, but capturing hydrodynamic effects only to calibrate energy losses in the Delft3d models. The new model results were then used in a revised intake decision matrix (see **Figure 7-7**). One weighted SWR was calculated for each alternative, and this SWR considered the duration of each operational stage. The weighted average SWRs were computed by estimating time durations when the respective river stages were historically recorded at the Belle Chasse Gage. Because the submerged culvert alternatives were eliminated from consideration at Workshop No. 2, they were not evaluated in this additional screening process.

The decision matrix was modified to include a column for the Workshop No. 2 FLOW-3D SWR and a column for the weighted SWR from the additional Delft3D modeling. The DT-guided decision model demonstrated the Open Channel at EL -20 as the best-performing alternative, with the Open Channel at EL -40 as the second best. At the end of July, CPRA informed the DT that the PMT performed a separate additional screening exercise internally, in which the scoring was modified to lend more weight to the adaptability scoring factor. As shown in Figure 7-8, this modified decision model demonstrated the Open Channel at EL -40 as having the highest composite ranking.

The scoring scales and weighted importance of the evaluation criteria in the matrix were also revised after Workshop No. 2. Extremes for the SWR scoring range were established by using the lowest and highest SWRs computed for historic TWIG modeling of intake alternatives and then divided into five equal banded ranges. The energy loss scoring range was selected as a range of 0.7 feet to 4.7 feet, in five equally banded ranges of 0.8 feet each. The construction cost scoring range for the headworks alternatives was established as $202 million to $867 million, comprising five equally banded ranges of

$133 million each. The "Risk (Design/Construction)" Category was used in lieu of the Operational Risk Category in the interim matrix because it was concluded that the "Operability/Maintenance" Category includes operational risk. The Adaptability Category remained the same as was established in the interim scoring matrix. Risk, Operability/Maintenance, and Adaptability are qualitative categories.

The weighting factors of the evaluation criteria were modified from the original matrix. The performance categories collectively were kept at a weighting of 50% of the overall ranking, but also included the SWR category for Delft-3d results. Adaptability was increased from 10% to 25% because scoring participants concluded that the uncertainty of future conditions and events demand a system whose design can be modified if required. For example, the construction or modification of marine facilities directly upriver of the diversion could adversely affect diversion performance. Other variables include regional subsidence and sea level rise. Potential countermeasures could include modifying intake geometry. A hardened intake, such as a U-frame, would require demolition and reconstruction, whereas the Open Channel could be more readily modified more readily by demolishing and reconstructing the training walls and additional dredging and stone armoring. The Risk Category rating was kept at 10%, as was the Operability/Maintenance Category. The Construction Cost category's weighting was reduced from 20% to 5%, reflecting that the most important quality is that the system meet performance specifications, both during the initial operational period and in the future as river and conveyance conditions evolve.

After revising the scoring matrix and ranking the alternatives, the Open Channel at EL -40 was selected as the preferred intake configuration.

Intake Structure Selection (Modified)		Criteria - Maintaining 75,000cfs at Current Boundary Conditions							Composite Ranking
	Importance	1	2	2	3	4	5	6	
		Hydraulics (Head Loss)	SWR River (Flow3d)	SWR River (Delft)	Adaptability to Change	Risk (Design/ Construction)	Construction Cost	Operability/ Maintenance	
	Factor %	25%	5%	20%	25%	10%	5%	10%	100%
Alternative 1	U-Frame with Walls @ EL -40	5	3	3	3	4	3	1	3.25
Alternative 2	U-Frame without Walls @ EL -40	4	2	3	3	4	3	1	3
Alternative 3	Open Channel @ EL -40	3	1	3	1	3	2	1	2.15
Alternative 4	Submerged Culvert @ EL -40								n/a
Alternative 5	Open Channel @ EL -50	3	2	4	1	3	3	2	2.45
Alternative 6	Submerged Culvert @ EL -50								n/a
Alternative 7	Open Channel @ EL -20	2	2	2	4	2	2	1	2.4
Alternative 8	U-Frame without Walls @ EL -20	3	2	2	5	3	2	1	2.9

Figure 7-7: Final Decision Model for Intake Selection

7.5 Alternative Selections Performed Apart from the Workshop Evaluation Process

Certain project features that 15%-level engineering investigated compared to the 2014 Designs were not done through a decision matrix scoring process. The DT has performed the evaluations and the recommendations are listed below:

Table 7-6: Alternatives Recommendations Developed Outside of Alternatives Workshop Screening Process

Project Component/Feature	Selection Recommendation
Headworks-River Armoring	Riprap armoring was selected as the armoring type. See **Section 10.4.6**.
Headworks-Diversion Gate Type	The tainter gate type is recommended. See **Appendix O**.
Headworks-riverine flood protection structures	Riverine protection constructed to EL 20.1 is recommended. See **Section 10.6**.
Headworks-on-site emergency crane	A dedicated, on-site crane is not required. See **Appendix P**.
Conveyance Channel-Revetment System	System will be either riprap or a modular revetment system. This will be selected during 30% Phase. See **Section 11.5** and **Appendix N**.
Conveyance Channel-Guide Levees, top of flood protection	Select EL 15.6 as top of levee. See **Section 11**.
Conveyance Channel-Guide Levees, ground improvements	Construct guide levees using wick drain system and staged construction. **See Section 9.16.4**.
Outfall Transition Feature-Ramp Geometry	A 1,500-foot ramp length is recommended. See **Section 8.8**.
NOGCC Railroad Bridge	Construct the bridge along the existing alignment of the rail line. The low chord of the bridge across the diversion intake will be at or higher than EL 16.4. See **Section 14**.
LA-23 Bridge	Construct the bridge along the existing highway's alignment. See **Section 13**.
Beneficial Use of Unsuitable Materials for Levee Construction	Construct wetlands in areas designated for disposal of materials. See **Section 24**.
Secondary Project Features	Incorporate into project those features identified in **Section 20**.

8. HYDROLOGY, COASTAL ENGINEERING, AND HYDRAULICS

8.1 General

The hydrologic and hydraulic analyses along with the numerical and physical modeling to support the E&D are described in this section. The major hydraulic features of the project and the key hydraulic processes are described in the beginning followed by summary of the various analyses. The details of the analyses are provided in **Appendix H**.

The methods were influenced by the fact that CPRA has a parallel, ongoing modeling effort with TWIG on MBSD. The DT's modeling is expected to be consistent with those efforts. The analysis performed leverages work already completed by CPRA/TWIG including the model setup (e.g. model geometry, boundary conditions, Relative Sea Level Rise and Subsidence) and findings.

The overall goal of the numerical modeling is to develop the design of an Intake Structure that can divert a maximum flow of 75,000 cfs flow when the Mississippi River reaches 1,000,0000 cfs at the Belle Chasse gage with as high Sediment-to-Water (SWR) ratio as achievable; approaching 1.0. The BODR presents modeling analysis under present Relative Sea Level Rise and Subsidence (RSLR) conditions. It also demonstrates diversion system performance under future (Year-50) RSLR conditions.

8.2 References and Publications

- Allison, M. and Meselhe, E. (2010). "The Use of Large Water and Sediment Diversions in the Lower Mississippi River (Louisiana) For Coastal Restoration." *Journal of Hydrology*, 387 (3-4),346-360.
- Allison, M. (2011). "Water and Sediment Surveys of the Mississippi River Channel Conducted at Myrtle Grove and Magnolia in Support of Numerical Modeling (October 2008 to May 2011)." *A report to the State of Louisiana Office of Coastal Protection and Restoration as subcontracted to C.H. Fenstermaker and Associates.*
- Allison, M., Vosburg, B., Ramires, M. and Meselhe, E. (2012). "Mississippi River Channel Response to the Bonnet Carre Spillway Opening in the 2011 Flood and Its Implications for the Design and Operation of River Diversions." *Journal of Hydrology*, 477, 104-118.
- Allison, M., Demas, C., Ebersole, B., Kleiss, B., Little, C., Meselhe, E., Powell, N., Pratt, T. and Vosburg, B. (2012). "A Water and Sediment Budget for the Lower Mississippi-Atchafalaya River in Flood Years 2008-2010: Implications for Sediment Discharge to the Oceans and Coastal Restoration in Louisiana." *Journal of Hydrology*, 432-433, 84-97.
- Allison, M., Vosburg, B., Ramires, M. and Meselhe, E. (2014). "Diversion of Mississippi River Water Downstream of New Orleans, Louisiana, USA to Maximize Sediment Capture and Ameliorate Coastal Land Loss." *Water Resources Management*, 28, 4113-4126.
- Bettelle. (2017). "Final Panel Comments on the Independent External Peer Review of the Louisiana Coastal Area Mississippi River Hydrodynamic Study Technical Report and Tool development: PHASE 3."
- Esposito, C., Liang, M., and Meselhe, E. (2017). "TO5 Outfall Management Mid-Barataria and Mid-Breton Diversions."

- Esposito, C., Liang, M., Sadid, K. and Meselhe, E. (2017). "Mid-Barataria Sediment Diversion Modeling Package."
- FTN Associates. (2013). "Numerical Modeling for LCA Medium Diversion at Myrtle Grove with Dedicated Dredging: Model Calibration and Validation." *Report prepared for CPRA.*
- Gaweesh, A. and Meselhe, E. (2016). "Evaluation of Sediment Diversion Design Attributes and Their Impact on the Capture Efficiency." *Journal of Hydraulic Engineering*, 142(5).
- HDR. (2014). "Mid-Barataria Sediment Diversion Alternative 1, Base Design Report 30% Basis of Design."
- HDR. (2014). "Mid-Barataria Sediment Diversion Hydraulic Report 30% Basis of Design and Value Engineering."
- Hydraulic Design Handbook, Larry W. Mays, McGraw-Hill Education, 1999
- Jung, H., Baustian, M. and Meselhe, E. (2016). "TO15/16: Outfall Areas in Barataria and Breton Sound Vegetation Model Improvement."
- Jung, H., Baustian, M. and Meselhe, E. (2016). "TO15/16: Outfall Areas in Barataria and Breton Sound Water Quality Model Improvement."
- Khadka, A., Sadid, K. and Meselhe, E. (2016). "TO 15/16 Analysis of Outfall Areas in the Barataria and Breton Sound Basins: Improvement of Sand Load Transport and Dredging Calibration in the Mississippi River."
- Liang, M., Meselhe, E., Messina, F. and Ortals, C. (2016). "TO5/TO41/TO44 Sediment Diversions: Optimization of the Operation Plans."
- Liang, M., Esposito, C. and Meselhe E. (2017). "Technical Memorandum Mid-Barataria Invert Elevation and Outfall Channel Configuration."
- Meselhe, E., Georgiou, I., Allison, M. and McCorquodale, J. (2012). "Numerical Modeling of Hydrodynamics and Sediment Transport in Lower Mississippi at a Proposed Delta Building Diversion." *Journal of Hydrology*, 472-473, 340-354.
- Meselhe, E.A., Baustian, M.M. and Allison, M.A. (2015). "Basin Wide Model Development for the Louisiana Coastal Area Mississippi River Hydrodynamic and Delta Management Study." The Water Institute of the Gulf, Baton Rouge, LA.
- Meselhe, E., Baustian, M. and Allison, M. (2015). "Basin Wide Model Development for the Louisiana Coastal Area Mississippi River Hydrodynamic and Delta management Study."
- Meselhe, E., Messina, F., Sadid, K. and Jung, H. (2016). "TO5/TO41/TO44 Internal Model Dynamic."
- Meselhe, E., Baustian, M. and Jung, H. (2016). "Technical Memorandum TO5/TO41/TO44: Vegetation Sensitivity Test Runs."
- Meselhe, E.A., Sadid, K.M., Xing, F., Jung, H., Baustian, M., Smiths, J.G.C., Van Maren, B.D.S., Sylvester, S.M.D. and Visser, J.M. (2016). "Louisiana Coastal Area (LCA), Louisiana Ecosystem Restoration Study Mississippi River Hydrodynamics and Delta Management Study (MRHDMS): Assessment and Analysis of Alternatives." *Final Report to the LA CPRA in support of TO # 36.1 and TO # 5.*
- Messina, F. and Meselhe, E. (2016). "TO 15/16 Improvements to the Hydrodynamics and Salinity in Barataria: Basin-wide Model Version 2."
- Meselhe, E., Sadid, K. and Allison, M. (2016). "Riverside Morphological Response to Pulsed Sediment Diversions." *Geomorphology*, 270, 184-202.
- Messina, F. and Meselhe, E. (2017). "Technical Memorandum TO46: Mid-Barataria Optimization Plane-Base Flow Option."
- Meselhe, E., Jung, H. and Esposito, C. (2017). "Technical Memorandum TO46: Existing Diversion Analysis and Head Differential Analysis."

- Meselhe, E., Sadid, K., Messina, F. and Jung, H. (2017). "Technical Memorandum TO48: Basin-Wide Delft Evaluation of Diversion Operations (Production Run 11-15) Appendix A."
- Meselhe, E., Sadid, K., Messina, F. and Jung, H. (2017) "Technical Memorandum TO48: Basin-Wide Delft Evaluation of Diversion Operations (Production Run 11-15)."
- Meselhe, E., Costanza, K., Jung, H., Ainsworth, C., Simpson, E., Chagaris, D., Addis, D., Rodrigue and M., Smits J. (2017). "Models Performance Assessment Metrics for the LCA Mississippi River Hydrodynamic and Delta Management Study (TO5)."
- Meselhe, E., Jung, H., Sadid, K., Messina, F. and Baustian, M. (2017). "Calibration and Validation of Delft3D Model Version 2 Comparison of V1 and V2 Model Performance."
- Meselhe, E., Sadid, F., Messina, F. and Jung, H. (2017). "Technical Memorandum TO46: Basin-Wide Delft Evaluation of Diversion Operations (Production Runs 11-15)."
- Nittrouer et al (2011) Punctuated sand transport in the lowermost Mississippi River. *J. Geophysical Research*, Vol. 116.
- Rego, J., Meselhe, E., Stronach, J. and Habib, E. (2010). "Numerical Modeling of Mississippi-Atchafalaya Rivers' Sediment Transport and Fate: Consideration for Diversion Scenarios." *Journal of Coastal Research*, 26, 212-229.
- Sadid, K., Xing, F., Messina, F. and Meselhe, E. (2016). "TO 15/16 Improvement of Model Grid and Bathymetry."
- Sadid, K., Meselhe, E., Allison, M. and Van Maren, B. (2016). "TO 15/16 Sediment Properties used in the Basin-wide Model."
- Wang, H., Steyer, G., Couvillion, B., Rybczyk, J., Beck, H., Sleavin, W., Meselhe, E., Allison, M., Boustany, R., Fishchenich, C. and Rivera-Monroy, V. (2014). "Forecasting Landscape Effects of Mississippi River Diversions on Elevation and Accretion in Louisiana Deltaic Wetlands under Future Environmental Uncertainty Scenarios." *Estuarine, Coastal and Shelf Science*, 138, 57-68.

8.3 Diversion System Components and Key Hydraulic Processes

The proposed MBSD is a sediment delivery system made up of several components or hydraulic structures/features. Each component supports one or more functions and together they help accomplish the project goals. The design of each component is driven by specific hydraulic processes that will be modeled. The system components, functions and hydraulic processes are shown in **Figure 8-1**. The general hydraulic characteristics of each component are described briefly in the following sub-sections.

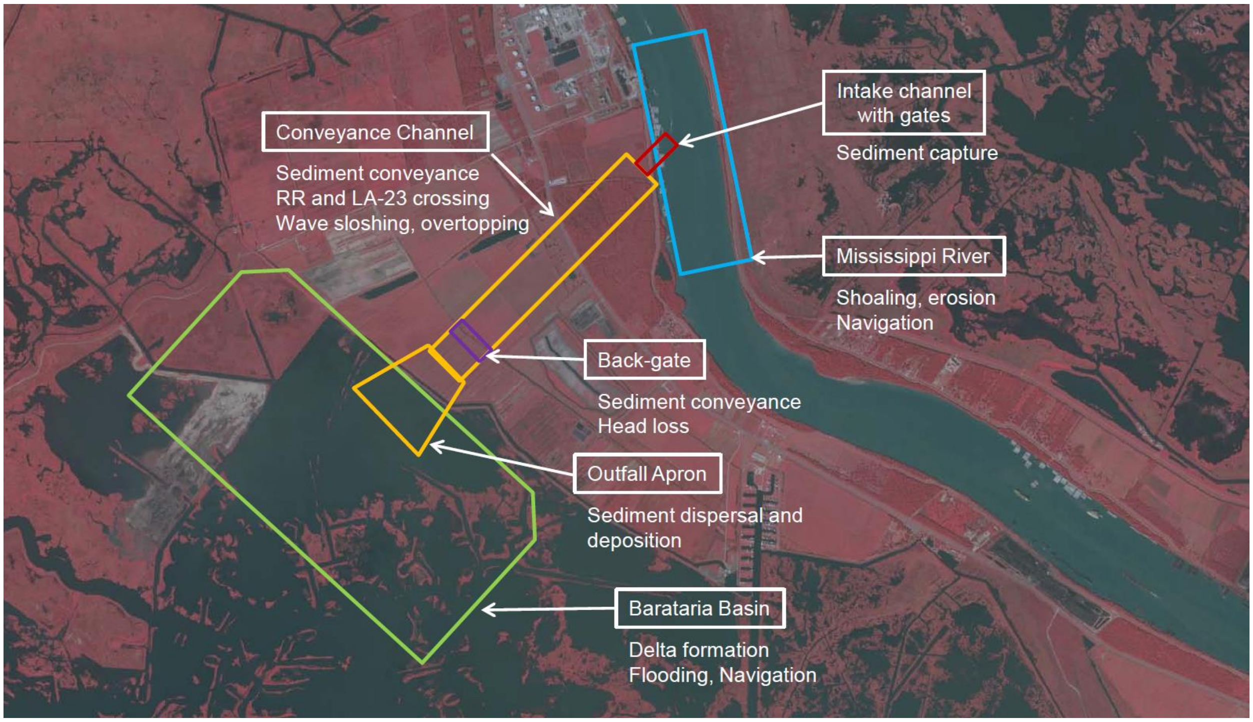


Figure 8-1: MBSD System Components, Functions and Key Hydraulic Processes

8.3.1 Mississippi River

The Mississippi River carries sediment-laden flows south to the Gulf of Mexico. At the project location, the depth of the River is approximately 120 feet and a sand bar exists at a depth of about 50 feet. The top width of the River is approximately 2,000 feet. At this location, the River carries a flow ranging from 425,000 cfs to 1,250,000 cfs during typical annual flood events. The transported sediment consists of clay, silt and sand particles. The dominant hydraulic processes in the vicinity of the diversion are longitudinal, transverse and vertical velocities due to the upstream river bend, suspended sediment transport through the water column and bed load transport along the sandbar present at the proposed diversion. The presence of the diversion may also induce erosion and deposition in the river. The changes induced in the velocity patterns in the river are also important to assess potential impacts on navigation.

8.3.2 Intake Channel with Diversion Gates, aka, Headworks (HW)

The intake channel and gated diversion structure is proposed on the west MRL bank to laterally divert a portion of the river flow and sediment. A riprapped or concrete river bank cut will lead to a rectangular bay(s) where gates will be situated. The bays will open in to the conveyance channel via a transition channel segment. The existing railroad will cross the rectangular bays.

The key hydraulic processes modeled in the river are the 3D velocity distribution, transport of sediment, potential erosion and deposition, shear stress variation and significance to the navigation. In the intake the important processes are the turbulence losses, sediment suspension, vorticity and any deposition.

The evaluated intake channels were either open cut channels or a concrete U-Frame type structure with inverts ranging from -50 to -20 feet NAVD88 (Note- all intake invert elevations referenced are in NAVD88 datum). A submerged culvert was also investigated. The gated diversion opening is approximately 150 feet wide. The gated structure is located on the protected side of the MRL. The structure is designed to capture as much sediment as possible at the design flow of 75,000 cfs and to obtain as high a SWR as achievable. Numerical models show that during operation, a complex three-dimensional velocity and turbulence field associated with sediment transport is generated at the structure entrance. The depth-averaged longitudinal velocities through the gate were around 15 ft/s. A transition section, most likely trapezoidal in section, is required between the back side of the Gated Structure and the Conveyance Channel which has a bottom width of about 300 feet.

8.3.3 Conveyance Channel

The discharge and sediment from the gated structure enters the Conveyance Channel which transports it into the Barataria Basin. This channel has a trapezoidal cross-section with berms, a bottom width of about 300 feet and side slopes of about 4:1. The length of the channel from the Diversion Gates to the Outfall at the basin-side is approximately 2 miles. The channel invert is EL -25. The maximum design flow capacity is 75,000 cfs. The Hwy 23 Bridge crosses the proposed channel. A gated back structure was evaluated toward the basin-side where the channel cuts through the non-federal NOV Levee System.

The key hydraulic processes being modeled are the transport of sediment, potential deposition of sediment along the channel and through the railroad and highway crossing, transport through the Back Gate Structure, and uphill through the Outfall Transition Feature to the basin-side, which has a prevailing mud bottom grade elevation at about EL -4. The energy losses at the entrance, railroad, highway crossings and at the Back Gate are an important factor in maintaining 75,000 cfs design capacity.

It is anticipated that a base flow or pulsed low flow will be required during river low flow periods (when the diversion is not operating for sediment delivery). This base or pulsed low flow is likely necessary to maintain water quality standards in the Conveyance Channel and the receiving basin. There is a possibility that sediment deposition will occur in the Conveyance Channel during these low flow operations and the potential for and extent of the deposition will need to be evaluated after the BOD Phase.

8.3.4 Back Gate Structure

A gated structure with multiple bays was evaluated along the NOV Levee where the Conveyance Channel cuts through to enter the basin. The key hydraulic processes at this structure are the complex velocity field upstream and downstream of the structure affecting discharge and sediment-carrying capacity of the Conveyance Channel. See **Appendix H.4** for the Back Gate Memo which describes analyses performed on the proposed Back Gate Structure.

The back structure was eliminated from the conveyance system at Workshop No. 2 conducted on 7 June 2018 and will no longer be included in conveyance modeling.

8.3.5 Outfall Transition Feature

The Outfall Transition Feature (or Outfall Area or Outfall Ramp) is a gradually flaring portion of the Conveyance Channel as it transitions from a deeper, regular trapezoidal cross-section to a shallower, wider basin Outfall. The key hydraulic processes in this region are the decelerating velocity field and uphill sediment transport. The purpose of this feature is to eliminate the sudden diversion system invert change from the conveyance channel to the basin so that the design flow can be achieved. The design is primarily useful in the initial years of full operation during which the transition will evolve.

8.3.6 Barataria Basin

The discharge and sediment are released directly into the middle portion of the Barataria Basin. The basin is about 1,600 square miles with depths ranging from 4 to 10 feet. The important hydraulic considerations in the basin are sediment dispersal and, deposition and erosion in the vicinity of the Outfall and the surrounding areas. Water levels near the communities in the basins and velocities and sediment deposition in the navigation waterways are also important considerations.

The Outfall and conveyance flow and sediment conditions shall be determined by the DT and provided to CPRA/TWIG who in turn shall model the changing basin conditions. A potential iterative process is currently being discussed between the DT and CPRA/TWIG to address conveyance aspects with respect to the future basin tailwater conditions. Tailwater conditions in the basin include the revised SLR provided to the DT in June 2018.

8.4 Overall Modeling Approach

As described in the previous section, the nature of the hydraulics varies from the MR to the basin. The three-dimensional nature of the flow is important at the intake side while two-dimensional treatment of flow is sufficient at the basin side. Analyzing this large system entirely with a three-dimensional model would have made meeting the project schedule impossible, would have been cost-prohibitive and was determined to be unnecessary. Instead, each of the system components was modeled separately using appropriate modeling programs, while maintaining consistency in the boundary conditions. In Phase 2, a larger model will be assembled combining the individually finalized diversion components so that the performance of the entire diversion system can be evaluated. The analysis of each of the diversion components is summarized below. The details of the modeling are found in **Appendix H**.

There are several related modeling activities that were not completed in Phase 1, as they do not directly affect the sizing of diversion system. The activities will be completed in in Phase 2. The activities are:

- River deposition and/or erosion
- Support for river navigation analysis
- Analysis of water quality in the Conveyance Channel during non-operation
- Diversion intake induced scour and point bar stability
- Evaluation of basin-side impacts on flooding
- Evaluation of secondary project features such as diversion intake marine protection features

8.5 Diversion Intake Numerical Modeling

The DT evaluated eight Intake Structure configurations in terms of the total energy head loss through the intakes and SWR. The energy loss was estimated using FLOW-3D model while the SWR was calculated using both the FLOW-3D and Delft3D models. All eight structures were simulated under Low, Medium and High Flow conditions in the Mississippi River. The energy loss and the SWR values were used as one of the parameters in the decision matrix that was used to make Intake Structure selection. The summary of model setup, inputs, and results is provided in the following sub-sections. The detailed modeling is described in **Appendix H**.

8.5.1 Intake Structure Configurations

The primary considerations for selection of the Intake Structure were the type of structure, such as an “open” configuration or a “submerged” configuration and an invert elevation. For “open” configurations, the possibilities were an Open Cut channel or a concrete U-Frame with or without interior walls. The "submerged" configuration consisted of a Submerged Culvert type Intake Structure. For intake inverts, a range from of EL -50 to EL -20 was considered. To limit the number of alternatives analyzed, eight representative Intake Structure alternatives were selected from which a final structure configuration and elevation would be selected. The eight alternatives are shown in Figure 8-2. The EL -40 invert was selected as a base case design as it was considered and analyzed in the 2014 Basis of Design Report (HDR, 2014). All four structure types were considered at this invert. The deeper invert, EL -50, was considered for the Open Channel and the Submerged Culvert structure types as these were anticipated to have access to deeper sediment. The shallower invert at EL -20 was considered for the U-Frame and the Open Channel structure types.

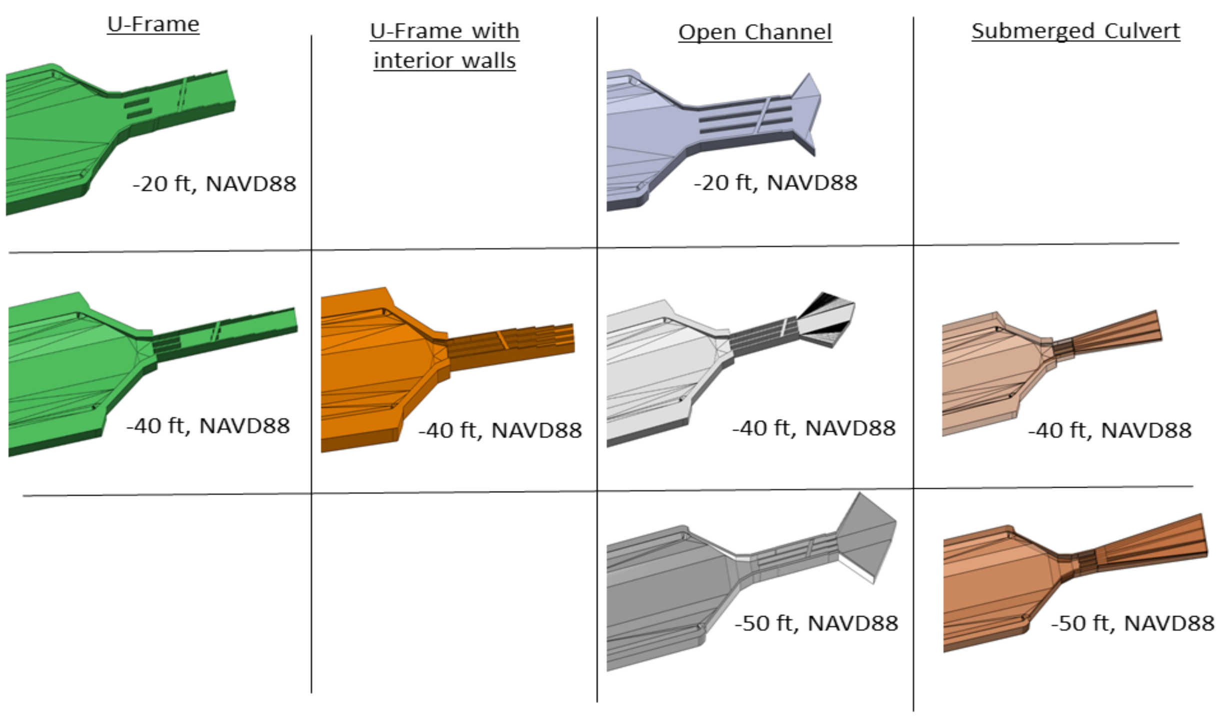


Figure 8-2: Schematic Representation of Eight Intake Structures Evaluated

8.5.2 Modeling Tools

Two modeling software programs were used in the analysis, FLOW-3D and Delft3D, due to their unique capabilities in modeling the relevant hydraulic processes. Being a non-hydrostatic model, the FLOW-3D provides a detailed simulation of three-dimensional velocity in the nearfield region of the structure and is most accurate to calculate energy losses and sediment particle capture efficiency through particle tracking. However, it does not have tested features for natural sediment transport process. FLOW-3D is also computationally intensive and suited to simulate relative smaller study area. On the other hand, Delft3D simulates velocities in the vertical dimension with less accuracy but does offer ability to simulate natural sediment process of suspended and bedload transport. In simulating detailed nearfield flow through Intake Structure, the energy loss for Delft3D was determined by calibrating it with energy loss obtained from a FLOW-3D model. Comparatively, Delft3D is less computationally intensive and therefore allows for simulation of larger study areas.

8.5.3 Model Geometry and Specification of Boundary Inputs

The eight structures were first modeled using FLOW-3D model. An example of FLOW-3D model domain is shown in Figure 8-3. The model focusses on simulating three-dimensional nearfield hydraulics of flow diversion. It extends from RM 58.1 to RM 62.7 on the MR Above Head of Passes (AHP) and includes a portion of the Conveyance Channel approximately 1,600 feet long.

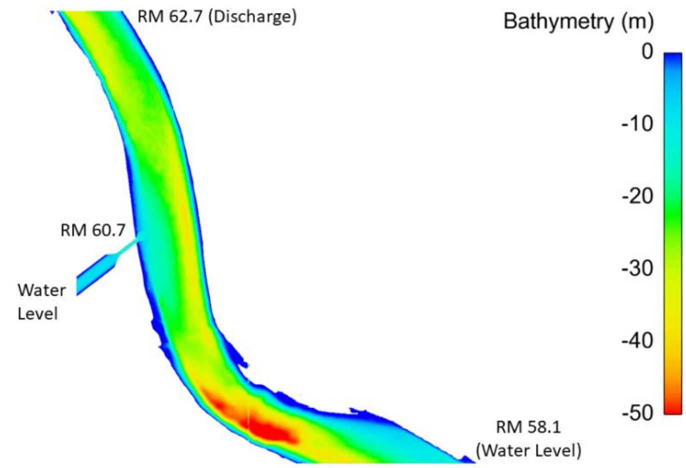


Figure 8-3: FLOW-3D Model Extent, Bathymetry (m, NAVD88) and Boundary Conditions. Horizontal Datum is UTM 15N.

An example of the Delft3D model domain is shown in Figure 8-4. The model is used for simulating sediment transport along the relevant segment of the MR and through the diversion. It extends from RM 56 to RM 66 on the MR and includes the same portion of the Conveyance Channel as the FLOW-3D. The model bathymetry was based on the USACE 2013 multibeam data and the USACE 2017 revetment survey data.

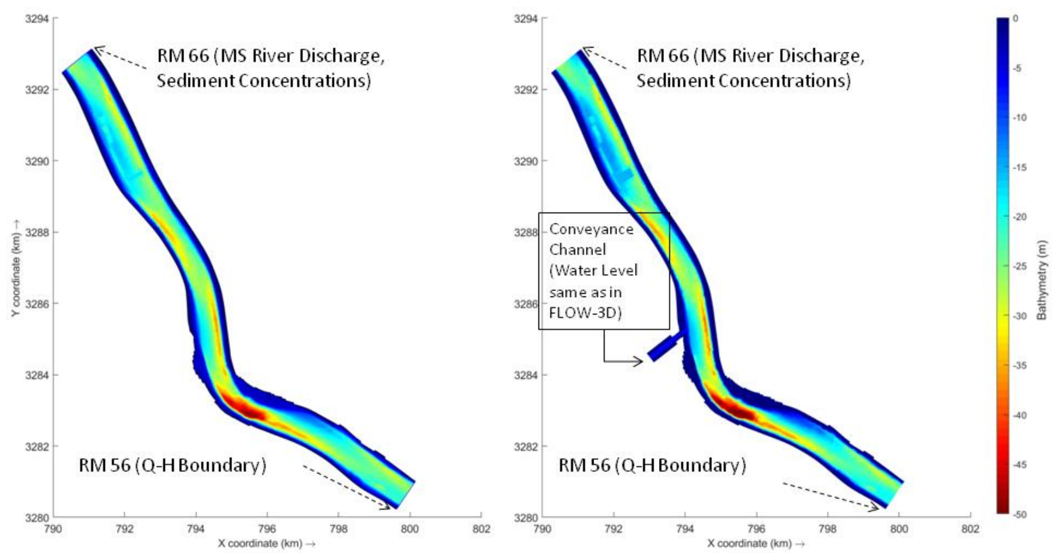


Figure 8-4: Delft3D Model Extent, Bathymetry (m, NAVD88) and Boundary Conditions. Horizontal datum is UTM 15N. Left panel shows the without-diversion case and the right panel shows with-diversion case.

8.5.4 Model Calibration and Validation

The models were calibrated using the discharge, water level and sediment data from April-2009 field collection performed by Allison (Allison, 2011). The MR flow was approximately 740,000 cfs. The models were then validated using March-2011 field data collected by Allison (Allison, 2011). The MR flow was approximately 966,000 cfs.

8.5.5 Discharge, Water Level and Sediment Boundary Conditions for Alternatives Evaluation

To satisfy the design criterion of delivering 75,000 cfs diversion flow at MR flow of 1,000,000 cfs, these conditions were modeled and termed as the High Flow (HF) condition. To account for the variable flows during a typical annual flood, Low Flow (LF) conditions were simulated when the MR was at 600,000 cfs. A Medium Flow (MF) condition was simulated at the midpoint of this range at 800,000 cfs. The flow was specified as an input to the models at the upstream end of the model geometry. The choice of the MR flows is discussed further in **Appendix H.1**.

At the downstream end of the MR and the basin-side of the Intake Structure, water surface elevations were specified. The MR water surface elevation were obtained and specified from a larger, system-wide model, which was previously completed by TWIG. At the basin-side of the Intake Structure, the water surface elevations were set such that the model drew 75,000 cfs through the Intake Structure.

In FLOW-3D, the sediment transport was simulated through particle tracking method. The particles were released upstream of the diversion and were converted to sediment concentrations as explained in **Appendix H.1**.

For sediment modeling, similar to TWIG's basin-wide model (Meselhe et al., 2017), both sand (defined as non-cohesive sediment with median grain size (d_{50}) $\geq$ 63 µ) and fines (defined as cohesive sediment with d_{50} < 63 µ) were modeled. The fines have been divided into silt and clay similar to all the previous modeling efforts by TWIG. The sand sizes are further divided into 83 µ, 125 µ and 250 µ sizes. Both suspended load (sediment moving in from upstream) and the bed material load (sources from the local sand bar) is modeled in Delft3D which helps to identify the distinct capture behavior of sand by various types of intake invert elevation combinations. The sediment concentrations were specified similar to the basin-wide model that is developed in parallel with and applied by TWIG for CPRA for this project.

8.5.6 Model Results of Energy Loss and SWR

To quantify the conveyance performance the total energy head loss was used. The total energy head is defined as:

$$Total\ Energy\ Head \quad = \frac{v^2}{2g} + WSE$$

Where, *v* indicates the depth-averaged velocity along the centerline of the structure, *g* is the gravitational acceleration and *WSE* is the water surface elevation in reference to the NAVD88 datum. The total energy loss from the MR to the outlet of the Conveyance Channel is used to evaluate hydrodynamic friction, expansion and contraction losses caused by the Intake Structures.

The main energy losses occur at two locations. First, where the river flow enters the structure between the intake and the interior U-Frame walls (for the U-Frame alternative) and second, just after the gated structure and through the vertical and horizontal transitions into the Conveyance Channel. The U-Frame structures have larger energy losses due to the presence of helical vortices compared to the Open

Channel. In general, the energy loss decreases as inverts become shallow. This is primarily due to the fact that as the shallower invert configurations have larger width (to maintain the cross-sectional area). The entrance and exit transitions in case of wider configurations are more gradual resulting in reduced contraction/expansion losses. Also, as the invert is deepened, the vertical transition becomes more abrupt causing more losses. Note that the EL -20 invert has no invert change transitioning into the Conveyance Channel. The submerged culvert at EL -50 , has better conveyance than that at EL -40 because the design for the EL -50 submerged culvert was altered to have a larger opening size (25 feet) than that at EL -40 (20 feet). Table 8-1 shows the structures ranked by conveyance (defined as the difference in energy head between the upstream MR and at the end of the downstream Conveyance Channel).

Table 8-1: Total Energy Head Loss for the Intake Structures

Structure Type	Invert (EL)	Total Energy Head Loss (ft)	
		High Flow	Low Flow
Open Channel	-20	1.81	1.22
Open Channel	-40	2.32	1.27
Open Channel	-50	2.38	1.17
U-Frame	-20	2.43	1.27
Submerged Culvert	-50	2.89	1.47
U-Frame	-40	3.71	1.83
U-Frame with Walls	-40	4.02	1.99
Submerged Culvert	-40	4.16	1.82

The total energy loss calculated for the High Flow scenario case was used in the decision matrix to select diversion intake alternative.

The DT had initially proposed to evaluate turbulence generating structures in terms of monopiles, but the concept was not carried forward. This is because the modeling showed that sufficient sediment suspension exists in the system to meet the target SWR close to 1.0 without the need of additional turbulence structures. Also, the presence of such structures, would lead to additional energy loss through the structures, which is a very important quantity that needs to be managed for better diversion capacity.

To quantify the sand capture performance, the sediment to water ratio at steady state was used and it is defined as

$$\text{Steady State Sediment To Water Ratio (SSSWR)} = \frac{\text{SSSL}_d/\text{SSSL}_r}{SSQ_d/SSQ_r} = \frac{\text{ParticleRate}_d/\text{ParticleRate}_r}{SSQ_d/SSQ_r}$$

where, SSSWR denotes the steady state sediment to water ratio, SSSL is the steady state sediment load, SSQ is the steady state discharge and Particle Rate indicates the particle passing rate at any given location. The subscripts *d* and *r* indicate diversion and river, respectively. The information on particle passing rates and flow discharge rates at both the diversion and immediately downstream of the MR was obtained from FLOW-3D results to calculate the final steady state sediment to water ratio.

Table 8-2 shows the ranking of the structures based on the total SWR for sand. The total SWR was used in the decision matrix to select diversion intake alternative.

Table 8-2: SWR for the Intake Structures from FLOW-3D at High Flows (1M cfs MR)

Structure Type	Invert (EL)	SWR: Total Sand	SWR: 250 µ Sand	SWR: 125 µ Sand	SWR: 83 µ Sand
Open Channel	-40	1.22	1.34	1.30	0.97
U-Frame	-20	1.20	1.29	1.31	0.96
U-Frame	-40	1.18	1.30	1.28	0.90
Open Channel	-50	1.17	1.28	1.28	0.92
Open Channel	-20	1.16	1.22	1.27	0.93
Submerged Culvert	-50	1.15	1.36	1.16	0.91
U-Frame with Walls	-40	1.11	1.22	1.19	0.87
Submerged Culvert	-40	1.01	1.31	1.01	0.69

The Open Channel with an invert of EL -40 (herein referred to as “Open Channel -40”) shows the highest SWR followed by the U-Frame with an invert of EL -20, (herein referred to as “U Frame -20”). Note that the SWR for difference size fractions increase with particle size. This is because of the variation in the distribution of these particles across the river. In the west bank region (where the diversion is located), the depths are shallow (10-50 feet) and the source of the suspended sand is primarily the sand bar that will have coarser sand. Towards the middle of the river (the east bank) with depths of 80 to 120 feet, the source of suspended sand is the river load which is coming from upstream with finer sand which is easier to suspend.

Calibrating the model to the physical observations by seeding particles near the surface allows the model to reproduce this cross-sectional variation. This causes the shallower west bank to have coarser particles (250 microns, 125 microns) near the bed which can enter the diversion. The finer particles that are uniformly distributed show a SWR approximately equal to 1.0. It is to be noted that though FLOW-3D was used here because of its superior hydrodynamics, a more robust sediment transport model like Delft3D which integrates bed morphology changes with suspended and bed load transport should also be used to compute values of the SWR. The difference in SWR capture efficiency by the different structures is correlated well to the difference in the zone in the MR from which the diversion draws the water under steeper velocity gradients. The higher SWR is a result of larger withdrawal zones with steeper velocity gradients. This is explained in greater details in **Appendix H.1**.

Table 8-3 shows the sand SWR values obtained from the Delft3D simulations. The SWR for fines was found to be 1.01 for all structures and hence does not impact the relative ranking of the structures. For the LF and MF scenarios, the U-Frame with an invert of EL -20 shows the highest SWR followed by the Open Channel with an invert of EL -20. The HF scenario shows the Open Channel with an invert of EL -40 to be the highest SWR followed by the U-Frame with an invert of EL -20. Note that the relative difference in sediment capture performance decreases at HF and is more prominent at LF and MFs.

In general, SWR for individual sand size fractions increases with particle size except at HF. This is because the smaller the particle size, the greater is the chance of it being in the suspended load and being well distributed across the river; this means a greater amount of it can be bypassed around the diversion when compared to the amount entering the diversion. Thus, the coarser particles are more likely to be locally sourced, hence greater is its relative concentration on the sand bar (near the diversion intake) compared to the Thalweg and the LDB, and hence the greater the SWR of coarser particles. At HF the medium to coarse sand on the sand bar enters into the suspension and the effect of the locally sourced sediment diminishes, yielding an almost uniform SWR for all sand classes.

Table 8-3: SWR for the Intake Structures (Delft3D)

Structure Type	Invert (ft NAVD88)	Sand 250 µ	Sand 125 µ	Sand 83 µ	Silt	Clay	Total Sand	Total Fines
		SWR: High Flow (1,000,000 cfs)						
Open Channel	-40	0.83	0.88	0.88	1.01	1.00	**0.87**	1.01
UFrame	-20	0.83	0.86	0.87	1.01	1.00	**0.86**	1.01
U Frame with Walls	-40	0.80	0.86	0.88	1.01	1.00	**0.86**	1.01
U Frame	-40	0.78	0.85	0.87	1.01	1.00	**0.85**	1.01
Open Channel	-20	0.78	0.84	0.86	1.01	1.00	**0.83**	1.01
Open Channel	-50	0.72	0.83	0.86	1.01	1.00	**0.82**	1.01
		SWR: Medium Flow (800,000 cfs)						
		Sand 250 µ	Sand 125 µ	Sand 83 µ	Silt	Clay	Total Sand	Total Fines
UFrame	-20	1.42	1.20	1.15	1.01	1.00	**1.21**	1.01
Open Channel	-20	1.33	1.17	1.13	1.01	1.00	**1.18**	1.01
Open Channel	-40	1.23	1.14	1.12	1.01	1.00	**1.14**	1.01
U Frame	-40	1.08	1.07	1.08	1.01	1.00	**1.08**	1.01
U Frame with Walls	-40	1.10	1.08	1.08	1.01	1.00	**1.08**	1.01
Open Channel	-50	0.82	0.98	1.04	1.01	1.00	**0.98**	1.01
		SWR: Low Flow (600,000 cfs)						
		Sand 250 µ	Sand 125 µ	Sand 83 µ	Silt	Clay	Total Sand	Total Fines
UFrame	-20	1.63	1.34	1.22	1.01	1.00	**1.33**	1.01
Open Channel	-20	1.54	1.32	1.21	1.01	1.00	**1.30**	1.01
Open Channel	-40	1.18	1.17	1.13	1.01	1.00	**1.15**	1.01
U Frame	-40	1.13	1.15	1.12	1.01	1.00	**1.13**	1.01
U Frame with Walls	-40	1.11	1.14	1.11	1.01	1.00	**1.12**	1.01
Open Channel	-50	0.97	1.10	1.10	1.01	1.00	**1.08**	1.01

It is observed that the SWR for sand decreases for increasing flow. This is because as the flow increases, the sand flux passing the deeper (near the thalweg) part of the river channel increases faster than that passing near the shallower inverts (EL -40 to -50), which causes a slower increase in sand load entering the diversion than the amount of sand bypassing the diversion.

Also, the shallower inverts have a better SWR, particularly at low and medium flows. This was also observed by TWIG in their invert screening modeling (Liang et al., 2017). As the invert becomes shallower the intake width increases, maintaining the same cross-sectional area (approximately 125 feet for EL -40 invert to approximately 240 feet for EL -20 invert). The increase in width entrains more suspended sediment which offsets for the decrease in local concentration at the diversion toe and accordingly increases the total load diverted.

Similar to the FLOW-3D SWR, the Delft3D SWR values were used for the decision matrix used to select diversion intake alternative. The total SWRs for the low, medium and high flows were aggregated to determine weighted averaged SWR using annual exceedance probabilities of the flows as weights. Table 8-4 shows the total weighted SWR values for each structure. These were used in the decision matrix. The MR flow exceedance probability curve is shown in Figure 8-5.

Table 8-4: Weighted Total SWR for the Intake Structures (Delft3D)

Structure Type	Invert (ft)	Total sand SWR			Weighted SWR
		Low Flow 600,000 cfs 73 days	Medium Flow 800,000 cfs 46 days	High Flow 1,000,000 cfs 29 days	
U-Frame	-20	1.33	1.21	0.86	1.20
Open Channel	-20	1.30	1.18	0.83	1.17
Open Channel	-40	1.15	1.14	0.87	1.09
U-Frame with interior walls	-40	1.12	1.08	0.86	1.06
U-Frame	-40	1.13	1.08	0.85	1.06
Open Channel	-50	1.08	0.98	0.82	1.00

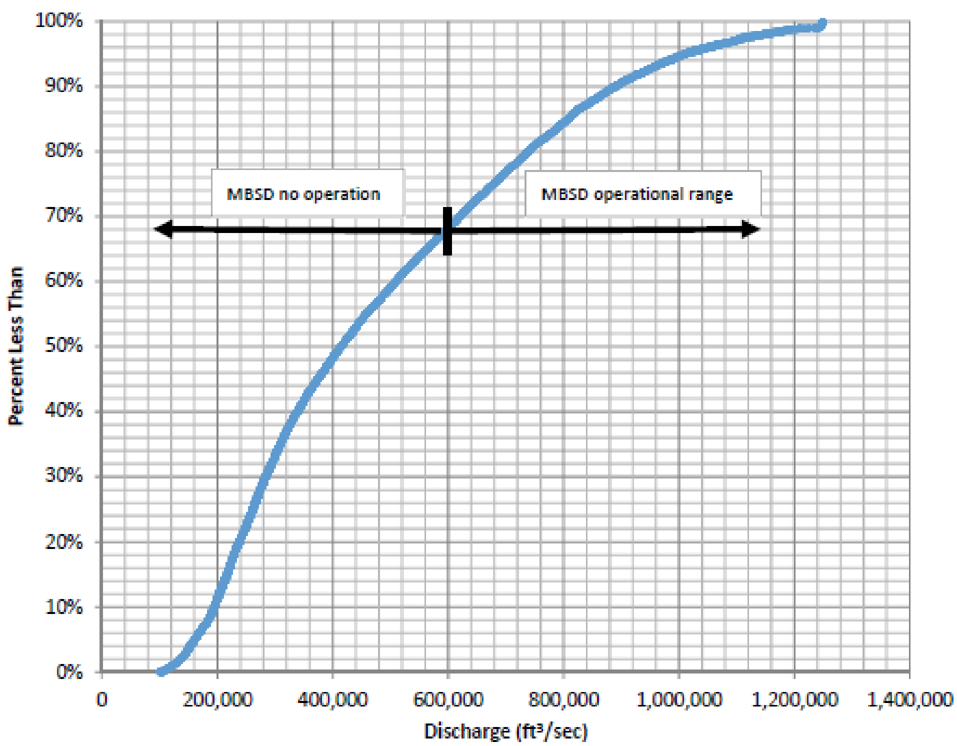


Figure 8-5: Occurrences of Mississippi River at Tarbert Landing Based on Data from 1961 to 2012 (Reproduced from HDR, 2014)

8.5.7 Optimization Testing for Intake Channel

Four modifications to the base geometry were evaluated in this study. The geometries were based on the Open Channel -40 feet alternative from FTN's study (FTN, 2018). The base geometry was modeled in a FLOW-3D model that includes 3.75 miles ft of the Mississippi River and about 1,000 feet of the Conveyance Channel from the narrowest part of the intake to the downstream end of the intake (see **Appendix H.5**). The width of the Open Channel with vertical side walls is about 150 feet with a length of 815 feet. The river bathymetry was the same as that used by FTN (FTN, 2018) The Open Channel is split into three equal widths with two divider walls that extend the full length of the Open Channel (Figure 8-6). The invert elevation of the 150 feet wide channel is -40 feet and the profile is shown in Figure 8-8.

The base geometry and the geometries of the four optimizations are shown in Figure 8-6 and Figure 8-7. Figure 8-8 shows a comparison of the footprint of the base case to Optimization 1 and of Optimization 1 to Optimizations 2 and 3. Figure 8-8 shows the elevation profiles along the centerline of the optimizations. The bottom elevation of Optimizations 1 and 2 is -40 feet from the intake entrance to just past the gate piers, where the bottom elevation slopes upward until it reaches EL -25 in the transition to the conveyance channel. The bottom elevation of Optimization 3 slopes upward from EL -40 starting at the intake entrance and reaches EL -20 at the upstream end of the gate piers. The base condition was not rerun as part of the optimization. Results for the base condition that are shown in Section 3.0 are taken from FTN (FTN, 2018).

Optimization 1 has a wider zone of influence as the training walls were angled further away from the channel center, the training wall top elevations were also lowered to allow more overtopping flow. The open channel in Optimization 1 is the same as in the base condition. All three optimizations extend to Station 22+00 (the same as the base intake geometry). The three optimizations flare out and end at approximately the same horizontal location (X-Y). The ends of the training walls also stop at the same river contours; the upstream wall at the EL -10 contour and the downstream wall at EL -25. The wall top stepped elevations were kept the same for all optimizations and the lengths were changed slightly to agree with the horizontal geometry.

Optimization 1A is the same as Optimization 1 except that the interior walls were removed and the bridge low chord was raised from 4.9 feet to 8.0 feet (1.5 m to 2.4 m).

Optimization 2 is an open channel with a gradually varying width that is widest at the Mississippi River. The opening width at the upstream end of the walls is 605 feet, and the invert EL -40. Optimization 2 has a riprap channel between the gate piers and the riverside end of the intake.

Optimization 3 has a more gradual taper in the vertical walls of the open channel than Optimization 2. The opening width at the upstream end of the vertical walls is 642 feet and the invert EL -40. The bottom profile of Optimization 3 differs from the other models and is shown in Figure 8-8. The cross sectional area of Optimization 3 closely agrees with that of Optimization 2 (this was accomplished by matching cross sectional areas at the gated structure, Station 28+00, and Station 25+00). The width of the Optimization 3 section was increased as needed to equal the area of the deeper Optimization 2 section. Optimization 3 has a riprap open channel between the gate piers and the riverward end of the intake, whereas Optimization 1 and the base case have the open channel at the intake followed by a rectangular concrete section divided into three channels by vertical concrete walls.

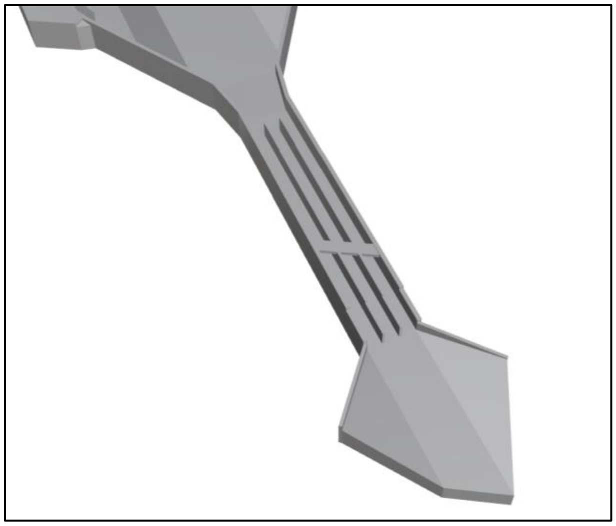


Figure 8-6: Open Channel EL -40 Invert Base Condition

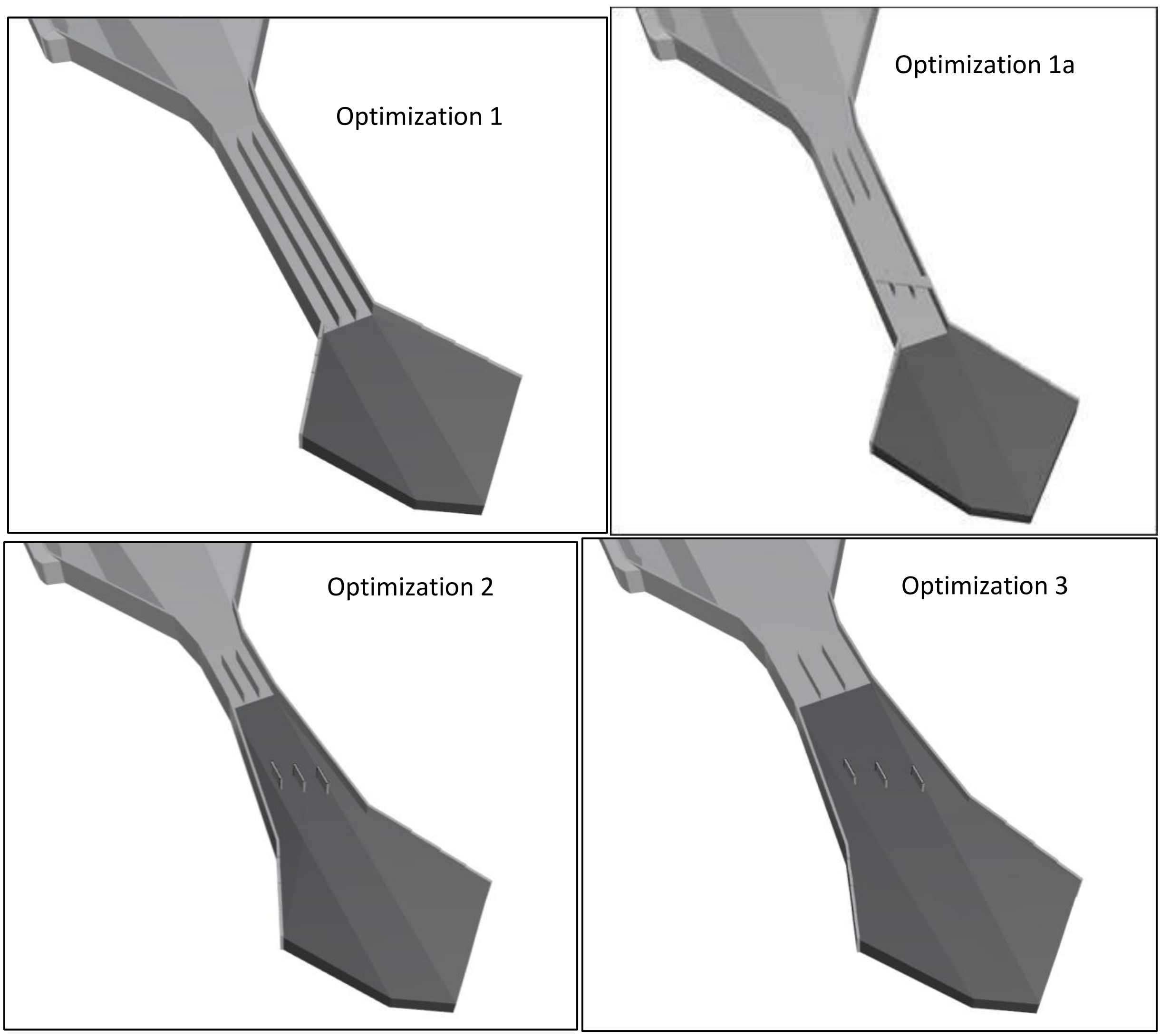


Figure 8-7: Open Channel EL -40 Invert Optimizations

A footprint comparison of the four alternatives is shown in Figure 8-8. The figure shows that each successive optimization provides a more gradual transition entering the diversion.

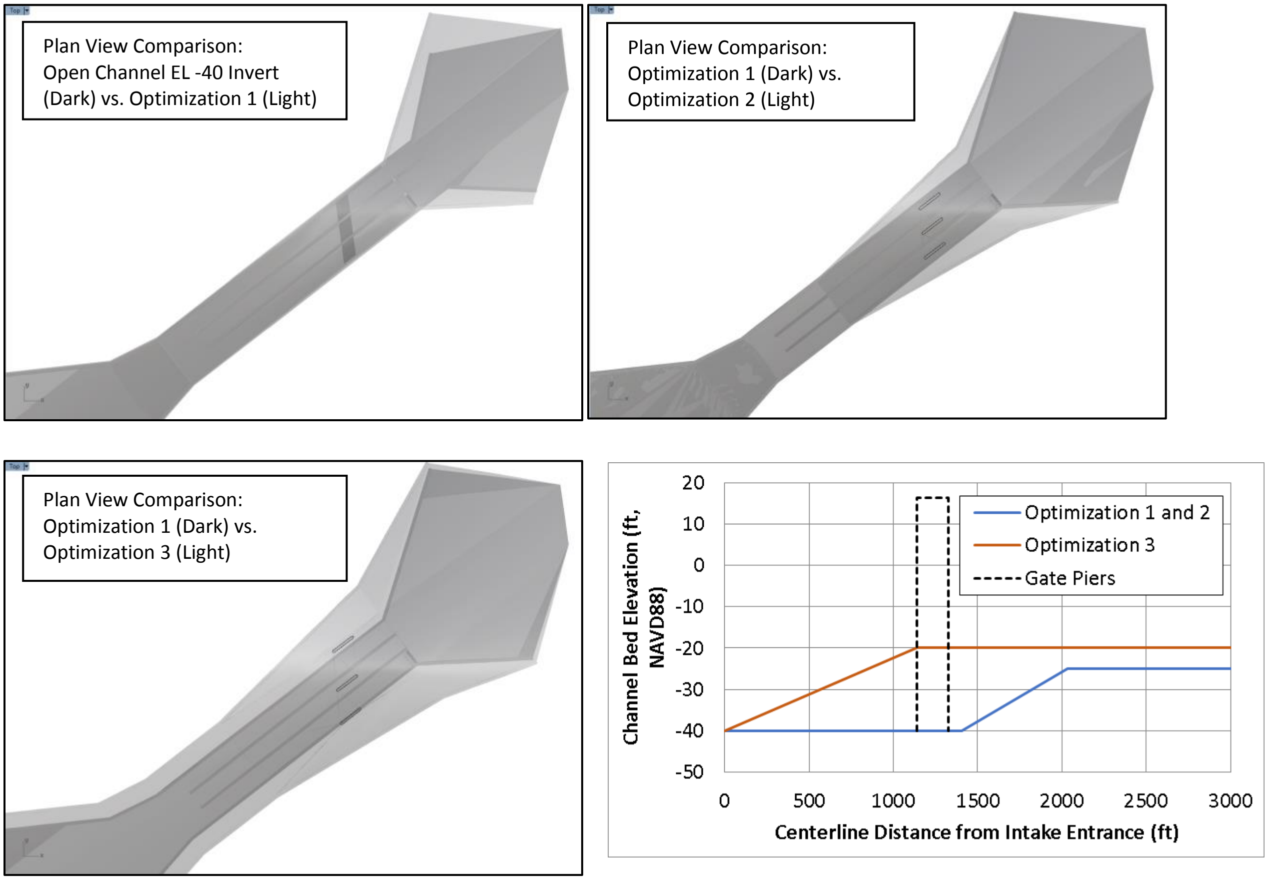


Figure 8-8: Footprint and Profile Comparison of Optimization Alternatives

Simulations were completed for low flow and high flow conditions. Boundary conditions for the model simulations were as shown in Table 8-5. The surface roughness for the model components is shown in Table 8-6.

Table 8-5: Model Boundary Conditions for FLOW-3D Optimization Simulations

Model Run	Upstream Flow Rate (cfs)	Diversion Flow Rate (cfs)	Downstream WSE (ft, NAVD88)
High Flow	1,000,000	75,000	7.81
Low Flow	600,000	48,000	3.48

Table 8-6: Roughness Heights used in FLOW-3D Model Optimization Simulations

Surface	Roughness Height (m)
River bed	0.600
Riprap	0.457
Concrete	0.006

Results
Each model was run until the hydrodynamic solver reach steady state flow. Particles were then added to the flow field and the simulation continued until the sediment water ratio reached steady state conditions. For each simulation, the total energy loss and the sediment water ratio (SWR) was determined. The SWR values are shown in Table 8-7 and the energy loss values are shown in Table 8-8. Water surface profiles are provided in **Appendix H.5**.

Table 8-7: Sediment to Water Ratio for Optimization Testing

Structure	SWR			
	Total Sand	250 µ Sand	125 µ Sand	83 µ Sand
Base Case	1.22	1.34	1.30	0.97
Optimization 1	1.10	1.07	1.23	0.96
Optimization 1a	1.13	1.10	1.25	0.97
Optimization 2	1.00	0.94	1.11	0.90
Optimization 3	0.78	0.66	0.92	0.72

Table 8-8: Total Energy Loss for Optimization Testing

Structure	High Flow Total Energy Head Loss (ft)	Low Flow Total Energy Head Loss (ft)
Base Case	2.32	1.27
Optimization 1	1.94	1.09
Optimization 1a	1.89	1.00
Optimization 2	1.90	0.95
Optimization 3	1.76	0.99

For each simulation the shear stress and depth averaged velocity was computed. Figure 8-9 shows contour plots of bed shear stress and Figure 8-10 shows depth averaged velocities for three of the optimizations. Optimization 1 has higher shear stress and depth averaged velocity extending further into the river than in Optimizations 2 and 3. Abrupt changes in the shear stress are attributable to changes in the surface roughness.

Optimizations 1 and 1A had the highest SWR and highest head loss of the three evaluated alternatives. Optimization 1 and 1A had a lower SWR than the base case. The more gradual contraction of Optimization 1 compared to the base case reduced the head loss, but also decreased the zone of influence of the intake. Optimizations 2 and 3 had even more gradual transitions to the concrete channel than Optimization 1, and had lower SWRs and head losses. The optimization geometries all had lower head losses than the base case because the expansion downstream of the gate piers was longer than in the base geometry and because the flared intake walls were lower in the optimization geometries.

Based on the FLOW-3D modeling, the preferred diversion is Optimization 1A. Optimization 1A has a SWR greater than 1.1 while realizing a significant reduction in head loss from the base condition. This conclusion may change once the Delft3D modeling has been completed, and it may also be influence by the numeric modeling of the upsizing condition. The preferred diversion stated here is based only on the hydraulic and sediment capture performance from the FLOW-3D modeling.

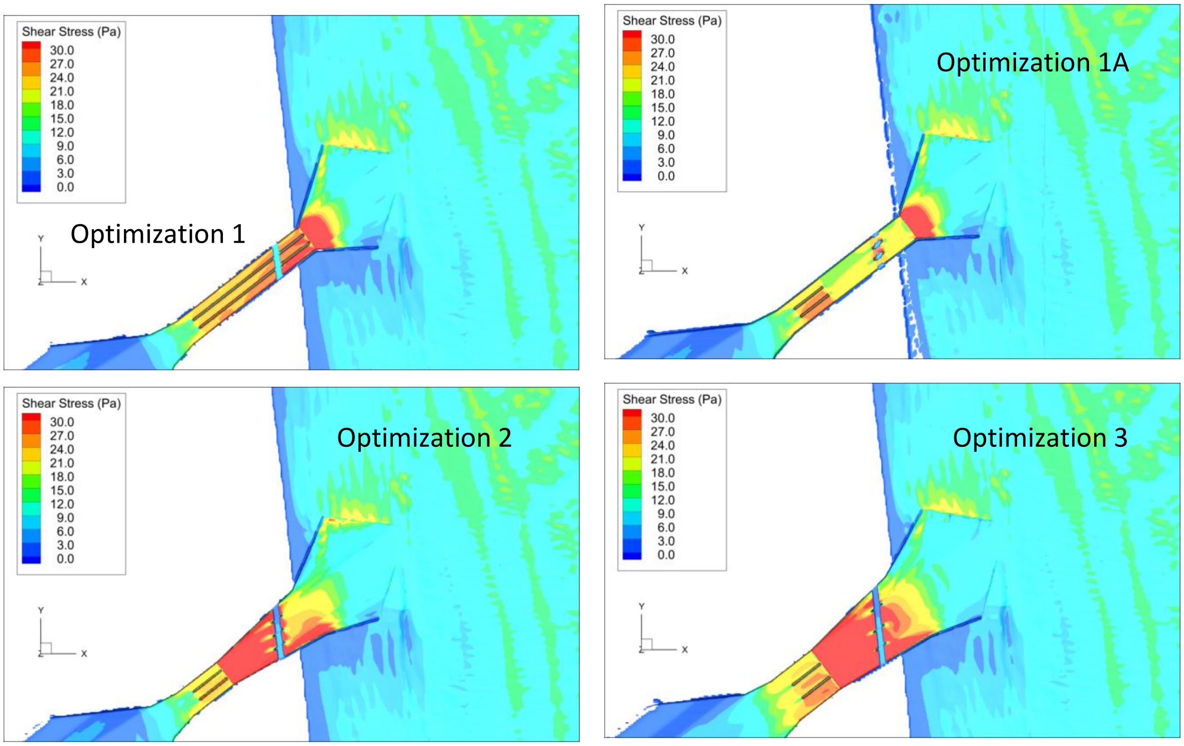


Figure 8-9: Shear Stress

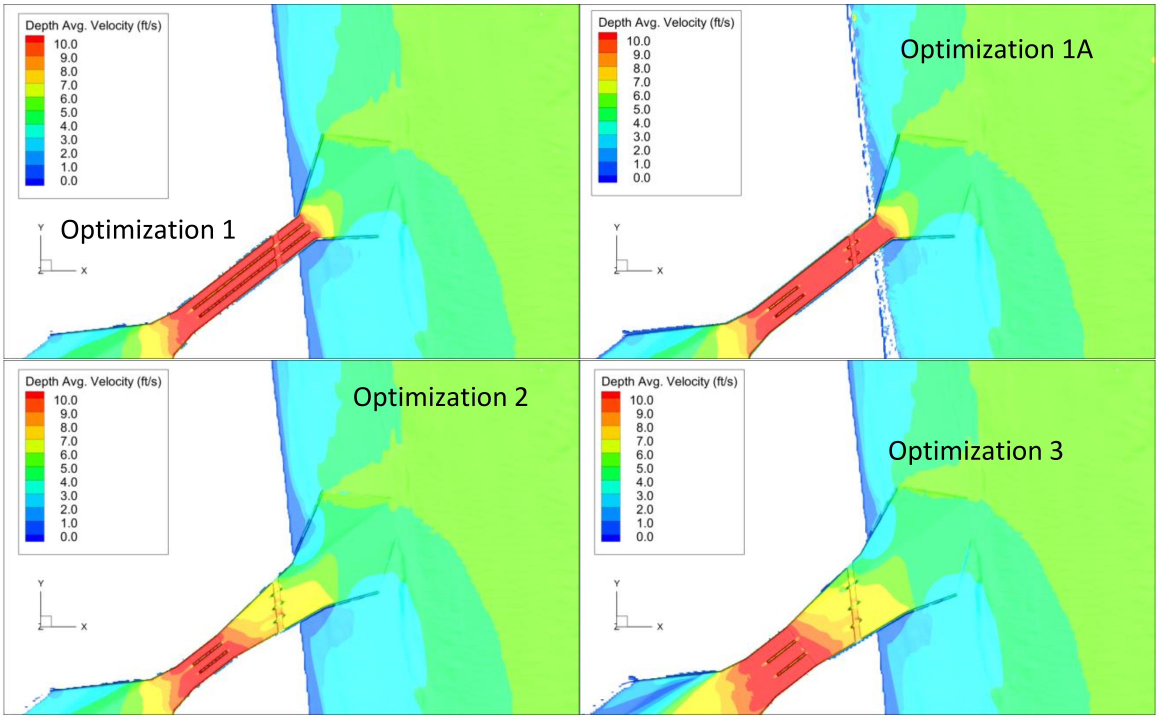


Figure 8-10: Depth Averaged Velocity Contours

8.6 Conveyance Channel Numerical Modeling

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.

Key design requirements of the channel are:

1) Convey design flow and SWR without any erosion or deposition in the channel.
2) Limit head loss along the channel – this provides the most flexibility for adjusting flows in the future, since the head loss can always be increased by reducing the Diversion Gate openings.
3) Limit armoring costs.
4) Limit degradation of water quality when diversion is not operational.

The first requirement 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 baseline design for the Conveyance Channel cross-section was provided in the 2014 Basis of Design Report (HDR, 2014). The cross-section geometry is shown in Figure 8-11 and consists of a 300-foot wide base at EL -25. The side slopes of 1:4 extend laterally until EL 2. At that point a berm extends laterally 97 feet increasing to EL 4. The berms are necessary to provide a stable platform for the channel guide levees. The total width of the Conveyance Channel is 734 feet.

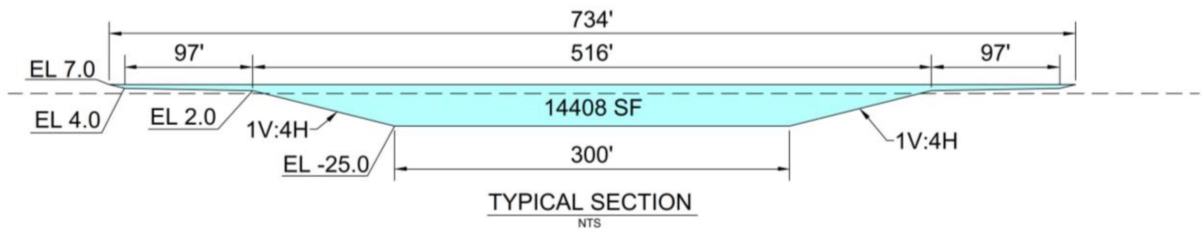


Figure 8-11: Conveyance Channel Cross-Section

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 8-12. 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 details are provided in **Appendix H.2**. The Delft3D structured grid was used with a constant grid cell size of 32.9 by 65.6 feet (10 by 20 meters).

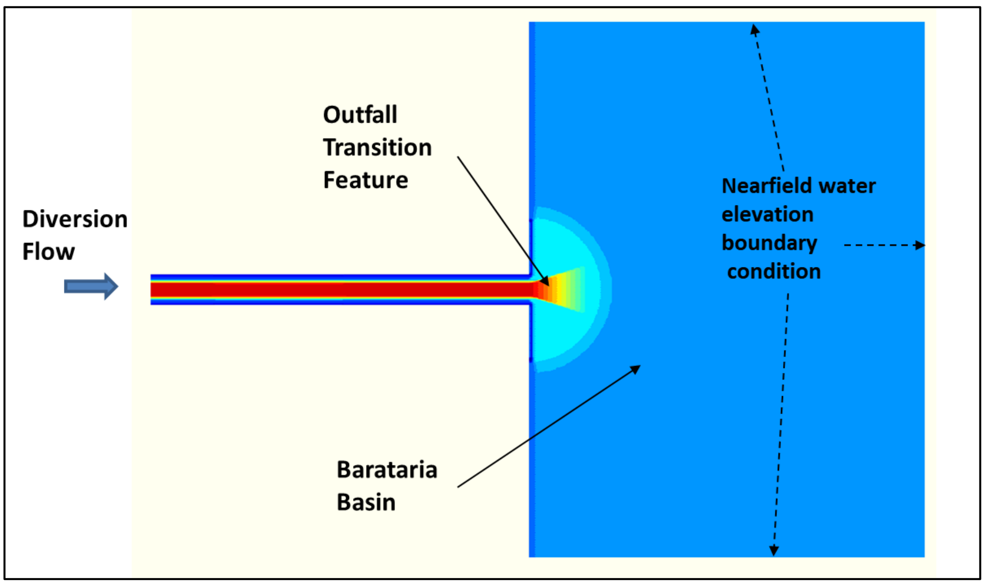


Figure 8-12: Delft3D Model Domain for Conveyance Channel Analysis

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. Details of their development are available in **Appendix H.2.**

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.

Since the final relationship between the diversions flows and loads has not been established, these results are considered preliminary. However, it is expected that the current baseline design is close to the final design and no further analysis has been conducted at this time. When the final intake design is completed, and the diversion flow and load conditions are established, the cross-section geometry will be evaluated with the new data. However, it is not expected that the design will vary significantly from its current configuration.

The last requirement, limiting water quality degradation, is considered an operational objective, and does not impose design constraints on the Conveyance Channel geometry. Therefore, it has not been addressed in this alternatives analysis. Operational strategies for maintaining water quality objectives, such as periodic flushing of the channel will be evaluated in a subsequent Phase of the project.

The remaining two design requirements, minimizing head losses and minimizing armoring costs have been directly addressed in an alternatives analysis of the Conveyance Channel design. The channel, in addition to transport of flow and sediment from the MR past the NOV Levee, must accommodate the elevation change from the bottom elevation of the intake to the basin elevation. This analysis of the Conveyance Channel was completed in parallel with the intake analysis, and consequently the final selection of the intake invert was not made until after this analysis was completed. However, at the time of this analysis, two intake bottom elevations had become more plausible and those two are considered in this analysis. The intake bottom elevations for the two more likely designs are EL -40 and -20. Note that the EL -40 intake design currently includes an expansion ramp that both expands the intake cross-section from 148 feet to the channel width of 300 plus feet while simultaneously raising the bottom elevation from EL -40 to -25. This is again based on preliminary designs from the 2014 Basis of Design Report (HDR, 2014), and are retained in this analysis.

In summary, there are two intake configurations that are considered, herein referred to as:

a) EL -40 Intake Configuration
b) EL -20 Intake Configuration

Another feature of the diversion system that will impact the Conveyance Channel design is the Outfall Transition Feature. This feature is essentially a ramp that extends from the end of the Conveyance Channel into the basin with the role of gradually increasing the bottom elevation from the base of the Conveyance Channel to the basin elevation, which is nominally EL -4. The gradual transition is intended to reduce head losses as the flow accelerates across the Outfall Transition Feature. A separate analysis of the Outfall Transition Feature has been conducted (see Section 8.8). For the Conveyance Channel alternatives analysis discussed here, the Outfall Transition Feature was set at 1,500 feet and the slope was dictated by the Conveyance Channel elevation at the Outfall.

The Conveyance Channel alternatives analysis consisted of comparing the head losses and armoring requirements for three alternative channel planforms. The analysis was completed separately for the EL -40 Intake Configuration and the EL -20 Intake Configuration. A schematic of each alternative configuration is shown in **Appendix D**, and they are briefly summarized below.

The three configurations for the EL -40 Intake Configuration consisted of the following alternative planforms:

Alt1: Constant flat 300-foot wide bottom at EL -25.

Alt2: Two sections of continuously sloping bottom, the first covering 75% of the channel, starting at EL -25 and ending at EL -15. The second section covers the remaining 25% and starts at EL -15 and ends at EL -7 at the Outfall.

Alt3: Flat 300-foot wide bottom at EL -25, then changing to a constant slope over the last 25% of the channel, ending with EL -12 at the Outfall.

The three configurations for the EL -20 Intake Configuration consisted of the following alternative planforms:

Alt1: Constant flat 300-foot wide bottom at EL -20.
Alt2: Two sections of continuously sloping bottom, the first covering 75% of the channel, starting at EL -20 and ending at EL -12. The second section covers the remaining 25% of the channel, starting at EL -12 and ending at EL -7 at the Outfall.
Alt3: Flat 300-foot wide bottom at EL -20, then changing to a constant slope at EL -10 feet at the Outfall.

For each alternative Conveyance Channel configuration, the channel width was increased as the bottom elevation increased so that the cross-section area remained constant. This is necessary to assure that a sufficient flow speed is maintained to support the sediment load and prevent deposition. Also, the slope of the Outfall Transition Feature naturally changed as the bottom elevation of the channel at the Outfall changes (i.e., where the Outfall Transition Feature begins).
A model of the Conveyance Channel and nearfield basin was developed with the Coastal Modeling System (CMS) and used to evaluate the hydraulic performance of each alternative Conveyance Channel profile. The model domain is shown in Figure 8-13 and was configured to simulate a designated flow through the channel across the Outfall Transition Feature and into the basin. The flow and boundary conditions shown correspond to a 75,000 cfs diversion flow. The simulated head loss was measured from the upstream end of the Conveyance Channel out into the basin, so that the adjustments to the Outfall Transition Feature associated with each alternative were included in the assessment. Two flow rates of 75,000, and 40,000 cfs were used to span the range of expected diversion flows. Additional details of the model configuration and boundary conditions are available in **Appendix H.2**.

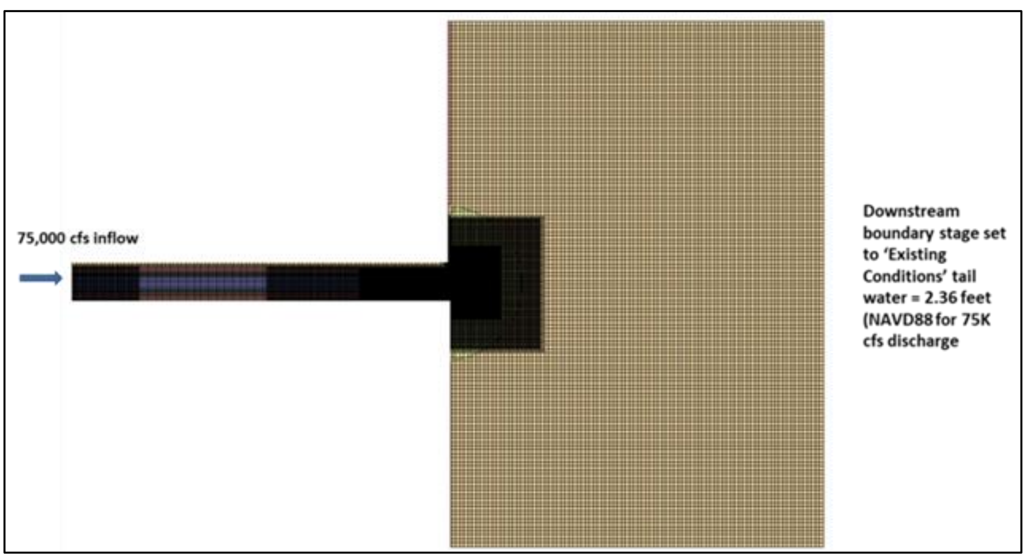


Figure 8-13: CMS Model Configuration For Evaluating Alternative Conveyance Channel Profiles

The results of the hydraulic analysis are shown in Table 8-9.

Table 8-9: Summary of Alternatives Analysis

Configuration	Flow Rate (cfs)	Head Loss (feet)		
		Alt1	Alt2	Alt3
-40 ft Intake	75,000	1.80	2.62	1.97
	40,000	0.92	1.28	1.00
-20 ft Intake	75,000	1.85	2.47	2.18
	40,000	0.96	1.18	1.14

For both intake configurations, Alternative 1 provides the minimal head loss.

8.7 Back Structure Numerical Modeling

The primary and initial motivation for the Back Structure, herein referred to as the "Back Gates", is to provide flood protection for the same flood and wave conditions as the NOV Levee. In concept, the Back Gates would be closed during extreme storm conditions, effectively continuing the NOV Levee protection across the Conveyance Channel outfall.

An alternative means for providing flood protection has been developed, which consists of designing the Conveyance Channel guide levees to the same flood protection standards as the NOV Levee. This approach would eliminate the need for the Back Gates as a flood protection feature. The approach would allow the flood waters and storm waves to enter the Conveyance Channel, and they would be prevented from impacting the area between the MR and NOV Levees by the Conveyance Channel guide levees. The costs of the Back Gates option is considered substantial relative to increasing the Conveyance Channel guide levee design to hurricane flood protection standards, and therefore an alternatives analysis has been conducted to determine the benefits and disadvantages of the two alternatives.

The two alternatives are herein referred to as the "Open Channel" and the "Back Gates". The "Open Channel" option would extend to the NOV Levee without changes to the cross-section. Flood protection would be provided by increasing the Conveyance Channel guide levees to hurricane grade. A representative sketch at the end of the Conveyance Channel is shown in Figure 8-14.

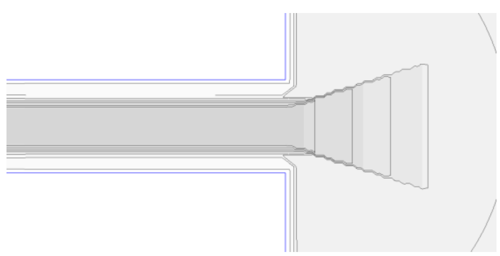


Figure 8-14: Open Channel Concept for Downstream End of Conveyance Channel

The "Back Gates" option includes a transition to a narrower cross-sectional area and then a 7-bay gate structure. A representative sketch at the end of the Conveyance Channel is provided in Figure 8-15. The Back Gates are integral with the existing hurricane protection levees, completing the flood protection system.

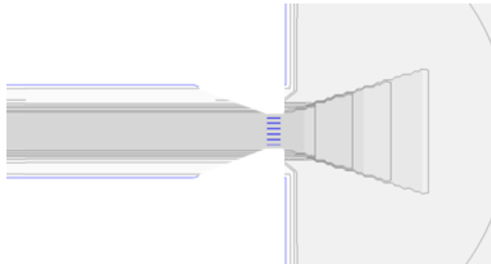


Figure 8-15: Back Gates Concept for Downstream End of Conveyance Channel

The Back Gate complex consists of two primary components. The first is training walls that funnel the flow from the side slopes and berms to the flat bottom section of the channel (approximately 300 feet wide). The Back Gate Structure then extends across the approximate 300 feet width with seven bays.

The DT, in conjunction with CPRA, has developed a set of performance measures for evaluating the two alternatives. They are:

1. Hydraulic head loss.
2. Sediment transport into the basin.
3. Management of siltation accumulating during non-operational periods.
4. Adaptive management.

Construction and maintenance costs are also a consideration but are not addressed here.

The concepts for adaptive management include three potential benefits that may be provided by the Back Gates:

- 4a. Flow jetting: by closing some of the gates (for example 5 of the 7 gates) the flow will be accelerated through the open gates. This "jetting" could be useful to periodically move sediment deposited in the Outfall Area further into the basin, potentially reducing dredging maintenance costs.
- 4b. Diversion flow management during opening and closing gates: During the opening and closing of the gates, the flow speeds in the Conveyance Channel may be slower than required to support the sediment load and deposition may occur. It may be possible to use the Back Gates to reduce the deposition.
- 4c. Radial gate configuration: The Back Gates could be oriented in a radial configuration and used to direct the diversion flow in different directions. This redirection of the flow may enhance the sediment dispersion in the basin.

Each of these seven performance measures (1, 2, 3, 4a, 4b, and 4c) was evaluated for both alternatives.

8.7.1 Head Loss

The head loss for each alternative was evaluated using the Coastal Modeling System (CMS) numerical model. The CMS model configuration for the "Open Channel" option is shown in Figure 8-16 to demonstrate the application of the boundary conditions. The head loss was evaluated for diversion flows of 75,000 cfs, 55,000 cfs and 35,000 cfs. The associated Mississippi River (MR) flows corresponding to these diversion flows are 1,000,000 cfs, 650,000 cfs and 500,000 cfs.

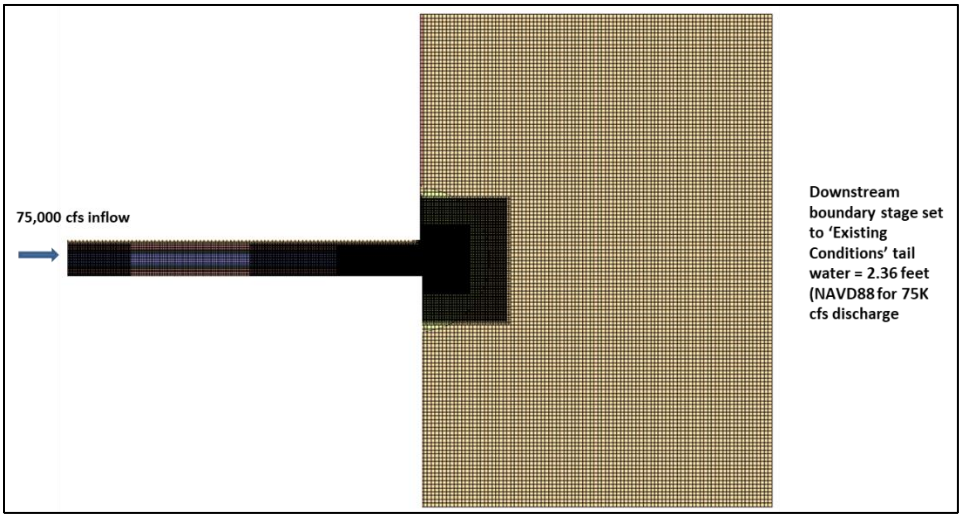


Figure 8-16: Model Grid Domain for Evaluation of Head Losses

The configuration in Figure 8-16 is for a diversions flow of 75,000 cfs. The water level at the downstream boundary is dependent on the diversion flow. Appropriate values were determined from an analysis of the existing conditions water levels from the TWIG's Basin Wide Model PR15. The TWIG Basin Wide Model simulation starts with existing conditions, and the first few years of simulation do not include land building. Therefore their simulated water elevations in the vicinity of the outfall represent the influence of the diversion flows on the water for existing conditions. The tail water elevation for 75,000 feet is 2.36 feet (0.72 meters), for 55,000 cfs diversion flow it is 2.1 feet (0.65 meters) NAVD88 and for the 35,000 cfs diversion flow the downstream water surface elevation is 1.8 feet (0.56 meters) NAVD88. Details of the model boundary conditions and bathymetry are available in **Appendix H.3.**

The hydraulic grade line (HGL) or water surface elevation for the Open Channel and Back Gate alternatives is shown in Figure 8-17 for the 75,000 cfs flow scenario. In the plot, the distance axis (horizontal) value of 4,250 feet corresponds to the end of the Intake Structure ramp/expansion and the beginning of the Conveyance Channel. The Conveyance Channel ends at 14,000 feet.

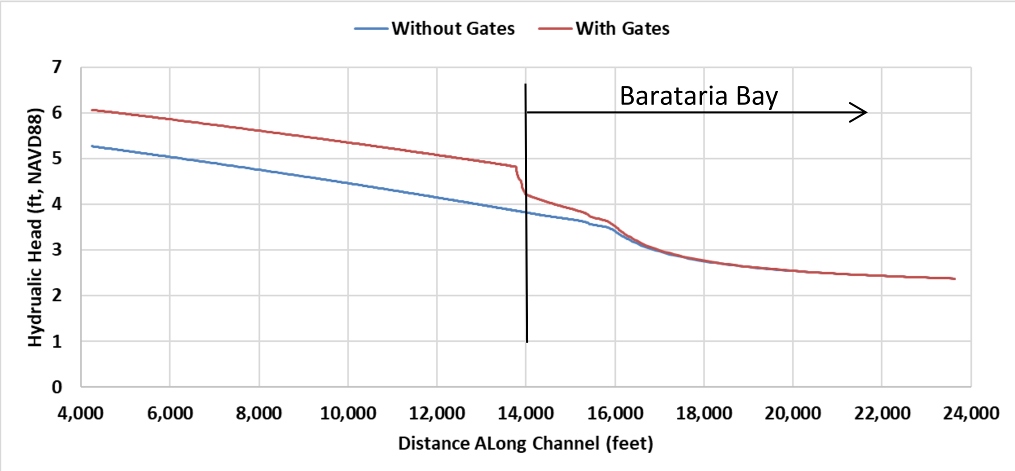


Figure 8-17: Hydraulic Grade Line Along the Channel and into the Basin (75K cfs Diversion Flow)

For all scenarios, the Back Gate alternative required a larger hydraulic head (i.e. upstream stage) due to the energy losses associated with the restricted flow cross-section (i.e., training walls), higher flow speeds, and gate bay walls. The increased hydraulic head required for the Back Gates (relative to the Open Channel alternative) are summarized in Table 8-10.

Table 8-10: Summary of Additional Head Loss for Range of Flow Rates

Flow (cfs)	Additional Head Loss due to Back Gate (ft)
75,000	0.83
55,000	0.49
35,000	0.20

These results indicate that a larger Intake Structure will be required if the Back Gates are included, to meet the design requirements of 75,000 cfs when the MR is flowing at 1,000,000 cfs. The relationship between intake size and head loss has not been established, and therefore these results cannot currently be interpreted in terms of cost associated with the increased intake design.

8.7.2 Sediment Delivery into the Basin

Conceptually, the increased flow speeds generated with the Back Gate alternative will enhance sediment transport into the basin. The enhancement consists primarily of transporting the sediment further into the basin, providing less risk of sediment accumulating in the Outfall Area and subsequently reducing the requirement for maintenance dredging.

A set of numerical model simulations was completed to evaluate the sediment transport and deposition characteristics for the two alternatives. The specific scenarios simulated are summarized in Table 8-11.

Table 8-11: Summary of Sediment Transport Model Simulations

Scenario	Alternative	Diversion Flow (cfs)	Sediment Inputs
1	Open Channel	75,000	MR Loads consistent with 1,000,000 cfs
2	Back Gates	75,000	MR Loads consistent with 1,000,000 cfs
3	Open Channel	48,000	MR Loads consistent with 600,000 cfs
4	Back Gates	48,000	MR Loads consistent with 600,000 cfs

A Delft3D model was configured for the analysis. The basic model domain is shown in Figure 8-18.

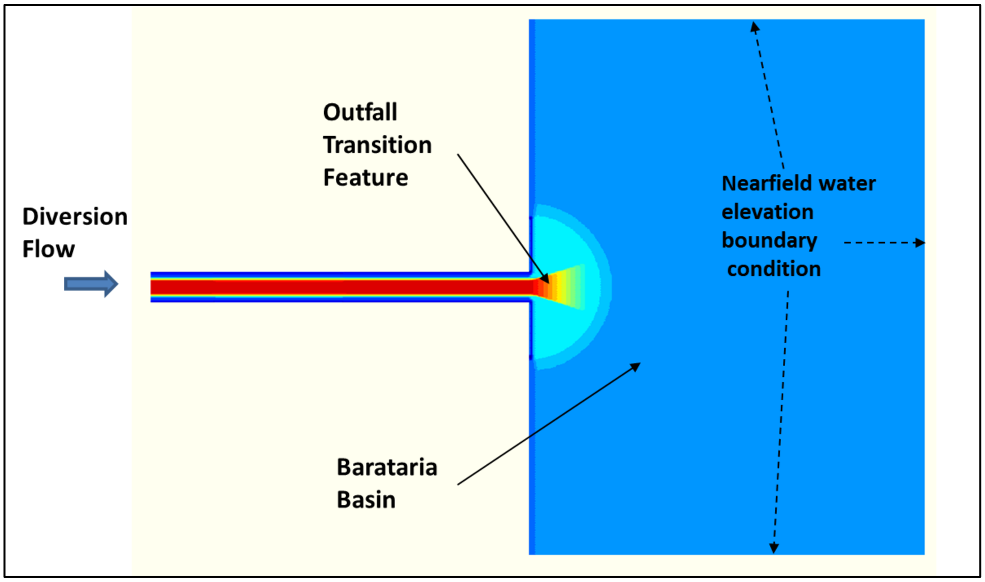


Figure 8-18: Delft3D Model Domain for Sediment Transport Simulations

It is assumed in this analysis that the diversion system will be designed (i.e., "sized") to provide 75,000 cfs for both alternatives when the MR is flowing at 1,000,000 cfs. (Thus it will be slightly larger if back gates were included in the design). The 48,000 cfs diversion flow is expected to occur when the MR flow is at 600,000 cfs. This is based on previous analysis of the diversion but may change during the design process. The suspended sediment concentration entering the Conveyance Channel was determined for 1,000,000 and 600,000 cfs diversion flow from the Belle Chasse Sand Load and Belle Chasse Hysteresis Sediment Rating Curves. Details of these the sediment input conditions is available in **Appendices H.2 and H.3**. The values used are summarized in Table 8-12.

Table 8-12: Summary of Sediment Concentrations Used in the Modeling Analysis

Flow (cfs)	Clay (mg/L)	Silt (mg/L)	0.83 mm Sand (mg/L)	0.125 mm Sand (mg/L)	0.250 mm Sand (mg/L)
48,000	37.5	112.5	7.92	8.88	7.2
75,000	50	150	19.8	22.2	18

The silt and clay erosion and settling properties were adopted for the values used in the TWIG's Basin Wide Model PR15 and are shown in Table 8-13.

Table 8-13: Parameters Characterizing the Fine Sediment Classes

Parameter	Silt	Clay
Settling Speed (mm/s)	0.1	0.001
Critical Shear Stress (Pa)	0.15	0.01
Erosion Rate (kg/m2-s)	0.001	0.001

Each scenario was simulated for 9 hours with a morphologic acceleration of 10, with a 3-hour delay in recording morphology (i.e. a spin-up period), yielding an effective simulation time of 60 hours. The results were analyzed by calculating the final deposit thickness. An example plot for Scenario 1 (open channel alternative) and Scenario 2 (with gate alternative) is shown in Figure 8-19. As the flow entered the basin the flow speeds dropped due to lateral spreading of the flow and sediment began to deposit.

There is some deposition along the berm in the Conveyance Channel, but the primary deposition occurs in the basin area outside of the ramp area. There is also significant deposition on the outer edges of the flow as it exits the channel.

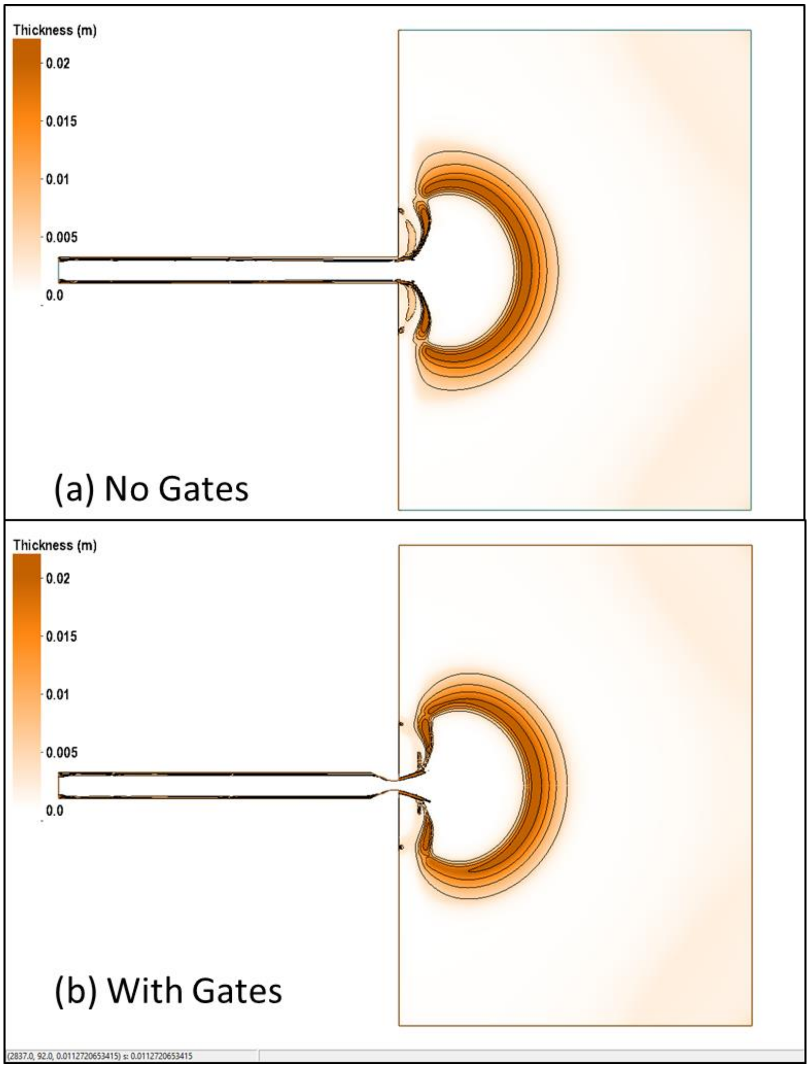


Figure 8-19: Depositional Patterns for Scenario 1 (a) and Scenario 2 (b)

The sediment deposits were recorded and their location for the Outfall determined. An example plot of the deposition along a transect aligned with the channel and extending into the basin is shown in Figure 8-20.

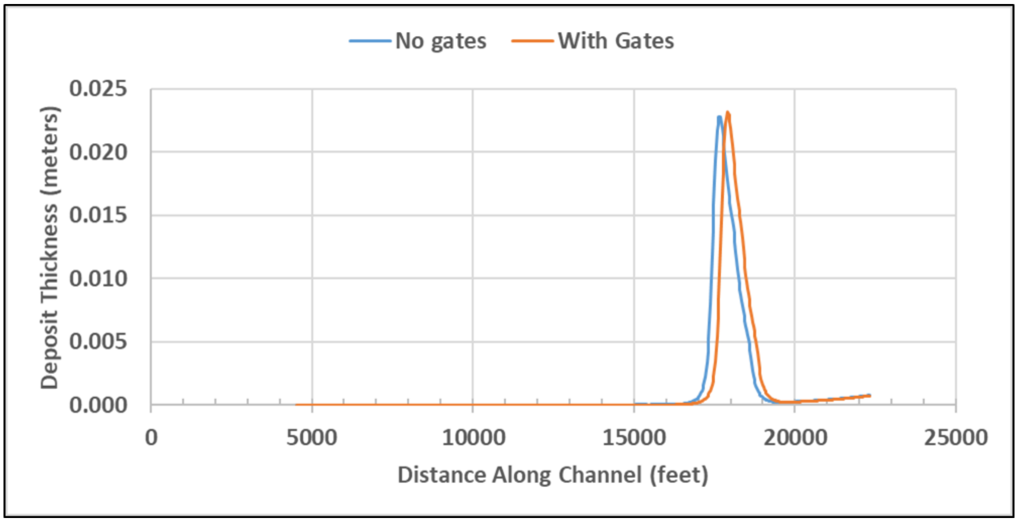


Figure 8-20: Example of Sediment Deposition in Basin for the Two Alternatives (Channel Outfall is at 14,000 Feet)

The distance from the end of the Outfall to the peak of the deposit is 3,636 feet for the "Open Channel" option and is 3,898 feet for the "Back Gates" option. For the 48,000 cfs flow scenario, the depositional peak for the "open channel" alternative is 3,060 feet from the Outfall, and 3,300 for the "Back Gates" alternative.

8.7.3 Management of siltation accumulating during non-operational periods

During non-operational periods, storm events affecting the basin may induce sediment transport, potentially causing sedimentation near the diversion outfall. The siltation analysis consists of determining the ability of the diversion flow to flush sedimentation that occurred during non-operational periods. The "Back Gates" alternative has the benefit preventing sediment accumulation in the Conveyance Channel, though accumulation in Barataria Bay will continue to occur. For the "Open Channel" alternative, sediment accumulation may propagate into the Conveyance Channel.

The analysis is comprised of two parts. The first is an estimate of reasonable siltation patterns that may occur. The second part is a modeling analysis to determine if the assumed siltation can be flushed, and if so, the time periods need to flush the deposits.

8.7.3.1 Estimate of Siltation Volumes

Reliable estimates of siltation have been developed from hydrographic surveys of the Barataria Water Way (BAWW). A set of survey data was identified and acquired for the Barataria Water Way, in response to Hurricane Rita, which provides insight into potential siltation rates related to storm activity in the basin.

Hurricane Rita made landfall on September 24, as a Category 3 hurricane at Johnson's Bayou, Louisiana, between Sabine Pass, Texas and Holly Beach, Louisiana, with winds of 115 mph. The Barataria Water Way is 12 feet deep and 125 wide. An estimate of the siltation depth was developed by comparing the two surveys. The results of the comparison are shown in Figure 8-21. A location map of the channel station is show in Figure 8-22.

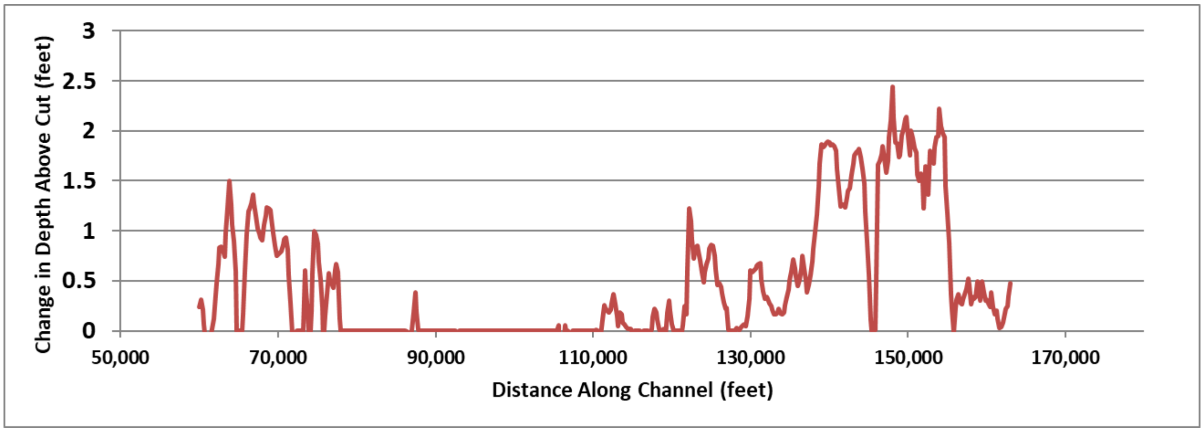


Figure 8-21: Estimated Siltation Depths Along the BAWW

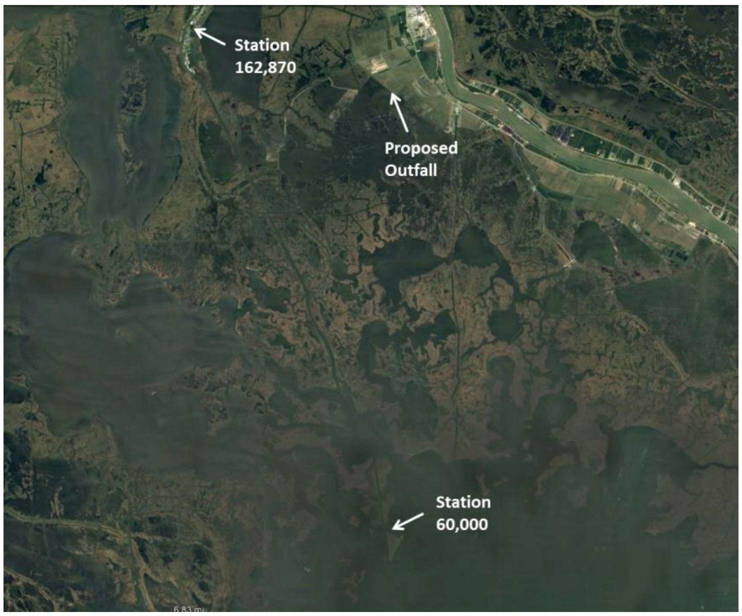


Figure 8-22: Station Location Map for the BAWW

The maximum siltation near the Outfall is approximately 2 feet. This data provides a quantitative basis for expected siltation in the basin due to extreme storm events. However, in addition to using these levels of siltation, other siltation depths and foot prints have been considered, due to the uncertainty in the expected siltation volumes. These additional volumes are considered very conservative and provide a rigorous test for evaluating the alternatives.

The siltation surface and footprint for the “Open Channel” alternative is prescribed as a surface elevation and a distance to which the surface extends into the channel. The volume also includes any portion of the ramp that is below the siltation surface elevation.

For the “Back Gates” alternative, the siltation volume and footprint are specified as a surface elevation extending over the ramp. The initial deposit is the volume above the ramp elevation and below the siltation surface elevation. These configurations are depicted in Figure 8-23 and Figure 8-24.

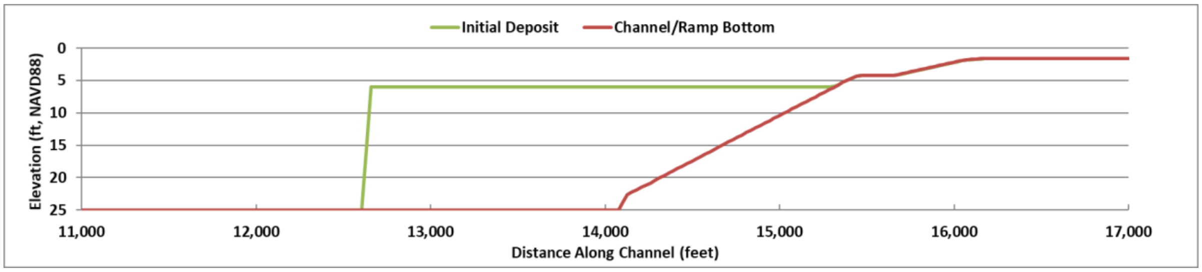


Figure 8-23: Initial Siltation Deposit for the "Open Channel" Alternative for Scenario 2 in Table 8-14

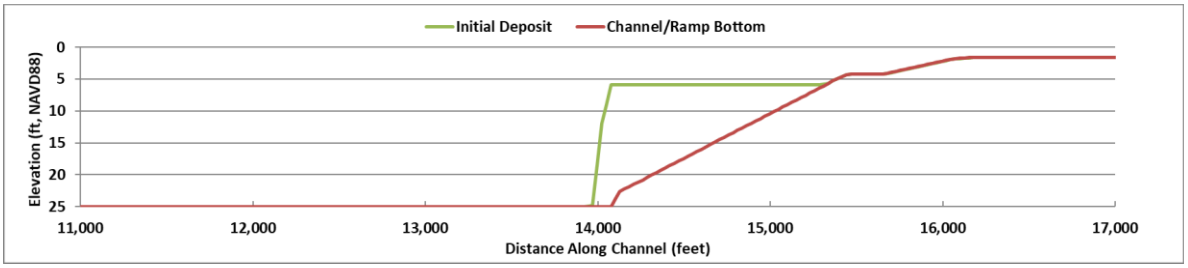


Figure 8-24: Initial Siltation Deposit for the "Back Gates" Alternative for Scenario 2 in Table 8-14

A summary of the modeling scenarios for the "Open Channel" and "Back Gates" alternatives are summarized in Table 8-14.

Table 8-14: Summary of Modeling Scenarios for "Open Channel" and "Back Gates" Alternatives

Scenario	Deposit Surface Elevation (ft, NAVD88)	Extension into Channel (feet)	No Gate Volume (cy)	Back gates Volume (cy)
1	-23	10,000*	184,789	5,489
2	-6	1,425	624,676	259,775
3	-12	2,100	454,048	88,768
4	-18	3,900	382,180	15,800

*entire length of channel

The initial deposits were assumed to be entirely composed of silt.

8.7.3.2 Model simulations of flushing

A Delft3D model simulation of the channel, ramp and basin sediment transport was completed for each of the 4 scenarios for both the "Open Channel" and "Back Gates" alternatives. The standard grid configuration was used for the analysis, with one modification. The upstream boundary condition was changed to a stage boundary condition. The value of the stage was determined through a series of model simulations of the open channel configuration (with no initial deposit) such that a 28,000 cfs flow was obtained. It was necessary to use a stage boundary condition so that any additional energy loses due to the presence of the deposits (and the Back Gates) would be reflected in the simulation. The flow of 28,000 cfs was selected to represent flow rates expected when the diversion is first operated at the beginning of each season (corresponding to 450,000 cfs in the MR). The actual initial diversion flow rate has not been determined at this time, and therefore 28,000 cfs was adopted. Details of the modeling analysis conducted for setting the stage boundary conditions are described in **Appendix H.3.**

The selected suspended concentrations selected for the 28,000 cfs flow rate were developed using the same approach as described in Section 8.8.2 and are summarized in Table 8-15.

Table 8-15: Upstream Suspended Sediment Concentrations

Flow (cfs)	Clay (mg/L)	Silt (mg/L)	0.83 mm Sand (mg/L)	0.125 mm Sand (mg/L)	0.250 mm Sand (mg/L)
28,000	31.25	93.75	4.95	5.55	4.5

For each simulation the change in the depositional surface and volume within the channel/ramp area was recorded. An example of the output for Scenario 2 for the "Open C" and "Back Gates" alternatives are shown in Figure 8-25 and 8-26. The entire deposit has not been eroded during the simulation for some of the scenarios, but the rates of volume change are established and can be used as a basis of comparison.

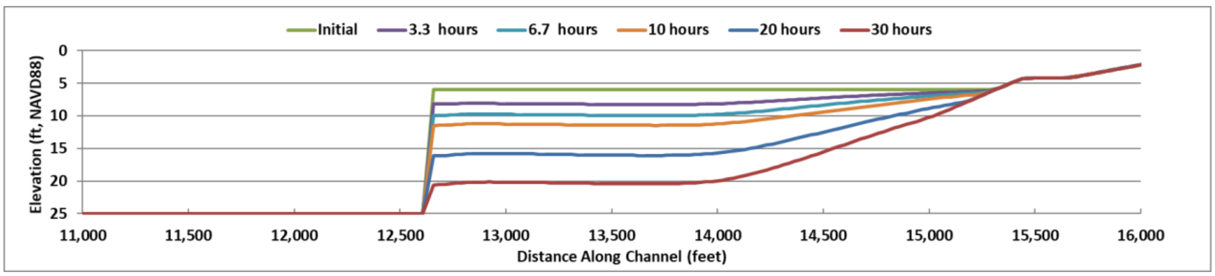


Figure 8-25: Erosion of the Initial Deposit for the "open channel" alternative

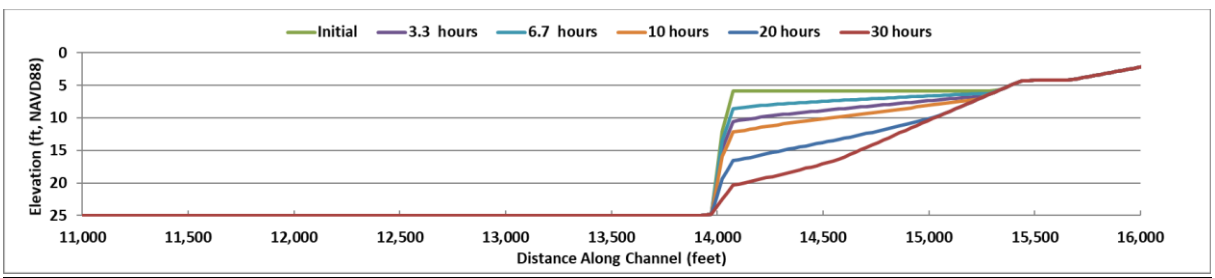


Figure 8-26: Erosion of the Initial Deposit for the "Back Gates" alternative

The deposits generally eroded downward from the upper surface, indicating that the silt deposits were easily eroded into the water column. The results for all of the scenarios listed in Table 8-14 showed similar results. A summary of the time evolution of the deposit volumes is presented in Section 8-16.

Table 8-16: Summary Deposit Volumes

Scenario	Open channel 1	Open channel 2	Open channel 3	Open channel 4	Back Gates 1	Back Gates 2	Back Gates 3	Back Gates 4
Initial Volume (cy)	184,789	624,676	454,048	382,180	5,489	259,775	88,768	19,847
Hours	***Percentage of Deposit Remaining***							
10	15.0	63.5	52.7	9.3	0.0	46.8	32.8	22.5
20	0	31.2	17.1	3.9	0.0	24.9	17.5	9.4
30	0	8.2	0.8	0.7	0.0	5.3	12.2	0.0
40	0	1.4	0.2	0.0	0.0	4.0	7.7	0.0
50	0	0.3	0.0	0.0	0.0	0.5	0.0	0.0

The “Back Gates” alternative has a slightly slower rate of erosion, based on the simulated percent-volume reduction rates. This is due to the reduced flow caused by the deposit blocking a portion of the channel cross-section. The flow for the “Open Channel” option is also reduced, but the combined effect of the deposit and the training walls for the “Back Gates” alternative is more severe, especially for the larger deposit (i.e. Scenarios 2 and 4). However, both alternatives will flush the deposit from the channel and Outfall Area within one to two days.

Additional analysis was conducted using a sand sized sediment for the initial deposit. Deposits consisting of sand will be much less mobile and it is possible the nozzle effect will increase the flushing rate significantly for sand relative to the “Open Channel” alternative, yielding a more favorable result for the “Back Gates” alternative. The simulation was repeated for Scenario 2, and the results indicated that approximately 50% of the initial deposit was removed after 20 days for the open channel alternative and 60% of the deposit was removed for the “Back Gates” alternative. Both results are likely not acceptable, and if a storm induced deposit contains significant amounts of sand sized material, then dredging would be required before the diversion was operated. However, the dredging costs would be lower for the “Back Gate” alternative. The storm deposit would be limited in size by the presence of the back gates, which prevent sediment from depositing in the conveyance channel.

8.7.4 Adaptive Management Opportunities

The three adaptive management opportunities developed with CPRA have been evaluated to determine any potential benefits. The first two potential benefits are solely attributed to the Back Gates alternative as they are not achievable with the open channel alternative.

8.7.4.1 4a: Flow Jetting

The jetting concept consists of closing some of the gate bays, resulting in a higher velocity through the remaining gates. The jet provides increased scouring potential to erode sediment that accumulated during normal operations. For this alternatives analysis, it was assumed that five of the seven bays would be closed.

A Delft3D model was developed and used to evaluate the potential benefits of the jetting. The model used is the same as described in Section 8.7.2. Prior to conducting the sediment transport modeling analysis, a hydrodynamic analysis was conducted to determine the flow reduction due to the gate closing. The closing of five gates will add additional flow resistance and since the diversion flow is gravity driven, the flow through the diversion will be reduced. The analysis indicated that when MR and basin conditions will yield the design flow of 75,000 cfs with all gates open, the flow will be reduced to 32,000 cfs when 5 of the 7 gates were closed. This is an important characteristic of the jetting operations, since the sediment load will be consistent with the 75,000 cfs diversion flow, and likely will not be kept in suspension along the channel when the flow is reduced to 32,000 cfs. Details of the flow reduction analysis are provided in **Appendix H.3**.

Two flow and sediment transport scenarios were used to evaluate the jetting concept and they are summarized in Table 8-17.

Table 8-17: Summary of Transport Modeling Scenarios

Scenario	Gate Configuration	Diversion Flow (cfs)	Sediment Inputs
1	5 Gates closed	32,000	MR sediment concentrations consistent with 1,000,000 cfs
2	5 Gates closed	20,000	MR sediment concentrations consistent with 600,000 cfs

An example plot of the simulated deposition along a transect aligned with the channel and extending into the basin is shown in Figure 8-27.

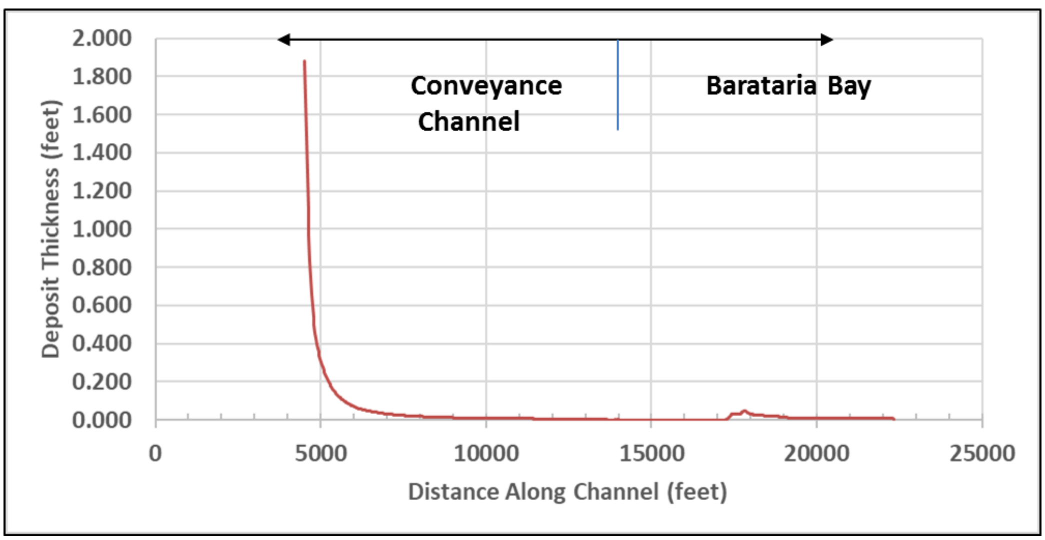


Figure 8-27: Deposits along channel and in the basin for jetting for Scenario 1 (channel Outfall is at 14,000 feet)

The patterns are similar for both scenarios. The most striking result is the large deposition at the upstream end of the Conveyance Channel. This occurs because the partially closed gates restrict the Conveyance Channel flow and velocity. Although the velocity through the gates is accelerated and on the order of 9 fps, it is typically below 2 fps in the Conveyance Channel. The flow entering the diversion through the intake is carrying the same suspended sediment concentration associated with normal operations, consistent with flow speeds on the order of 6 fps in the Conveyance Channel. At the reduced flow and velocity, the sediment transport capacity of the Conveyance Channel is reduced and sediment immediately begins to accumulate in the area of the intake.

At the Outfall the jet is formed yielding higher velocities, and the remaining suspended load is carried into the basin until the flow velocity decreases sufficiently for sediment deposition to occur. Based on the deposit in the basin shown in Figure 8-27, the deposit begins about 4,000 feet into the basin. This is similar to the length into the basin for the case when all gates are open. The accelerated flow due to the jetting is much narrower and despite its higher local velocity, it is spreads out quicker, and subsequently loosing speed, and does not carry sediment further into the basin. The jetting does have more “power” to flush sediment in the region from the Outfall out to about 4,000 feet into the basin. However, based on the analysis in performance Section 8.7.2 (sediment delivery), deposition of sediment in this region is not expected for both the open channel and Back Gate alternatives.

The potential benefits of the jetting can be summarized as providing additional flushing power in the near field region adjacent to the Outfall. However, the results of the sediment delivery analysis (Section 8.7.2) indicate that there will not likely be any deposition in the nearfield region adjacent to the outfall. Thus the potential benefit of extra flushing power is not helpful. Furthermore, it has the potential disadvantage of increasing sedimentation at the beginning of the Conveyance Channel, near the diversion Intake Structure.

8.7.4.2 4b: Diversion flow management during opening and closing gates

The use of the Back Gates to aid in adaptive management has been promoted as a potential benefit of the Back Gate alternative. However, the DT has not been able to develop any clear benefits to adaptive management. It is expected that the flow rates through the diversion can be controlled by the Diversion Gates and there is no additional benefit to using the Back Gates to provide additional control.

There is one potential benefit of the Back Gates while the Diversion Gates are being closed at the end of an operational period. When the MR stage falls below the operational range, the Diversion Gates will be close to prevent flow from the MR into the basin. During the gate closing, which may take up to 10 minutes, the flow speeds in the channel will be reduced (eventually to zero), but the flow will be carrying a sediment load that is consistent with diversion flow speeds. The sediment load during this period will deposit into the channel. A conservative estimate of the depth of the deposit is less than 1mm, (see **Appendix H.3** for the basis of this estimate) and it is expected that this deposition can be flushed by the diversion flow when the diversion is opened during the next operational season.

It is possible that the Back Gates could be used to reduce the volume of water flowing through the channel while the system is being closed. By simultaneously closing the Diversion Gates and the Back Gates, additional flow resistance will be incurred and the total flow going through the conveyance channel will be reduced. Subsequently, the volume of deposited sediment will be reduced. However, since the expected deposit thickness is on the order of 0.1 mm or less, the benefit is not significant.

8.7.4.3 4c: Radial gate configuration

The radial gate concept has been developed to change the general orientation of the diversion. The general premise is that the distribution of sediment into the Barataria Basin can be enhanced if the direction of the diversion flow emanating from the Outfall is periodically changed. For instance, the flow may initially be directed westward, consistent with the current design, but then changed to a southwesterly direction, and eventually a northwesterly direction. This redirection of the flow and sediment may widen the area impacted by the diversion, spreading the land building over a larger area.
A conceptual design of such a system capable of redirecting the flow, as well as maintaining the multiple gate complex for each orientation is shown in Figure 8-28.

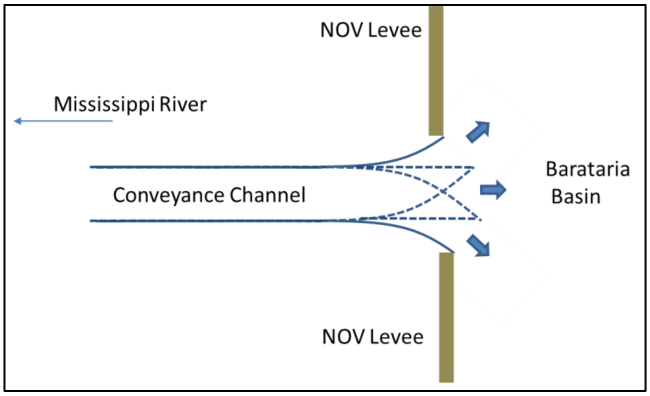


Figure 8-28: Concept Drawing for Radial Gate System

The mechanism needed to guide the flow through one of the three Outfalls shown in Figure 8-28 is not considered here. The design of such a mechanism is not a simple task, as the hydraulic efficiency of an adaptable system must remain high to maintain the design flows. Furthermore the cost of this type of system is expected to be relatively high. The 7-bay gate system that has been used in analysis in this section is expected to cost over $300 million.

Another alternative is to retain the single Outfall design and use channel dredging and possibly temporary walls to redirect the flow after it exits the Outfall. A concept drawing is shown in Figure 8-29 for diverting the flow in one direction.

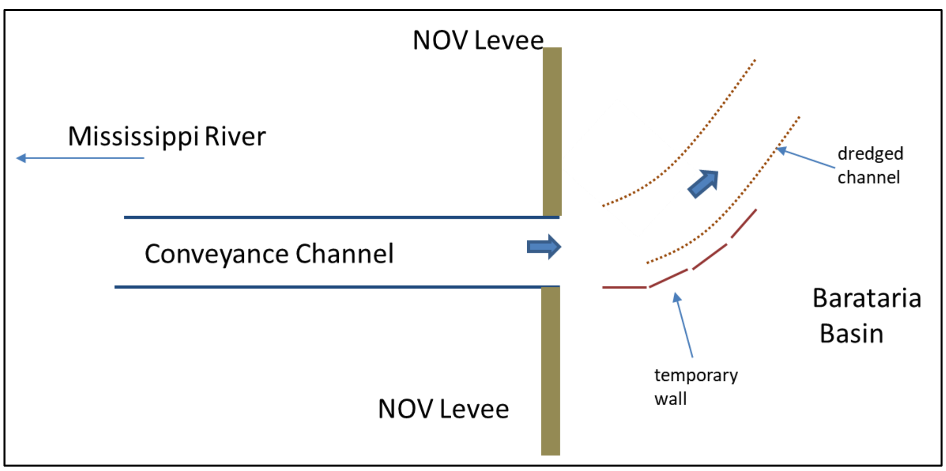


Figure 8-29: Concept Drawing for Flow Diverting Approach

The premise that reorienting the diversion flow to the north or south will improve land building over the Barataria Basin (relative to not changing the orientation) has not yet been demonstrated. The diversion flow analysis conducted for this and other alternatives analysis indicate that most of the momentum associated with the diversion flow decays within approximately a mile of the Outfall, at which point the diversion water moves with the ambient wind and tide driven flow. As the Barataria Basin is much larger, extending up to 20 miles from the Outfall, it is not clear if the flow reorienting impacts confined to one mile from the Outfall will have impacts over the entire basin.

Therefore, no additional analysis has been applied to the radial gate alternative as an adaptive management benefit. The concept does have merit, but the potential beneficial impacts on improved sediment dispersion and increased land building should be demonstrated before further consideration is given to the concept. The land building analysis is beyond the existing scope but it is recommended that it be pursued during subsequent design work.

8.8 Outfall Transition Feature Numerical Modeling

An alternatives analysis of the Outfall Transition Feature has been conducted to guide the selection of the final transition design. The primary function of the Outfall Transition Feature is to provide a gradual transition from the Conveyance Channel to the basin. The invert of the Conveyance Channel is on the order of EL -25 and the basin elevation near the Outfall is on the order of EL -4. The Outfall Transition Feature is intended to be a temporary component of the design. The results of the TWIG's Basin Wide Model and Outfall Management Models indicate that a channel will be eroded through the area of the Outfall Transition Feature and further into the basin. Thus, the role of the Outfall ramp is to provide an initial transition during the first few years of operation, until a channel is eroded.

The basic configuration of the Outfall Transitions Feature is shown in Figures 8-30 and 8-31. The Outfall Transition Feature begins at the end of the Outfall where the channel bottom width is approximately 300 feet wide and at EL -25. The feature will slope upwards as it extends into the basin, until it reaches the nominal basin elevation of EL -4. The ramp will expand laterally (defined by the half-flare angle) and at the lateral edges, it will also slope upwards to the basin elevation of EL -4.

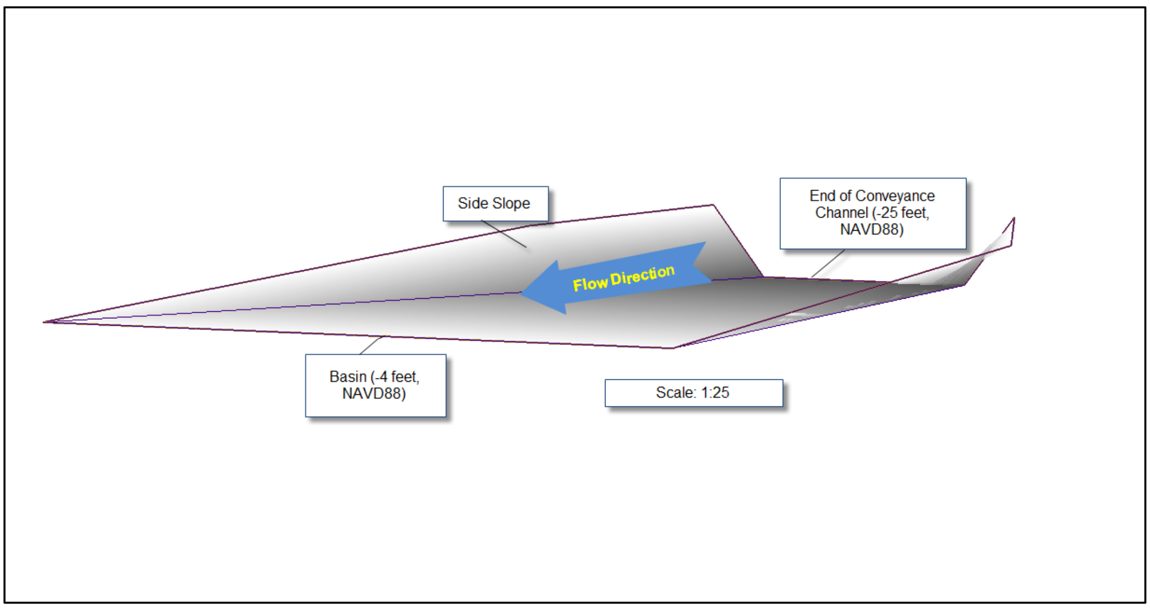


Figure 8-30: Three-dimensional view of the Outfall Transition Feature

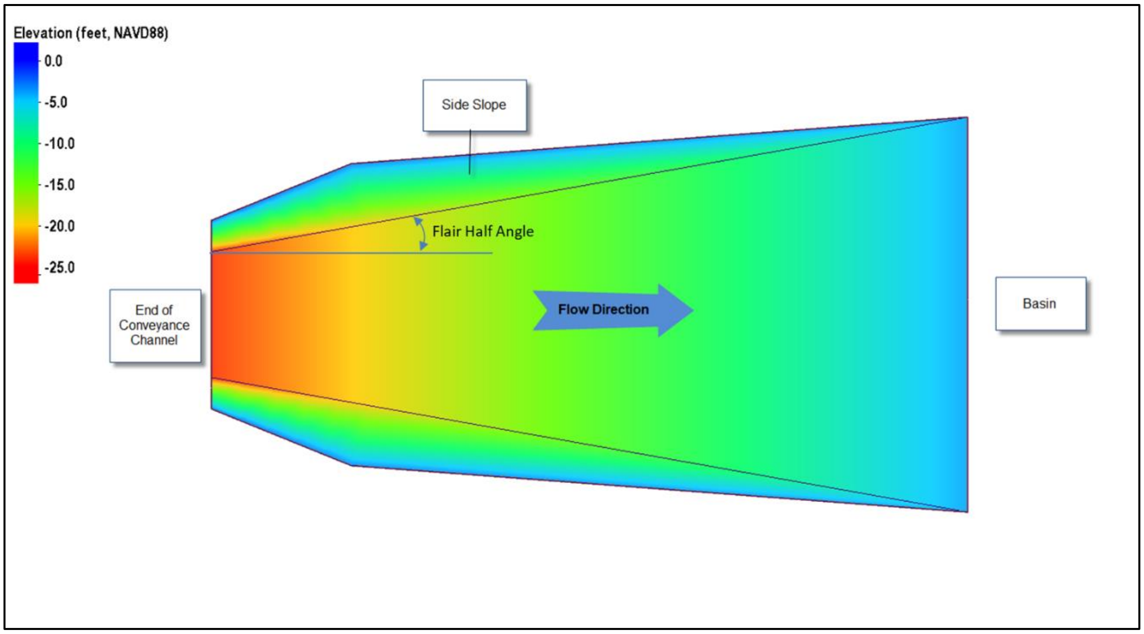


Figure 8-31: Map view of the Outfall Transition Feature Configuration

A Delft3D model was configured to evaluate the alternatives. The portion of the model domain encompassing the Conveyance Channel, the Outfall Transition Feature and the near field part of the Barataria Basin is shown in Figure 8-32.

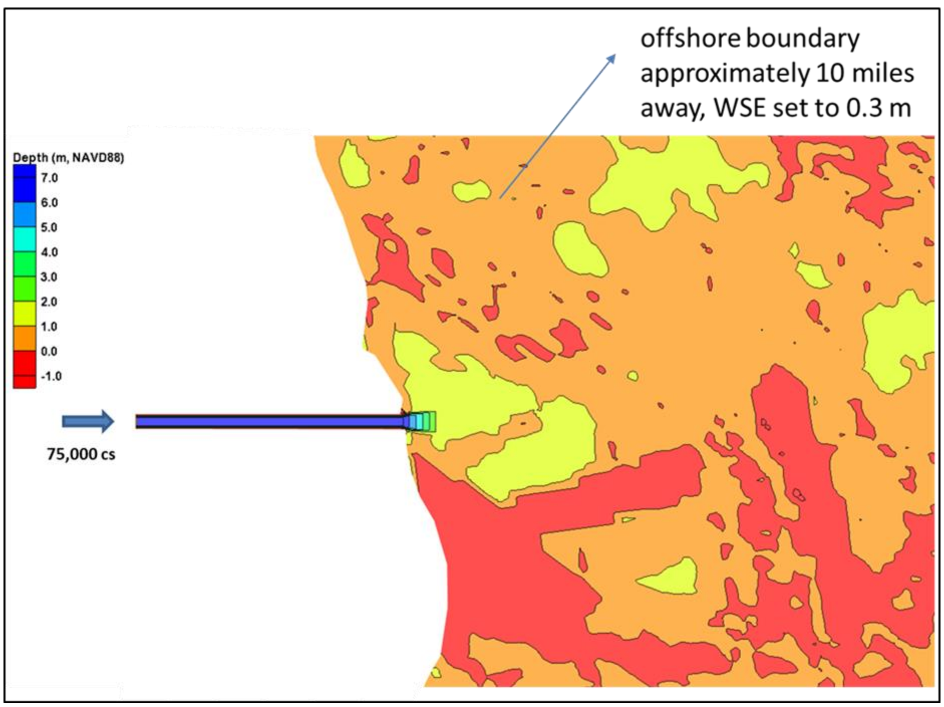


Figure 8-32: Delft3D Model Domain and Bathymetry for Outfall Transition Feature Analysis

The primary metrics in the evaluation of the alternatives are:

a) the hydraulic head loss, and
b) the total volume of material that will need to be dredged to form the feature.

The alternatives analysis was conducted in two phases. In the first phase, four ramp lengths (1,000, 2,000, 3,000, and 4,000 feet) and three flare half angles (10, 15, and 20 degrees) were considered.

The water elevation along the centerline of the Conveyance Channel and ramp was extracted for each alternative ramp configuration. A plot of the results for the 4 lengths using a flare half angle of 10 degrees is shown in Figure 8-33.

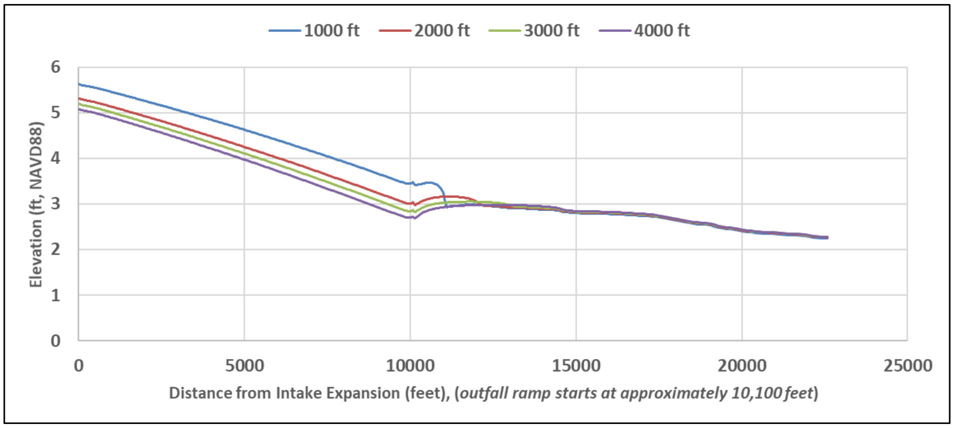


Figure 8-33: Water Elevation Profiles for Ramp Alternatives (10 degree half angle)

The results for all the alternatives are summarized in Table 8-18, which show the upstream stage elevations for each configuration. The results show very little sensitivity to the flare half-angle with the differences in stage on the order of 0.09 feet or less. The upstream stage decreases as the ramp length increases for all flare angles, with diminishing impacts as the ramp is lengthened.

Table 8-18: Summary of Ramp Upstream Stages

Angle/ Distance	10 (deg)	15 (deg)	20 (deg)
1000 (ft)	5.63	5.58	5.54
2000 (ft)	5.31	5.27	5.23
3000 (ft)	5.19	5.14	5.11
4000 (ft)	5.08	5.05	5.05

Subsequently, additional evaluation of a 500, 1500, and 5,000-foot Outfall Transition Feature was completed using the 10-degree flare half-angle to provide more resolution on the variation of the head loss with the transition feature length.

The results of these additional analysis and the original analysis (with a 10 degree flare half angle) are summarized in Table 8-19. A tailwater stage was selected at about 14,500 feet along the transect, which represents a point where all the stage profiles have the same value (within 2%). The tailwater EL there is 2.84 and was used to quantify the stage differences (i.e. head losses) for the different ramp configurations.

Table 8-19: Summary of Stage Impacts

Ramp Length (feet)	Upstream Stage (ft, NAVD88)	Tailwater Stage (ft, NAVD88)	Head Loss* (ft, NAVD88)	Relative Difference** (feet)
500 ft	6.13	2.84	3.30	1.06
1000 ft	5.63	2.84	2.79	0.55
1500 ft	5.42	2.84	2.58	0.34
2000 ft	5.31	2.84	2.48	0.23
3000 ft	5.19	2.84	2.36	0.11
4000 ft	5.08	2.84	2.24	0.00
5000 ft	5.08	2.84	2.24	0.00

*Head Loss does not include velocity (difference in stage only)
**Compared to Head Loss for the 5000-foot ramp length

A graphical representation of the relative differences is shown in Figure 8-34.

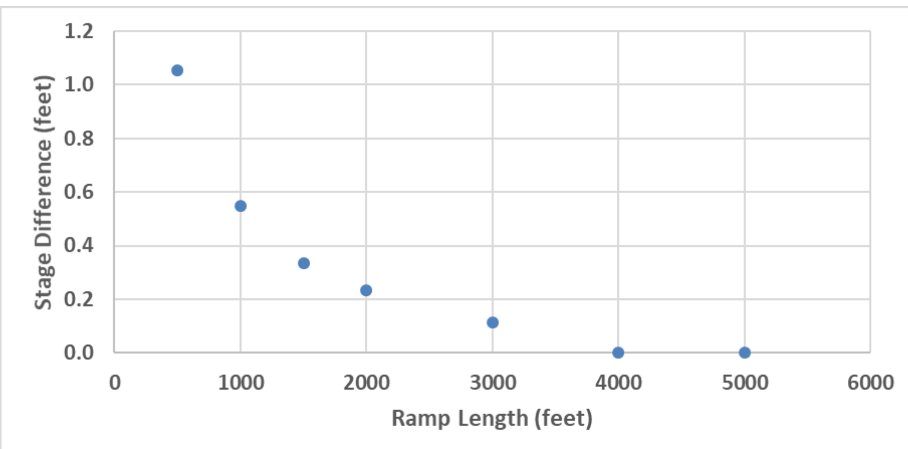


Figure 8-34: Difference in Head Loss (compared to 5000-foot ramp length)

The footprint of each ramp alternative and the dredge volume required to construct each ramp configuration are provided in Table 8-20.

Table 8-20: Summary of Head Loss and Dredging Requirements

Length (ft)	Flare Half-Angle (deg)	Relative Difference (feet)	Footprint Area (ft^2)	Dredge Volume (cy)
500	10	1.06	321,000	89,900
1000	10	0.55	679,000	195,300
1500	10	0.34	1,163,000	335,100
2000	10	0.23	1,693,000	483,300
3000	10	0.11	3,096,000	866,900
4000	10	0.00	4,826,000	1,329,900
5000	10	0.00	6,891,000	1,874,700

The DT recommends that the 1,500-foot-long Outfall Transition Feature be selected as the preferred alternative. Numerical modeling results indicate that the 1,500-foot-long ramp will produce an energy loss of four inches. Reducing the energy loss to where it approaches zero requires extending the ramp 2,500 feet, which requires dredging an additional 994,800 cubic yards of in-situ soils. The DT estimates the unit cost to be $15/cubic yard for Outfall dredging (see **Appendix O**), inclusive of piping it to the designated fill area near Bayou Dupont. This equates to $14,992,000 of additional construction cost. The four-inch energy loss is a consideration for the initial period of the diversion's operational life because that is when the Outfall Ramp's constructed geometry affects the distribution of the sediment-laden discharge flows into the Basin. During the initial operational period, a four-inch energy loss will not impede distribution. The ramp's geometry will evolve as the discharge flows erode in-situ material and deposit conveyed sediments, and it is this evolved geometry that will affect sediment distribution into the Basin during the remainder of the diversion's operational period. Numerical modeling to be performed during the 30% Phase will model the evolution of the Outfall Ramp's geometry, and the effectiveness of the 1,500-foot-long ramp design will be evaluated as part of that effort. This will be documented in the BODR Update. In any case, with monitoring and adaptive management, the ramp area can be dredged later to promote conveyance and distribution, if monitoring determines this necessary. For all these reasons, the DT recommends the 1,500-foot-long Outfall Transition Feature Alternative as the basis for the Outfall Transition Feature's detailed design.

8.9 Performance of the Current Conditions Diversion System under Future Conditions

The basin-side water surface elevation is expected to rise in the future due to a combined effect of the Relative Sea Level Rise (RSLR) (TWIG-SLR-Memo, 2018) and land building as a direct result of sediment delivery to the basin during the diversion life-cycle. TWIG's Basin Wide and the OMBA land-building models show that a typical deltaic system with well-defined channels develops in the vicinity of the Outfall. This results in a significant impact on the tailwater and reduces the diversion capacity. Further, TWIG's PR15 basin-wide model (Meselhe et al., 2015) used an internal boundary connection between the river and the diversion intake (defined as a fixed mathematical relation that prescribes the diversion discharge as a proportion of the river discharge), which disregarded the actual head difference available in order to draw the prescribed diversion flow. Therefore, the DT developed a Delft3D model (FTNOMBA) consisting of the intake, the Conveyance Channel, the Outfall region and the Barataria Basin up to the Gulf of Mexico. The model bathymetry was developed using land-building predicted at year 49 by the TWIG's PR15m 1.5m SLR Basin Wide Model.

To simulate the one-year hydrograph run, the 49th year MR hydrograph would have been required. However, the 49th year MR hydrograph does not reach the required design condition in the MR of 1,000,000 cfs. Therefore, the 44th year MR hydrograph (Figure 8-35) was used for this one-year simulation. The peak discharge is seen to reach over 1,200,000 cfs. The southern Gulf of Mexico boundary was kept at a constant water surface elevation (WL) of approximately 1 foot corresponding to Mean Tidal WL data from TWIG's Basin Wide Model at Port Fourchon. Figure 8-35 (in red) also shows the diverted discharge hydrograph. As seen from the figure the diverted discharge does not reach 75,000 cfs at 1,000,000 cfs in the MR under future conditions if the current conditions diversion system design is used. Figure 8-36 shows the discharge rating curve between the MR and the Outfall discharge and indicates that the currently designed system can convey only approximately 55,000 cfs under future conditions. A redesign of the sizing of the intake and/or Conveyance Channel is necessary to meet the

75,000 cfs target flow. Figure 8-37 shows the basin-wide water surface elevations at time when MR reaches 1,000,000 cfs during the one-year simulation.

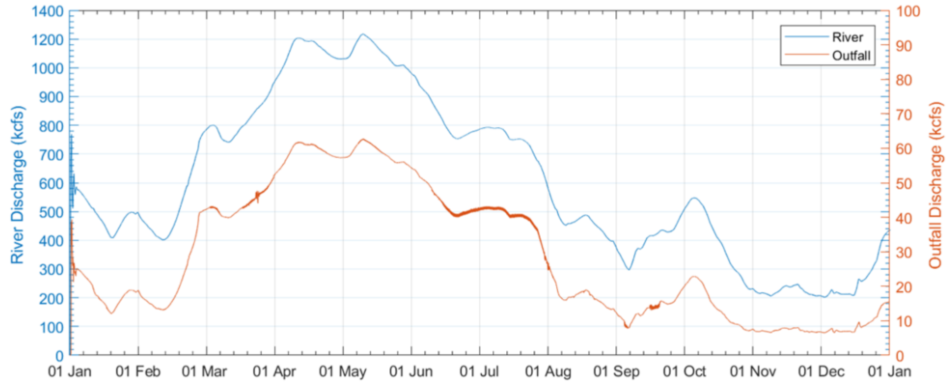


Figure 8-35: MR discharge hydrograph for future condition (44th year conditions is shown here as the 50th year hydrograph did not reach design condition of 1,000,000 cfs in MR) from TWIG PR15 1.5m SLR model (blue) on left axis and corresponding discharge at the diversion outfall (red) is plotted on the right axis

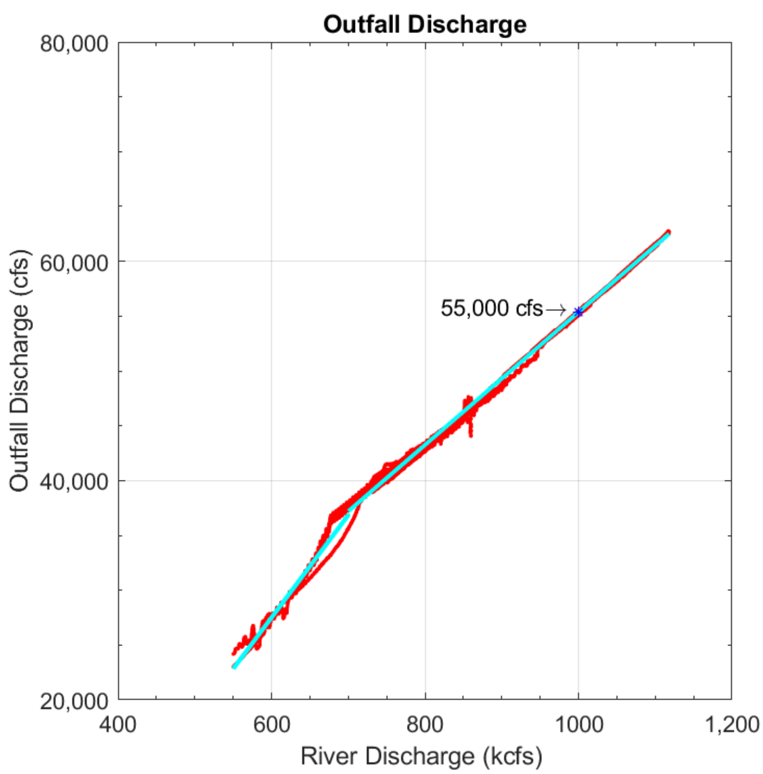


Figure 8-36: MR Discharge vs Outfall Discharge under future conditions with curent diversion system size. At 1,000,000 cfs in the MR river the current design allows only approximately 55,000 cfs discharge at the outfall under future conditions.

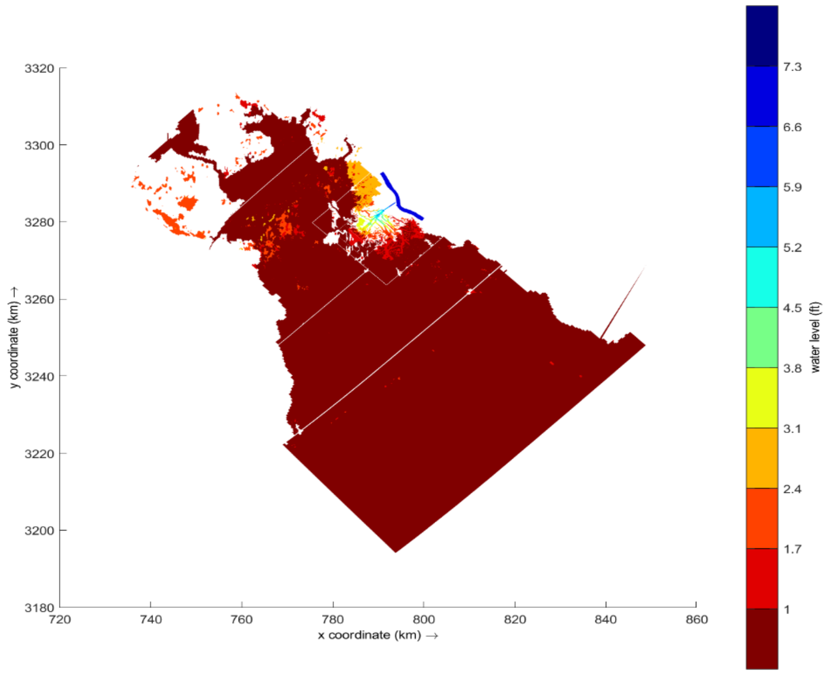


Figure 8-37: FTNOMBA Model simulated Water surface elevations when MR reaches 1,000,000 cfs.

Table 8-21: Sensitivity of diversion discharge at the Outfall to the Conveyance Channel roughness, Manning's n .

Manning's roughness coefficient (n) for Conveyance Channel	River Discharge (cfs)	Diversion Discharge at Outfall (cfs)
0.024 (unlined earth channel)	1,000,000	65,500
0.035 (rip-rapped channel)	1,000,000	55,800
0.050 (natural, vegetated, channel with pools)	1,000,000	48,000

Modeling simulations showed that, for a given system dimension, the diversion system discharge capacity depends on the the choice of Manning's roughness. A limited sensitivity analysis using three steady state runs at 1,000,000 cfs MR discharge and with the currently designed three-component system (combining the headworks, the conveyance channel and the outfall transition) showed (Table 8-21) that considerable diversion discharge variation is possible based on the type of channel lining or the level maintenance. In order to accurately estimate the three-component diversion losses, the FLOW-3D model should be used to model the complex 3D turbulence, frictional and contraction/expansion losses through this complex system. The development of FLOW-3D modeling of the three-component system is currently underway as a next step to determine the required increase in the intake conveyance and calibrate the Delft3D model for further tasks for this project.

8.10 Physical Modeling

The numeric modeling program is also supported with a physical modeling program. A physical model plan was developed for a 1:65 scale model. The physical modeling will investigate the effectiveness of the diversion and the performance of the Conveyance Channel. Two physical models will be constructed: One model that includes about 12,500 feet of the Mississippi River and the diversion. A second model will include about 7,000 feet of the Conveyance Channel and about 2,000 feet of the basin to determine the sediment transport characteristics in the channel and near field deposition in the basin. Figure 8-38 shows the physical model domain.

Figure 8-38: Mississippi River Physical Model Domain

The physical model is a live bed model that includes sediment and captures both bedload and suspended load transport processes. A detailed discussion of the physical model and the physical model scaling is presented in **Appendix H.6**.

Part of the model scaling, was confirmation that the selected model sediment will move as bedload and suspended load. A small flume test was conducted to determine the incipient motion characteristics of the sediment and confirm that it will move in suspension. A 20 feet long by 2 feet wide flume was used for the test. The center 10 feet of the flume was filled with about 2 inches of sediment. Upstream and downstream of the sediment, a false floor was installed making the floor of the flume and the top of the sediment level. Figure 8-39 shows the model flume with sediment. The flume was tested at three water depths and three velocities. The flume depths correspond to prototype water depths of 19.5, 39 and 78 feet and the water velocities corresponded to prototype velocities of 1, 3, and 5 ft/s. Prototype velocities range from about 1 ft/s to 7 ft/s depending on the river flow and location in the cross section. Mid channel velocities are 4.5 to 7 ft/s. Appendix H.6 (Figure 5-4) shows a plot of depth averaged water velocity as a function of distance from left bank for flows of 712,000 cfs and 959,000 cfs.

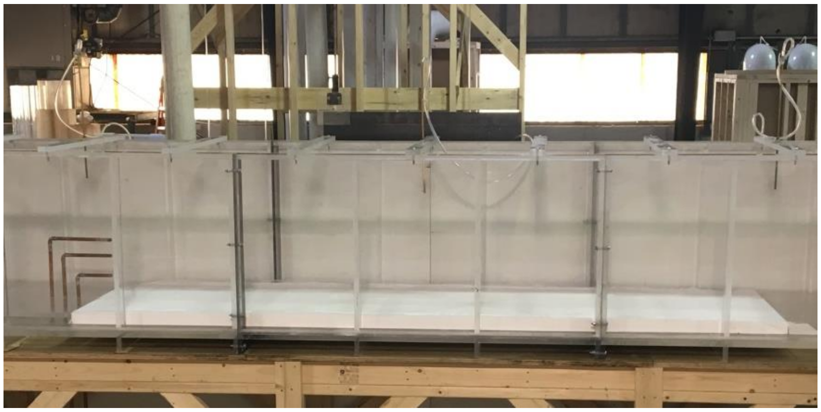


Figure 8-39: Sediment Test Flume

During each test, isokinetic samples were collected at three depths at the downstream end of the live bed. The data shows distinct sediment concentration profiles at higher velocities with near bed concentrations near 1000 mg/l. Figure 8-40 shows the measured sediment concentration profiles from the flume test. In addition to the measured sediment concentration profiles, photographs during testing showed the formation of bedforms. The geometric similarity of the flume bedforms to the Mississippi River bedforms is not yet known, however, it is known that bedforms exist in the MR for some of the flows tested. Figure 8-41 shows the bed forms observed at a prototype velocity of 5 ft/s and a prototype depth of about 19.5 feet.

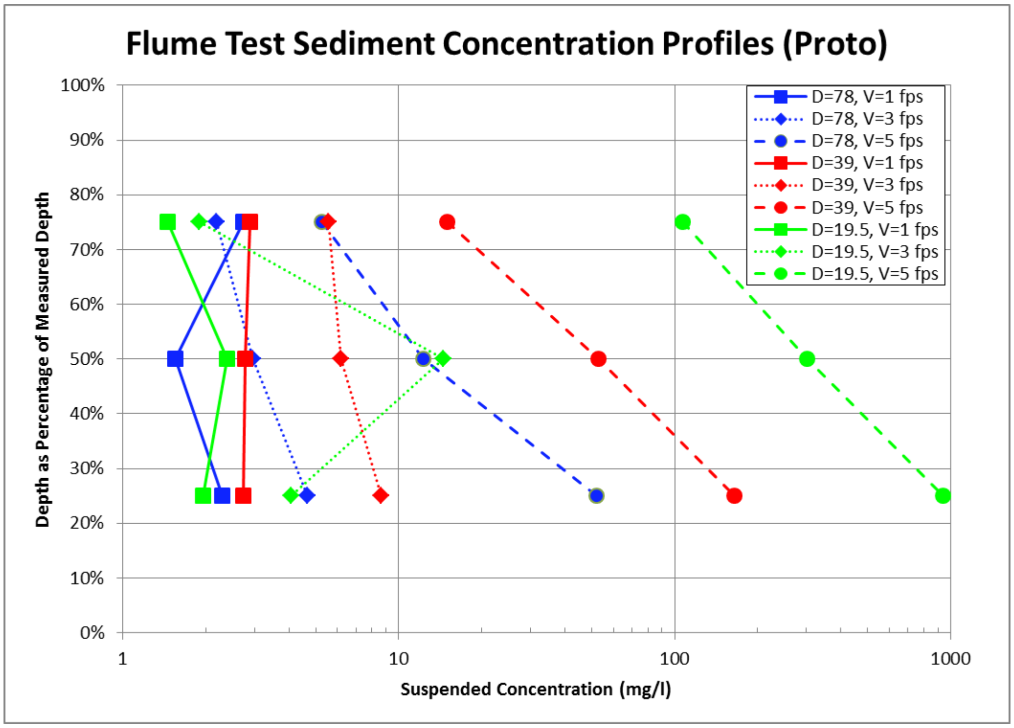


Figure 8-40: Measured Sediment Concentration Profiles in Small Flume

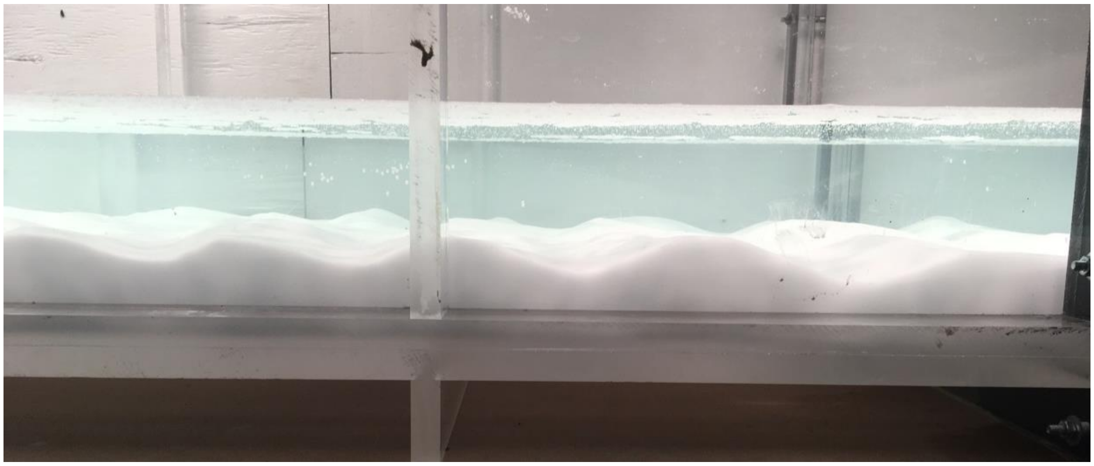


Figure 8-41: Bedforms Observed in the Small Flume Test

The observed bedforms were a plain bed (no bedforms) for a velocity of 1 ft/s at all depths. At a velocity of 3 ft/s ripples were observed and at a velocity of 5 ft/s dunes were observed. The largest dunes were observed for the lowest water depth.

The Rouse number is a dimensionless number that describes the uniformity and shape of the vertical sediment concentration profile model report. The Rouse number was estimated for the Mississippi River at the diversion site and compared with the Rouse number computed for the flume based on the sediment characteristics of the sediment that is planned for use. Both the prototype and the model have a Rouse number near one for flows of about 1,000,000 cfs. A detailed description is provided in **Appendix H**.

A detailed report on the model scaling and the flume test results is included in **Appendix H**.

8.11 Interior Drainage and Siphon Design

This subsection documents the work completed to date and the work planned for development of the BOD for Interior Drainage improvements. The key design element of the Interior Drainage improvements is the inverted Siphon, the conduit that will connect the drainage area bisected by the sediment diversion channel.

8.11.1. Data Gap Analysis

The DT's first step in developing the BOD for Interior Drainage was to take inventory of the available information, and to identify data gaps. These identified data gaps were documented in the January 30, 2018 Data Gap Analysis Report. Though some of these data gaps have been resolved, the following gaps remain:

8.11.1.1 Topographic Surveys

The DT is in the process of performing topographic surveys. Once complete, these surveys will allow the DT to update the existing HEC-HMS and HEC-RAS models to reflect current conditions and to confirm existing drainage features.

8.11.1.2 Access to Wilkinson Pump Station

Though not a specific data gap, the DT has not been authorized access to the Wilkinson Pump Station nor allowed coordination with the station's operations staff. Once access and coordination are granted, the DT will calibrate the existing conditions model utilizing data gathered on site, including but not limited to, pump start/stop times, run times and suction bay elevation readings. The time sequence of this data would ideally begin several hours prior to a significant rainfall event and continue to a time after the event when pumping ends. The calibrated existing conditions model will be used as the foundation of the existing conditions models and be further developed to reflect the improved conditions for each of the alternatives.

8.11.1.3 Wilkinson Pump Station Construction Drawings and Operation Data

As the installation of an inverted Siphon will increase the system headwater elevation, the DT must evaluate the extent to which tailwater can be lowered at the Wilkinson Pump Station. However, the DT does not have as-built drawings of the Wilkinson Pump Station, nor operation and maintenance data reflecting the standard operation procedures. Once provided, this information will inform the DT the degree to which operating levels of the pump station can be lowered through operational modifications, and the cost of structural and equipment changes necessary to further lower operating levels.

8.11.2. Design Criteria

The criteria dictating the design of the Interior Drainage improvements, given in **Appendix I**, establish the design basis for the surface drainage features, the hydraulic design of the inverted Siphon, and the design of the Inverted Siphon's inlet and outlet structures. The design criteria also establish the applicable USACE, State of Louisiana and Plaquemines Parish codes and standards that apply to the design of the Interior Drainage improvements.

8.11.3. Design Assumptions

The evaluation of hydraulic conditions and the subsequent design of hydraulic features to maintain Interior Drainage upon construction of the diversion channel will be based on a level of service consistent with a 25-Year, 24-Hr storm event. The anticipated longevity and estimated net present value of the operation and maintenance of new drainage features will be based on a useful life of 50 years.

It is also assumed that a zero net increase of water levels in the existing drainage system upstream of the inverted siphon must be maintained and that any additional head imparted into the drainage system by the installation of an Inverted Siphon can be operationally mitigated at the Wilkinson Pump Station by lowering the suction bay water surface elevation by an amount equal to the head added by the Inverted Siphon along with some minor mechanical modifications at the station. These modifications are assumed to include i) replacement/re-trimming of pump impellers, ii) removal and replacement of existing 800 hp diesel drives with new 900 hp drives, iii) removal and replacement of or modifications to the existing right angle gears to accommodate the new drives and iv) modifications to existing fuel oil, compressed air, cooling and other process piping and control wiring as well as reprogramming of controls as needed.

8.11.4. Existing Conditions Drainage Model

The DT has reviewed and will update the existing HEC-HMS model to determine storm quantities and the existing HEC-RAS model to determine water surface elevations, both under the 25-Year, 24-Hr storm event. The DT has not yet updated the models as topographic surveys of the area are not yet complete and data describing the Wilkinson Pump Station's operation are not yet available. The DT will initiate updating the models once the survey and operations data are available.

The HEC-HMS model update will verify the digital elevation model describing the drainage basin, as well as the sub-basin delineations and characterizations. The updated unsteady state HEC-RAS model will incorporate the operation of the Wilkinson Drainage Pump Station, and flows generated by the updated HEC-HMS model. Using the updated data, the DT will validate the sub-basin discharge quantities and identify the Outfall locations of each. To calibrate the models, the DT will monitor the channel elevation at its most downstream point, the Wilkinson Pump Station, and correlate flow rates through the channel with station's pumping performance. With the calibration of the models, the DT will develop a post-model map and compile the results to reflect the current performance of the system.

A detailed description of the procedures for updating and calibrating the existing conditions models will be provided in **Appendix I** of the updated BODR which will be included as part of the 30% phase of work.

8.11.5. Siphon Conceptual Sizing

As previously stated, the DT has not been allowed access to the Wilkinson Pump Station, and has been unable to calibrate the existing conditions drainage basin model. Therefore, the DT has not been able to independently establish the required capacity of either a pump station alternative or inverted siphon alternative at the time of this submittal. In an effort to establish a valid comparison between the Inverted Siphon and the pump station alternatives with the limited information currently available, it was decided to compare a pump station sized to 740 cfs as presented in the 2014 Base Design, with an Inverted Siphon of the same capacity. Further, a statistical analysis of five-years of rainfall data collected from the Belle Chasse Naval Air Station north of the project site reveals a low-flow condition of 35 cfs at the new Siphon structure. Considering a minimum Siphon velocity of 2.5 fps (to mitigate sedimentation) and the low flow condition the DT preliminarily recommends a combination of three 48-inch and five 60-inch Siphon tubes. This configuration includes a single redundant tube of each diameter. Maintenance of the minimum flow velocity within the inverted siphon tubes during pumping events will be accomplished through the design of a stepped weir system that will only allow flow into a predetermined number and combination of tubes depending on the amount of flow, and therefore, the elevation within the channel. Mechanical equipment requiring operation during a storm event to maintain minimum velocities is not a consideration for the design. Final sizing of the Siphon can only be completed once the HEC-HMS and RAS models are checked in detail, adjusted as needed and calibrated. A detailed description of the Siphon sizing effort is given in **Appendix I**.

8.11.6. Alternate Intake Design for the Inverted Siphon

As a potential measure to reduce the amount of head introduced by the drainage siphon, the DT will also consider, as part of the 30% design, an alternative siphon intake bay design. The concept driving the alternative intake design is to utilize the nearly unlimited amount of water available in the diversion channel for periodic siphon cleaning in lieu of achieving scour velocities for cleaning during normal operation.

Having such a reliable water supply available may give the DT the ability to increase the size of the siphon tubes, thereby lowering design flow velocities to a level below the recommended normal operating siphon velocities, which were developed by the relevant Agencies to achieve scour velocity.

This would result in a significant decrease in the head losses through the siphon during normal operations, which would in turn reduce the amount that the tailwater would have to be lowered at the Wilkinson Pump Station to mitigate for the head introduced by the siphon.

The physical concept of the alternate siphon intake bay includes construction of the bay in close proximity to the diversion channel levee with the ability to be isolated from the upstream drainage channel. This would be accomplished through the installation of a control gate structure at the upstream portion of the bay. Isolation gates would also be installed on each of the siphon tubes themselves. The concrete walls and/or earthen berm defining the intake basin walls will be built to an elevation at or near the elevation of the top of the diversion levee. A pump or true siphon will be installed between the diversion and siphon intake bay. Periodically, as defined in an operation and maintenance protocol to be developed by the DT, or as required due to current conditions, the intake bay isolation gate will be closed along with the siphon tube gates. The intake bay would then be filled with water from the diversion channel via the pump or true siphon. Once the intake bay is filled to a predetermined elevation, related to velocity requirements within the drainage siphon tubes to promote scouring, the drainage siphon tube gates would be opened, allowing for flow equal to or above the scour velocities required for cleaning. This process could be repeated as many times as necessary to complete siphon cleaning by virtue of the water available in the diversion. This would also require coordination with the operation of the Wilkinson Pump Station to pump out the water used for cleaning of the siphons.

8.11.7. Modeling of Proposed Alternatives

Once the exiting conditions models have been updated and calibrated, the DT will evaluate the alternative configurations by which an inverted Siphon system can maintain drainage in conjunction with the sediment diversion channel. The models will consider the drainage basins yielded by two candidate NOV Levee alignments: along the existing back levee alignment and along Timber Canal further inland. A detailed description of the procedure for developing these models of proposed alternatives will be provided in the Site Drainage Report Outline in **Appendix I** of the updated BODR, which will be included in the 30% phase of work.

9. GEOTECHNICAL EXPLORATION AND ENGINEERING

9.1 General

The DT performed the geotechnical engineering for the project's permanent structures. Temporary structures will be designed by the CMAR utilizing the exploration data developed for the project and the DT's preliminary analyses from the 15% design effort. The CMAR will also require additional geotechnical data (e.g., pump test). The DT will provide technical review of the CMAR's design efforts where appropriate.

The DT retained subject matter experts for seismic faulting evaluations, Outfall erodibility, and independent technical review. Dr. George Filz has been designated a subject matter expert and will provide consultation on settlement induced bending moment on pile supported features, and deep mixing method (DMM), and soil structure interaction numerical modeling. Subject matter experts are in the process of being retained for consult on Outfall channel erodibility and seismic considerations (Risk of Faulting). Outfall erodibility will be addressed by Dr. Kehui Xu, Associate Professor, Louisiana State University, as the subject matter expert. His work will be primarily reviewed by Dr. Nina Stark, Assistant Professor AY, Virginia Polytechnic University. A seismic subject matter expert is currently being identified among several experts.

9.2 Design Approach

Design standards outlined in the Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG), Interim, Revisions through June 2012 and Louisiana Flood Protection Design Guidelines (LFPDG), Geotechnical Section Version 1.0 are the standards referenced for geotechnical design of flood protection. The "LADOTD Bridge Design and Evaluation Manual" (which defers to the AASHTO LRFD Bridge Design Specifications) will be the design standard for the Hwy 23 Bridge. Refer to the Project DCD (**Appendix U**) for detailed discussion of all geotechnical criteria.

9.3 References and Publications

- AASHTO LRFD Bridge Design Specifications, 8th Edition.
- Cofferdams, 2nd Edition, Columbia University Press, New York, L. White, and E. A. Prentis, 1950.
- Comments to the U.S. Army Corps of Engineers, New Orleans Division in reference to The Environmental Impact Statement for the Mid-Barataria Sediment Diversion, McClindon, C. et al., September 2017.
- Dewatering Cofferdam for Construction of Olmsted Locks, Journal of Geotechnical and Environmental Engineering, ASCE, Volume 126, No. 6, June 2002.
- EM 1110-2-1913, Design and Construction of Levees, USACE, April 2000.
- EM-1110-2-2503, Design of Sheet pile Cellular Structures, Cofferdams, and Retaining Structures, September 1989, USACE.
- Fine-Grained Alluvial Deposits and Their Effects on Mississippi River Activity, War Department, Corps of Engineers, Mississippi River Commission, Vicksburg Mississippi, Harold Fisk, July 1947.
- Finite Element Analysis of Lock and Dam 26 Cofferdam, Journal of Geotechnical and Environmental Engineering, ASCE, Volume 111, No. 4, G.W. Clough, Thangavelu Kuppusamy, April 1985.

- Geological Investigation, Mississippi River Deltaic Plain, Distribution of Deltaic and Marine Deposits, Quadrangle, Pointe A La Hache, U.S. Army Corps of Engineers, 1987.
- Geotechnical Data Report for 30% Design, Mid Barataria Diversion Project (BA-153), Plaquemines Parish, La., January 24, 2014.
- Hurricane and Storm Damage Risk Reduction System Design Guidelines, Revised June 2012, including references.
- Louisiana Department of Transportation and Development, Bridge Design and Evaluation Manual, including AASHTO Standards.
- Louisiana Flood Protection Design Guidelines Version 1.0, July 16, 2015, including references.
- Mid-Barataria Sediment Diversion, Geotechnical Report, HDR Engineering, Inc., July 2014.
- TM 3-424, Investigation of Underseepage and Its Control, Lower Mississippi River Levees, Volume 1, Waterways Experiment Station, USACE, 1956.
- TM 5-818-5, CI, Dewatering and Groundwater Control, Department of the Army, the Navy, and the Air Force, 27 June 1985.
- TO15/16: Sediment Erodibility in Outfall Areas of Barataria and Breton Sound, Geotechnical Field Investigation, The Water Institute of the Gulf, June 2016.
- U.S. Army Engineer Waterways Experiment Station, CE, Distribution of Soils Bordering the Mississippi River From Donaldsonville to Head of Passes, C.R. Kolb, TR No. 3-601, June 1962.
- U.S. Army Engineer Waterways Experiment Station, CE, Geology of the Mississippi River Deltaic Plain, Southeastern, Louisiana, C.R. Kolb and J.R. Van Lopik, TR No. 3-483, Volume 1, 1958.

9.4 Available Investigation Summary

9.4.1 Available Boring and In-Situ Data

Available geotechnical data include data gathered from 11 borings made by the USACE. Three borings were 5-inch diameter fixed piston sample borings made to depths between 50 and 82 feet. The remaining eight borings were 3-inch diameter general type borings sampled with a Shelby tube sampler. The 3-inch diameter borings vary in depths between 27 and 149 feet. Three of these borings were made along the alignment of the MRL. GeoEngineers, in association with HDR Engineering, completed the preliminary geotechnical exploration. This preliminary geotechnical exploration supplemented the USACE borings with 19 borings obtaining 5-inch diameter fixed piston sampler borings, five individual 3-in. diameter general type Shelby tube sample borings made in the Mississippi River, 20 cone penetration tests (CPTs), and four field vane (FV) tests made adjacent to the 3-in. diameter Shelby tube borings. In addition, two pump tests (PTs) were conducted and monitored with 16 piezometers. The PTs were also sampled with 3-in. diameter borings. One PT was performed near the NOV Levee within predominantly fine-grained deposits and is not relevant for the BOD. The other PT was performed in shallow silt deposits, through which the excavation for the intake gate at the HW will be completed landward of the MRL. We provide detailed discussion of recommended - additional PTs in Section 9.14. Together with the USACE borings, a total of 66 exploration points were available from the existing and preliminary explorations, east of the NOV levee that borders Barataria Bay, and 21 additional 3-inch diameter borings in the Barataria Bay marsh, west of the NOV Levee. Existing exploration points are shown on the drawings in **Appendix D**. These exploration points are concentrated in or very near to the MRL.

9.4.2 Available Laboratory Data

The USACE's borings and borings from the 2014 Baseline Study provided samples for laboratory tests. These tests were supplemented by the results of CPTs and FV shear tests obtained during the 2014 Baseline Study. Besides classification tests, laboratory tests consisted of numerous unconsolidated undrained triaxial shear (UU) three-point tests, unconfined compression (UC) tests, grain size analyses, and -#200 particle size sieve determinations. The 2014 Baseline Study also included flexible wall permeability tests and 51 consolidation tests. However, of the 51 consolidation tests, 31 tests were performed on samples obtained from borings made to the west of the NOV Levee in the Barataria Bay marsh. Four consolidated undrained (CU) triaxial tests were performed on samples in a single boring between the ground surface and 10-foot depths. As a result, the data are isolated and in areas of the subsurface that do not fully support the geotechnical design.

9.5 Site Geology Based on Existing Data and Information

The site can be divided into several major depositional units including a complex point bar deposit at the Mississippi River. This point bar deposit is overlain by natural levee deposits extending into the marsh area to the west of the project's intake at the Mississippi River. Both the marsh and natural levee deposits overlie undifferentiated interdistributary/intradelta sequences lain in brackish water environments and, in turn, nearshore Gulf and prodelta deposits lain in salt water environments. The deltaic deposits are incised by two abandoned distributary channels identified in the preliminary exploration. These abandoned channels are shown in the USACE's geologic study as part of the Cheniere Traverse Bayou entrenchment and deposition. Studies by the USACE indicate these abandoned channels may be interconnected extending westward toward the remnant abandoned distributaries of Bayou Barataria. The surface of deposits from the Pleistocene Epoch appears to be between EL -100 and EL -125 (referenced to the MSL Datum) at the Mississippi River with a general trend at approximate EL -110 along the proposed Conveyance Channel alignment. Area geology is shown in plan and cross-sections shown in **Appendix D**.

9.6 Supplemental Geotechnical Exploration Program

9.6.1 General

The goal of the supplemental geotechnical exploration program is to provide deeper exploration in the vicinity of the structural features of the project and sufficient area coverage along the guide levees and Conveyance Channel, highway bridge, tie-ins to the NOV Levee, near-field Outfall basin, and potential back structure and Siphon locations. Undisturbed borings will obtain samples from borings using 5-inch diameter fixed piston samplers and general type borings will obtain 3-inch diameter Shelby tube samples. CPTs and small diameter direct-push borings will be used to supplement the other boring data. Exploration points and depths are summarized in **Appendix D**. The field program began in July 2018 and will be completed in November 2018. The field program has been and will continue to be performed in accordance with the Quality Control Plan for the project dated November 2017.

9.6.2 Headworks

The Headworks (HW) structures will be installed or partially installed within dry excavations requiring dewatering and hydrostatic pressure relief. These excavations will also require cofferdams for construction. Preliminary engineering in this 15% design effort have cofferdams comprising cellular type and earthen structures. The earthen structures will eventually be guide levees for the Conveyance Channel. Data will be obtained to evaluate pile capacity, lateral stability of the HW structures, settlement, seepage, slope stability of tie-in flood protection, and conceptual design requirements for dewatering, pressure relief, stability, and seepage for the temporary works (cofferdams). Additional explorations for the cellular structures (i.e., protective cellular structures in the River), structural cofferdams, earthen cofferdams, and dewatering and pressure relief will be performed after the 15%

design as the designs of permanent works progresses and when the CMAR starts designs of the temporary works.

Point bar deposits comprise two predominant units and are a significant concern for design. The lower unit comprises coarser and more uniform sands and extends from the river bottom at approximate EL -50 to EL -130 (reference to NAD 83/WGS 84 Data). Pressure relief in this stratum will likely be feasible using relatively widely spaced pumped wells around the perimeter of the headworks excavation. Landward point bar deposits comprise interbedded fine grained silts, silty sands, and clay. These deposits extend from approximately EL -10 to EL -110 and overlie the coarser point bar sands. The upper point bar deposits will be difficult to dewater; closely spaced wells along the top of sloped excavation will likely be necessary to lower the phreatic surface to several feet below planned excavation subgrade elevation. It also may be necessary to seal the perimeter wells and either pump them with jet eductors or with submersible pumps supplemented with vacuum pumps to achieve vacuum inside of the well casings. Where the stratum of coarse point bar sand exists below the fine point bar silt and clay, groundwater in the higher point bar deposit may drain into the lower point bar sands when this stratum is pumped and provide adequate lowering of the phreatic surface in the point bar silt. Conceptual recommendations for pumping tests in both the lower coarse point bar sand and the upper point bar silt are presented in Section 9.14. The results of such testing are needed to complete the design of temporary groundwater control systems for the headworks excavation. Unprotected slopes of open excavations in the fine point bar deposits will erode readily. Even if the phreatic surface in the fine point bar deposit is lowered several feet below the planned excavation bottom, these soils will still be almost fully saturated and will be easily disturbed under construction traffic. It is expected that the upper point bar silt will drain slowly; it is possible that additional wells inside of the excavation may be necessary to lower the phreatic surface in the fine point bar if the drainage to a perimeter well system is too slow.

Supplemental exploration for the HW structures and earthen structures include 10 undisturbed borings obtaining 5-inch diameter fixed piston samples seven CPTs. Borings and CPTs have been or will be made to depths of 140 feet below the existing ground surface or mudline in the river to investigate the HW structures. These borings will penetrate the point bar deposits and extend into the Pleistocene formation. One additional undisturbed boring was made to 200 feet below the existing ground surface to further investigate the nature of the Pleistocene deposits. Additional deeper explorations (to EL -130, or 140 feet deep) will be necessary in future design phases to better delineate the extent and thickness of the coarse point bar sand where the dewatering wells/drains are to be installed around the perimeter of the planned HW excavation.

9.6.3 Conveyance Channel and Guide Levees

Data will be obtained to evaluate slope stability, settlement, and underseepage. The USACE and the CPRA require exploratory points at 500-foot center-to-center spacings along levee structures. In this regard, we will utilize a combination of CPTs and 5-inch diameter fixed piston undisturbed sample type borings made at 500-foot center-to-center spacings along the north and south levee alignments. In general, the CPT locations and the undisturbed boring locations alternate. These exploration points will be made to 80 and 100-foot depths. We have supplemented these borings with 3-inch diameter Shelby tube sampled borings made along the centerline of the Conveyance Channel. The purpose of these borings is to evaluate near-surface borrow material for use in the levees, and to complement the borings made along the levee centerline for stability analysis purposes. The borings will be made to 60-foot depths considering we anticipate the Conveyance Channel will be as deep as EL -25.

9.6.4 Siphon

Data will be gathered to support analyses for bearing capacity and settlement of the Siphon structure. Seepage analyses will also be required. One row of three borings will be located along the centerline alignment of the Siphon. Depending upon the Siphon's location (and if more than one Siphon will ultimately be required), north and south guide levees, and conveyance centerline borings will be adjusted to accommodate the Siphon location(s). One boring will be a 5-inch diameter undisturbed boring, one boring will be a 3-inch diameter general type boring, and one CPT will be made to depths of 140 feet.

9.6.5 Outfall Structure

Data will be obtained to evaluate pile capacity, lateral stability of the Outfall structures, settlement, slope stability of tie-in flood protection, and conceptual requirements for excavations. Additional explorations necessary for excavation (if any) will be included with the CMAR contracts. The Outfall structure may be located at the extreme western terminus of the guide levees, or may be located within the Conveyance Channel as the USACE is considering relocating the NOV Levee inland to reduce the overall length of the flood protection. Two additional undisturbed borings will be made at the structure and Outfall channel. Two CPTs will be made to assist design of the tie-in to the existing back levee. These borings and CPTs will be made to 140-foot depths.

9.6.6 Hwy 23 and Approaches

Geotechnical analyses for the bridge will focus on pile capacity and settlement for piers. Special consideration will be given to floodwalls (pile supported T-Walls) beneath the bridge along the guide levee alignments. Data must be gathered to evaluate pile capacity, lateral stability, settlement, settlement induced bending moment at the floodwalls, and seepage. Proposed locations of the exploration points for the bridge and approaches are shown in **Appendix D**.

Discussions have been undertaken with the State of Louisiana, Department of Transportation and Development (LADOTD). LADOTD has indicated they require borings or CPTs made at every bent or pier supporting the bridge. With the bridge design being in preliminary stages and bent/pier locations not established, borings and CPTs will be made at approximate, alternating 100-foot center-to-center spacings and at the currently planned bent/pier locations. These borings/CPTs will be made to 170-foot depths. We propose to perform seven 3-inch diameter Shelby tube sample borings and eight CPTs at 15 bent locations. Borings at two bents will be slightly relocated and positioned at T-Wall features below the bridge. At these locations, two 5-inch diameter undisturbed borings will obtain fixed piston samples extending to 140-foot depths. These borings will be completed to 170-foot depths obtaining 3-inch diameter Shelby tube samples. LADOTD also requires shallow borings to characterize subgrade materials along approaches. Six approach borings will be sampled with an auger and sampled continuously to 10-foot depths outside of the approach ramps. Two additional 3-inch diameter borings will be made to 120-foot depths at the ramps. These borings in association with the 3-inch diameter borings at the edges of the bridge are intended to support design of the ramps.

9.6.7 Point Bar Sampling and Outfall Channel Sampling

General type borings obtaining standard penetration tests (SPTs) are planned to investigate the erodibility characteristics of the Mississippi River point bar deposits at the project's intake and the Barataria Bay marsh deposits at the project's Outfall. Six exploration point locations are planned for each of these two areas, and will be established by the project's hydraulic engineers.

9.7 Supplemental Laboratory Testing Summary

9.7.1 General

Soil laboratory testing on samples obtained from the borings follows the same schedule as the field program (began in July 2018) and will finish a few weeks after completing the field program (approximately November 2018). The soil laboratory testing program has been and will continue to be performed in accordance with the Quality Control Plan for the project dated November 2017. We are providing a testing protocol consistent with current USACE, CPRA, and LADOTD standards for samples obtained from the borings. We will also perform consolidation tests, enhance and expand CU tests (including CKoTXC), and provide direct simple shear (DSS) tests to compliment UU and UC tests. To assist in classification and the evaluation of the drainability of the fine point bar silts, silty sands, and clays, both field and laboratory visual descriptions will include dilatancy (reaction to shaking) observations for all samples of these fine soils.

HW features, and guide levees parallel to the HW, will be located in point bar deposits of interbedded sands and silts for the region river side of the abandoned distributary shown in **Appendix D.** The remaining features will be located in primarily natural levee clay; abandoned distributary sequences of clays, organics and silts; and backswamp/marsh clay; and organic clay in deltaic interdistributary deposits. The underlying Pleistocene deposits are also primarily clay-type soils.

9.7.2 Undisturbed Borings

USACE and CPRA testing protocols require most samples be subjected to UU tests. In this regard, we will provide a UU test at every 10-foot depth of cohesive deposits. UC tests will alternate with UU tests at every 10 feet thus providing undrained shear strength tests for every 5 feet of sample. The USACE and CPRA also require Atterberg limits determinations for each UU test. This requirement will be part of Eustis Engineering's testing protocol.

Special concern will be given to the nature of point bar deposits in the HW area. These deposits do not lend themselves well to undisturbed sampling and shear strength testing. We anticipate standard penetration testing (SPT) to obtain samples of these point bar deposits and grain size analyses on the -#200 sieve. We will also perform hydrometer dispersion tests on silt and sand deposits to characterize these deposit's propensity to be drained by pumping. Clay deposits encountered with the upper point bar deposits will be subjected to the testing protocols previously described for cohesive materials. Dilatency testing is also being performed to characterize the ability of clay and silt soil samples to drain. Given the heterogeneous nature of point bar deposits, our testing protocols will be flexible and adjusted based on sample recovery.

Consolidation tests obtained for the 2014 Baseline Study, were primarily concentrated in the vicinity of the MRL or in the Barataria Bay marsh borings west of the NOV Levee. Only 11 consolidation tests were available along the levee alignments. Eustis Engineering will supplement these consolidation tests near the vicinity of the HW in the MRL and focus on obtaining additional consolidation tests along the Conveyance Channel guide levees, Siphon structure, and Outfall canal structure.

Point bar deposits (sands and silts) are predominate in the HW structure and MRL tie-in areas. The USACE evaluates undrained shear strength parameters using CU triaxial shear tests (three-point) with pore pressure measurements. Representative samples will be subjected to these tests for verification of parameters typically assumed in silt materials. Shear strength parameter selection will depend highly on a critical evaluation of UU tests. Sampling and testing techniques introduce potential disturbance that affect the test results. CU triaxial testing can mitigate sample disturbance using normalized testing methods according to the SHANSEP. We will evaluate normalized parameters for the various geologic units along the Conveyance Channel guide levees to verify typically assumed strength to effective stress ratios. We will use perform CU and CKoU triaxial shear tests, consolidated and sheared at various pressures to represent normally consolidated behavior to obtain representative parameters for SHANSEP evaluations of strength gain during stage loading of the levees. We will similarly perform DSS tests in the SHANSEP framework to further aid in parameter selection and help define design strengths for soils sheared in the DSS failure mode.

9.7.3 General Type Borings

In cohesive materials, we will obtain one-point UU tests at 10-foot depths and alternate these with UC tests at 10-foot depths resulting in shear testing every 5 feet. Atterberg limits determinations will be performed for each UU test. Grain size analyses to the -#200 sieve will be performed on any cohesionless material encountered. Of notable concern will be the near surface natural levee backswamp/marsh and undifferentiated interdistributary/intradelta deposits. The general type borings are also planned to investigate these materials as a potential borrow source. In this regard, we will perform moisture content, organic content, and Atterberg limits determinations to establish material quality and constructability.

9.7.4 Small-Diameter, Direct Push Borings

Small-diameter, direct push borings will extend to 20-foot depths to investigate the extent of natural levee deposits and potentially underlying interdistributary/intradelta deposits. Moisture contents will be obtained at 2.5-foot intervals and Atterberg limits determinations at 5-foot intervals. Organic content tests will be established for each sample that has a moisture content in excess of 80%.

9.8 Description of Subsurface Conditions Based on Existing Geotechnical Data

9.8.1 General

The Delineation of Soil Parameters Report presents the DT interpretation of soil reaches and soil design parameters as they interrelate with the proposed project features and general design requirements for MBSD. The Delineation of Soil Parameters Report builds on the Data Gap Analysis Report that was published by the DT in February 2018 and the 2014 Baseline Study documents prepared by HDR Engineering, Inc. and GeoEngineers, Inc. Considering all available information, we designated nine design reaches. Please refer to **Appendix D** for descriptions and extents of the reaches. We developed stratigraphy and parameters for each reach and have based the analyses for the 15 percent design effort on these assumptions. We developed data plots for moisture content, unit weight, undrained shear strength, standard penetration tests, and D10 sizes. We selected soil design parameters based on the plots that are included in the Delineation of Soil Parameters Report. We also designated consolidation parameters in the Soil Delineation Report. We describe the nine soil design reaches in the following sections.

9.8.2 Reaches 1 and 2

Reaches 1 and 2 extend from the riverside extreme limit to Station 25+00 and comprise the river point bar deposits at the Intake Structure. Reach 1 includes locations of the two easternmost borings made in the deeper river and was separated from Reach 2 to investigate potential differences due to location.

Reaches 1 and 2 were subsequently judged to have similar characteristics and may be consolidated after the final exploration. In these reaches, point bar deposits extend to the Pleistocene surface at EL -128. Point bar deposits are coarser sand deposits and the Pleistocene deposits are pre-compressed clays. The deepest exploration point extends to EL -200.

9.8.3 Reaches 3 and 4

Reaches 3 and 4 represent the continuation landward of the point bar deposits between Station 25+00 and Station 35+00. They are overlain by natural levee deposits and the MRL. The deeper point bar deposits have similar characteristics as those in Reaches 1 and 2, i.e., coarser sand deposits but they are overlain at shallower depths by fine sand, silt, and clay deposits. The MRL fill and natural levee deposits are clay soils, more competent below the levee centerline (Reach 3). Natural levee deposits extend to EL -10 and underlying point bar deposits extend to EL -132. Upper point bar deposits between EL -10 and EL -80 are interbedded silts and clays. Separate parameters were selected for these deposits and used for analyses to investigate sensitivity to these variations.

9.8.4 Reach 5

Reach 5 comprises the shallow point bar deposits between Station 35+00 and Station 48+00 and extend to EL -106. These deposits are interbedded clays, sands, and silts that underlie natural levee deposits and interdistribuary deposits. The natural levee and interdistributary deposits are primarily clays and silty clays from the existing ground surface to EL -37. An abandoned distributary indicated by geologic mapping may extend into these natural levee deposits but was not encountered by the preliminary field exploration. Point bar deposits interface with Pleistocene Age clay deposits at EL -104.

9.8.5 Reach 6

Occurring between Station 48+00 and 53+00, Reach 6 is characterized by an abandoned distributary incised into the natural levee and interdistributary deposits that comprise Reach 5. The abandoned distributary deposits are interbedded clays and silts with organic matter. The clays and silts are extended to approximate EL -48 and overlie prodelta clay deposits. Pleistocene clays are encountered at EL -120.

9.8.6 Reach 7

Reach 7 extends from Station 53+00 to 85+00 and comprises natural levee deposits from the existing ground surface extending to EL -10 and interface with deltaic deposits that continue to the Pleistocene interface at EL -120. Two subreaches, Reach 7A and Reach 7B were identified as abandoned distributaries incising the deltaic deposits to approximate EL -42. The deltaic deposits are an interdistributary unit extending to EL -50 and a prodelta unit extending below the interdistributary deposits to EL -115. A sand deposit was encountered between EL -115 and the Pleistocene unit at EL -120. Deltaic deposits are primarily clay with interdistributary deposits containing silt lenses and layers. Abandoned distributary deposits are interbedded clays and silts with organic matter. Reach 7A extends between Station 53+00 and Station 59+00 and Reach 7B extends between Station 78+00 and Station 83+00.

9.8.7 Reaches 8 and 9

Reaches 8 and 9 are similar in geology but differ in land use. Reach 8 is inside (i.e., protected side) of the line of the levee flood protection. Reach 9 is outside (i.e., flood side) of the protection and within Barataria Bay. Both reaches are characterized by surficial marsh deposits underlain by deltaic deposits of interdistributary and prodelta units. Marsh deposits in Reach 9 are weaker and extend to deeper depths. These deposits extend to EL -10 in Reach 8 and EL -15 in Reach 9. The extent of marsh deposits will be a significant design consideration effecting both stability and settlement. In addition, marsh deposits are not suitable levee fill requiring their delineation in the borrow areas. Pleistocene deposits were encountered at EL -120 in Reach 8 but explorations in Reach 9 were not deep enough to establish the Pleistocene surface.

9.9 Mississippi River Flood Protection

9.9.1 Levee Stability and Seepage

At the 15% level, the DT researched the appropriate flood side analysis of the MRL that was performed by the USACE, New Orleans District. The USACE performed a flood side analysis of the MRL using the LMVD Method of Planes and considered a LWL of EL 0.0 (NGVD). The USACE established a stability control line along which all safety factors for levee stability toward the river were at least 1.30. The DT referred to that stability control line when considering various scenarios of excavation (e.g., in-the-wet, in-the-dry, varying intake elevations). The DT ensured that excavations did not encroach upon the stability control line so that contemplated excavations were considered safe with respect to MRL stability.

The existing MRL is underlain by vast point bar deposits of varying sands and silts. Excavations in these deposits present challenges for dewatering and pressure relief. However, with the existing grades on the protected side of the MRL near EL 2 to EL 5 and considering a SWL of EL 12.6, these deposits have shown to have suitable clay blankets overlying the point bar deposits. We conclude that adequate safety factors for heave and exit gradient have been achieved.

9.9.2 Bank Stability

Similar to the levee stability, the DT researched at the 15% level the appropriate flood side analysis of the bank adjacent to the MRL that was performed by the USACE, New Orleans District. The USACE established a stability control line along which all safety factors for bank stability toward the river were at least 1.30. This control line extends from approximately EL 2 along a 4.5H:1V slope down to EL -50, then along a 2H:1V slope down to EL -120. The DT referred to that stability control line when considering various scenarios of excavation (e.g., in-the-wet, in-the-dry, varying intake elevations).

9.10 Mississippi River Scour Protection

The Mississippi River scour protection was not considered in the geotechnical analysis/design at the 15% level. The DT will consider the presence of scour protection along the proposed banklines for flood side analysis of the MRL stability.

9.11 Mississippi River T-Wall Design

The Mississippi River T-Walls will consist of six monoliths (T-1 through T-6) on each side of the U-Frame at Station 29+00 and will connect to the existing MRL. The ground surface on the protected and flood sides is EL 2 at Monoliths T-1 through T-5 and EL 10 at Monolith T-6. Top of wall grade is EL 16.4. Braced excavations will be used to construct the T-Walls and these stability analyses will be performed after the T-Wall construction sequence is developed with the contractor.

The DT performed stability and seepage analyses on select T-Wall monoliths with the flood side water level at EL 16.4 to evaluate unbalanced loads and required sheetpile tip elevations using methods outlined in Section 3.5.11 of the Project DCD using design parameters for Soil Reach 4. A summary of the stability and seepage results performed for the T-Walls in Table 9-1 and the supporting calculations are provided in **Appendix G**.

Table 9-1: Stability and Seepage Results for Mississippi River T-Walls

Monolith No(s).	Protected and Flood Side Ground Surface EL	Base Width (feet)	Bottom of Base Design EL [with 2-feet Working Pad]	Required Sheetpile Tip for Seepage	Stability Factor of Safety	Unbalanced Load (lbs)
T-1	2	32	-49	-111	2.61	0
T-2	2	32	-39	-101	2.71	0
T-3	2	24	-27	-80	2.15	0
T-4	2	15	-17	-80	1.57	0
T-5	2	15	-7	-80	1.44	0
T-6	10	15	-7	-80	3.41	0

Allowable pile load capacities for the Mississippi River T-Walls were computed using methods outlined in Section 3.4.3 of the Project DCD. Various sizes of open end pipe piles with the top of piles at EL -3, -25, and -47 were analyzed using design parameters for Soil Reach 4. Estimates of allowable pile load capacities and supporting calculations are provided in **Appendix G**.

9.12 Headworks Excavation Design and Groundwater Control during Construction

9.12.1 Recommendations for Pumping Test in Clean Point Bar Sand and Overlying Point Bar Silt

The DT recommends that a carefully planned pumping test be performed and analyzed to support final design engineering by the CMAR for the excavations for the HW. The following discussion assumes that an adequate supplemental subsurface investigation with laboratory testing will be completed before the location and design of the test pumping program is started.

9.12.1.1 Purposes of Test and Conceptual Installation, Pumping, and Monitoring Plan

One of the purposes of the pumping test will be to evaluate the actual hydrogeological properties of the clean point bar sand stratum, the distance to the effective source of steady state seepage at the conclusion of the test, the efficiency and safe collection capacity of the test well, and the storativity of the clean point bar sand disclosed by non-steady flow. Although it is useful to evaluate these parameters, a pumping test will elucidate the drainability of the overlying point bar silt and silty fine sand. Advance knowledge of the drainability of the silt will be essential to the timely, effective design and installation of a successful groundwater control system for the HW excavations. It is contemplated that the test well will consist of 10- or 12-inch stainless steel continuous slot pipe size screen and Sch 80 or SDR 21 PVC riser pipe installed in a 24-inch diameter hole drilled using either the flooded reverse circulation or bucket auger method using water or a polymer for the drilling fluid. The well will be screened in the lower 30 to 60 feet (depending on its location) of the clean point bar sand stratum and the screen will be surrounded by a high quality, commercially available uniform silica sand filter graded using appropriate filter design criteria. Because the clean point bar sand is fine and uniform, it is likely that the filter will be either 20/40[1] or 16/30 sand and that the required well screen slot size for a uniformly graded filter will be about 0.020 in. A tentative location for the test well is the north side of the excavation opposite about baseline Station 32+00. Piezometric head and pore pressure monitoring will include 2 radial lines of piezometers at radii of about 25, 50, 100, and 200 feet from the test well. Each piezometer will be equipped with a non-vented pressure transducer with either an onboard datalogger or connected to a master datalogger. The test well flow will be monitored with a flow meter that also includes a datalogger. River stages and barometric pressures will be monitored before, during, and after the pumping test either manually or automatically using level (or pressure) transducers. One little more than half of these piezometers will be installed at two levels in the silt stratum, and the remainder will be installed in the clean point bar sand. One test boring will be drilled and sampled at 5-foot centers full depth in advance close to the proposed test well. At least two tensiometers will be installed in the silt stratum close to the test well at depths of 5 feet and 10 feet below the static phreatic surface for the purpose of estimating the degree of saturation before and during the test.

It is possible that the point bar silt and silty sand will drain vertically into the underlying clean point bar sand stratum by simply lowering the piezometric head in the underlying clean point bar sand formation. Pore pressures in piezometers installed in the overlying point bar silt will be monitored as well as piezometric levels in the underlying clean point bar sand when the test well is pumped. The required test duration is uncertain; the duration should be sufficient to evaluate the time required for 90% drainage of the fine point bar deposits. A reasonable tentative estimate for the pumping test duration is two weeks, followed by two weeks of recovery monitoring. Such a test is necessary to evaluate whether or not gravity drainage of the silt will occur, and if so, what pumping duration is needed for 90% drainage.

However, if clay lenses or layers within the point bar silt stratum prevent vertical drainage of the silt into the point bar sand, additional (2-inch completed diameter, un-pumped) low capacity wells will be needed to induce vertical drainage of this stratum into the underlying clean point bar sand stratum when it is pumped. It is possible that the silt will not drain vertically by gravity even with the addition of closely spaced smaller sized wells around the perimeter of the excavation. To attempt to overcome this potential problem, a vacuum pumping system will be installed and operated with a manifold to produce a small relative vacuum (5 to 10 inches of mercury, or Hg) in the 2-inch low capacity well casings, which will be sealed. The effectiveness of 2-inch un-pumped low capacity wells in achieving drawdown in the point bar silt will be evaluated by installing a 400-foot (total length) section of 2-inch completed diameter wells screened through the both the silt and at least 15 feet into the underlying clean sand stratum. To evaluate the need for and the effectiveness of low capacity wells in expediting drainage of the point bar silt, 17 low capacity wells will be installed in 10-inch diameter jetted or drilled holes 25 feet apart, centered on the high capacity test well. These wells can either be installed concurrently with the installation of the high capacity test well or after the initial pumping indicates that such wells either are or may be necessary. During the installation of the 17 low capacity wells, a vacuum pump and manifold will be installed to connect the pump to the sealed low capacity well casings. The vacuum

[1] These numbers indicate the range of US Standard sieve sizes for the gradation of commercially available filter sands.

pump will be started after an elapsed test pumping time of a few days and the application of vacuum to the low capacity wells will continue until the end of test pumping. Recovery of water levels in the piezometers installed in both the point bar silt and in the underlying point bar sand will be monitored after pumping for at least the same duration as the active pumping. All piezometers, the barometric pressure at the site, and the river stage at the site will be monitored at least 4 times per day for two weeks before the test, hourly during active pumping until 24 hours following the end of pumping, then at least 4 times per day for another two weeks. Instantaneous and cumulative flow measurements will be accurate to within 1% of the measured flow and shall be monitored continuously throughout active pumping. The accuracy of automated instruments (except for the flow meter) will be checked at least twice per day) during active pumping using suitable manual measurement methods. The flow meter will be new or calibrated by the manufacturer within 3 months before its use onsite. Water temperature, pH, conductivity, and oxidation-reduction potential will be monitored at least once per day during active pumping and again one to two weeks after pumping is stopped.

Groundwater samples will be taken at the end of active pumping and one week after the end of pumping and shipped to an approved laboratory for a battery of tests recommended by the laboratory to determine inorganic water chemistry, organic content, the presence and identification of microbiological organisms (bacteria), and the probability of well/pump/pipe fouling or corrosion in pumped well systems and other drainage systems. The laboratory will prepare a report summarizing the test results and its opinions of the potential for well and pump clogging and/or corrosion of metallic well screens and discharge piping. The laboratory will have a successful experience record of preparing such interpretive reports for well and piping systems on a minimum of 10 projects in the preceding 10 years and the report shall be reviewed and co-signed by a subject matter expert in well fouling and corrosion.

The DT will summarize the results of the pumping test in an engineering report, including all collected data, groundwater testing and interpretative report by the well fouling / corrosion subject matter expert, estimations of transmissivity and storativity for the clean point bar sand stratum, and the drainability of the silt stratum under gravity conditions as well as under a small relative vacuum.

9.12.2 Conceptual Designs for Groundwater Control during Construction

Groundwater control during construction was evaluated conceptually for four in-the-dry and one in-the-wet alternative designs, all for the intake gate located at baseline Station 33+50 (450 feet from the MRL):

1. U-Frame intake in-the-dry with invert at EL -40[2]
2. Open channel intake in-the-dry with invert at EL -20
3. Open channel intake in-the-dry with invert at EL -50
4. Submerged culvert intake in-the-dry with invert at EL -50
5. U-Frame in-the-wet cofferdam with invert at EL -40

Summaries of the elements of the five designs are discussed in Sections 9.12.2.1 through 9.12.2.5. In all five cases a (redundant) seepage barrier between EL -60 and EL -135 was included in the conceptual designs and cost estimates as described below to reduce seepage through the clean point bar sand stratum. The dewatering systems were designed independent of the seepage barrier (i.e., assuming

[2] All elevations cited in this report section are in feet and refer to the North American Vertical Datum of 1988, 2009.55 epoch.

that the seepage barrier is not installed). Conceptual designs and corresponding cost estimates for groundwater control during construction include the following common components:

- single panel[3] jet grouted cutoff extending from EL -60 to 5 feet below the Pleistocene clay stratum, completely surrounding the planned excavations where they are underlain by the clean point bar sand stratum, or riverward of approximately baseline Station 38+00, where the clean point bar sand is assumed to pinch out;
- high capacity 10-inch completed diameter pumped wells screened in the clean point bar sand stratum for pressure relief with 300-gpm submersible electric pumps and motor controls;
- low capacity 2-inch diameter un-pumped wells screened in both the upper point bar silt and the underlying clean point bar sand, supplemented if necessary by applying low vacuum (5 to 10 inch Hg) to sealed well casings to induce drainage of the silt;
- low capacity 4-inch completed diameter wells screened only in the upper point bar silt to at least 10 feet below excavation subgrade, sealed and pumped using 4-inch parallel pipe jet eductors to achieve a small relative vacuum (5 to 10 inch Hg) in the wells to induce drainage of the silt, or sealed and pumped using fractional horsepower 4-inch submersible pumps supplemented by an electric vacuum pumping system sized to produce the same small relative vacuum in each of the sealed well casings;
- low capacity 2-inch completed diameter un-pumped wells in each cofferdam cell screened full depth through the cell backfill, natural point bar silt, and the underlying clean point bar sand, supplemented if necessary by a vacuum pumping system to induce a small relative vacuum in the individual well casings;
- system for controlling precipitation and surface runoff that collects within the excavations, including sumps, pumps, piping, and ditches;
- primary and secondary 3-phase electrical distribution, motor controls and monitoring devices, provisions for automatically switched standby power;
- instrumentation for system monitoring (piezometers, flow meters, drawdown in wells, coupons in pumped wells to monitor encrustation, discharge water quality, relative vacuum, current, voltage, frequency, and motor status);
- operation and monitoring, including pump replacement, periodic exercise of standby power generators, sump cleaning, sand content measurements, manual checks of transducer data, plotting, and reporting performance data for systems;

Design calculations and working sketches of conceptual dewatering designs for these alternative cases are included in **Appendix G**. Summaries of the designs are discussed in the following sections, as well as tabulations of assumed design parameters and the results of analytical calculations. Design assumptions/parameters common to all cases analyzed are given in the Table 9-2. A summary of the conceptual designs and cost estimates is presented in Table 9-3.

[3] A more positive jet grouted seepage barrier would comprise double panels that are cris-crossed between grout injection points. Such a barrier would cost approximately twice as much as a single panel barrier.

Table 9-2: Design Assumptions Common to All Cases Analyzed[4][5]

Parameters	Value
Design River Stage (ft, NAVD88)	17.5
Average K_h of Clean Point Bar Sand (cm/sec)	0.015
Average Thickness D of Clean Point Bar Sand below excavation (ft)	60
Average Well Collection Capacity Q_w (gpm)	300
Bottom of Clean Point Bar Sand (ft, NAVD88)	-130
Azimuth and Station of Intersection of Line Source of Seepage with Baseline (degrees / Station)	349 / 15+75

Table 9-3: Summary of Conceptual Design Options and Cost Estimates

Option	Q_t=Flow in Clean Point Bar Sand (gpm)	No. of 10-inch High Capacity Wells	No. of 2-inch Low Capacity Wells (Un-Pumped)	No. of 4-inch Low Capacity Wells (Pumped)	Length of Seepage Barrier (ft)	Cost Estimate With Seepage Barrier	Cost Estimate Without Seepage Barrier
U-Frame Intake In-The-Dry Invert at EL -40	4,000	19	128	37	4,350	$32,275,000	$12,700,000
Open Channel Intake In-The-Dry Invert at EL -20	2,200	7	133	29	3,250	$26,642,500	$12,197,500
Open Channel Intake In-The-Dry Invert at EL -50	4,600	15	102	62	3,580	$28,325,000	$12,215,000
Submerged Culvert Intake In-The-Dry Invert at EL -50	5,200	17	132	53	5,000	$34,410,000	$13,260,000
U-Frame Intake In-The-Wet Invert at EL -40	3,000	11	101	122	2,230	Not Estimated	Not Estimated

9.12.2.1 U-Frame In-the-Dry Intake with Invert at EL -40

9.12.2.1.1 High Capacity Well System

The purpose of the high capacity well system is to depressurize the clean point bar sand stratum beneath the excavations, lowering the piezometric head below the excavation to at least 5 feet below

[4] For the in-the-dry submerged culvert intake at EL -50, the intersection of the assumed line source of seepage with the project baseline is Station 14+40.

[5] For the in-the-dry open channel intake at EL -40, the intersection of the assumed line source of seepage with the project baseline is Station 21+20, azimuth 345°.

planned subgrade (or to EL -55). See **Appendix G** for dewatering design calculations and sketches showing the design assumptions discussed below. Wells will be installed to 5 feet into the Pleistocene clay underlying the clean point bar sand, or to about EL -135. The well borehole diameter will be about 24 inches and the finished well diameter will be 10-inch pipe size, which will allow 300-gpm capacity pumps to be installed in them. Wells riverward of the MRL will be installed on the inboard side of the cellular or the combi-wall cofferdam. Each well will be screened completely through the clean point bar sand stratum. The actual hydraulic conductivity of the clean point bar sand stratum is estimated to be somewhere between 0.015 and 0.05 cm/sec, based on experience and the USACE K_h vs. D_{10} correlation[6], which was used to estimate K_h for representative samples from (Geotechnical Reach 2) Borings R-1A through R-6A. Because the lower two-thirds of the point bar sand is very dense, based on Standard Penetration resistances and the DT's experience with fine sand formations in Louisiana having similar average D_{10} values (about 0.08 to 0.10 mm), the average K_h assumed for all dewatering flow calculations was 0.015 cm/sec. The average stratum thickness at the river end of the cofferdam is estimated to be 83 feet. The borings indicate that the clean point bar sand stratum extends from its outcrop in the river channel landward to the "pinch-out" in the vicinity of boring NL-9A, or at about baseline Station 38+00. The average thickness *D* of the clean point bar aquifer in the area of the excavation riverward of Station 38+00 was assumed to be 60 feet. The effective source of seepage was assumed to be an infinite fully penetrating slot in the river channel about 300 feet (or more) riverward of the river end of the well system (intersecting baseline at Station 15+75, azimuth 349 degrees). The total system flow is estimated to be 4,000 gallons per minute (gpm) using the equation for steady combined artesian-gravity flow to an equivalent well with a radius r_e of 536 feet, as described in **Appendix G**, drawdown inside of the ring of wells to 5 feet below subgrade (or to EL -55), the common assumptions listed in Table 9-2, and the case-specific assumptions listed in Table 9-3. As estimated in **Appendix G**, the estimated average well capacity is about 300 gpm assuming an average formation K_h of 0.015 cm/sec along the well screens, an effective individual well diameter of 1.5 feet, 10 feet of incremental drawdown at individual wells due to well interference, and a wetted screen length of 65 feet at each well. Using a uniform well spacing of 200 feet, 19 wells are required, and for a total system flow of 4,000 gpm, the average flow per well is a little more than 200 gpm. Therefore 300-gpm pumps and appurtenant discharge piping have a Factor of Safety of about 1.5. The required head capacity of each pump is about 93 feet, comprising the sum of the lift [17.5-(-65) = 82.5 feet] and friction and minor losses [allow 10 feet]. For a 300-gpm pump capacity and assuming 2-pole, 460-volt, 60-Hz, 3-phase submersible motors, the pump bowl diameter will be between 6 and 8 inches (inches), and 10-inch-pipe-size wells are appropriate and conservative (robust) for that range of bowl diameters.

9.12.2.1.2 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum

The purpose of the low capacity 2-inch un-pumped well system, which would be screened in both the silt and in the underlying sand on a close (25-foot) center-to-center spacing around the perimeter of the intake excavation landward of the cellular cofferdam is to lower the phreatic surface in the point bar silts to at least the approximately planned subgrade level (EL -50). The total number of these wells is about 98. Because of the fineness of this formation, it is known from experience that the individual well flows and aggregate system flow will be very small and have not been estimated.

[6] USACE (2000) Engineer Manual 1110-2-1913, Design and Construction of Levees, Figure 3.5b, page 3-10

9.12.2.1.3 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum in Cofferdam Cells

The stability of the cofferdam cells requires that the phreatic surface in the individual cells be lowered to the elevation of the inboard stability berm, or to about EL -20. In the DT's judgment, the phreatic surface inside each cofferdam cell can be lowered and maintained at or below EL -20 using one 2-inch diameter un-pumped well in each cell that is screened through the cell backfill, natural levee, fine point bar, and extending 10 feet or more into the underlying clean point bar sand, in conjunction with pumping the high capacity well system to lower the head in the clean point bar sand to EL -55 or deeper. As indicated in **Appendix G** for this design case, 30 of these wells will be required. It will also be necessary to produce a small relative vacuum in the casings to induce drainage of fine grained silts.

9.12.2.1.4 Low Capacity 4-inch Diameter Low Capacity Pumped Well Screened in Point Bar Silt to at Least 10 feet Below Planned Subgrade

Landward of the pinch-out at about baseline Station 38+00, the clean point bar sand does not exist below the point bar silt and lowering the phreatic surface will probably require installing and pumping closely-spaced (25-foot) low capacity wells in the point bar silts. This design spacing will require approximately 37 low capacity pumped wells. Pumping the anticipated small flow from the silt and simultaneously producing a vacuum in the sealed well casings can be accomplished using either 4-inch diameter parallel pipe jet eductors, which will pump both air and water, or by installing small 4-inch diameter submersible pumps in the wells to pump water and using vacuum pumps to produce a small relative vacuum in the sealed well casings. The principal advantage of using submersible pumps rather than jet eductors is that the air-handling capacity of the vacuum pump is much higher.

9.12.2.1.5 Length and Face Area of Seepage Barrier

As indicated in **Appendix G**, the length of the jet grouted seepage barrier will be about 4,350 feet. For treatment between EL -60 and EL -135, the calculated seepage barrier face area is 326,250 square feet.

Table 9-4: Dewatering System Design Summary for U-Frame Intake In-the-Dry with Invert at EL -40

Parameters	Value
Q_t=Flow in Clean Point Bar Sand (gpm)	4,000
r_e = radius of equivalent well (ft)	536
L = Distance to Line Source of Seepage	1,177
No. of 10-inch High Capacity Wells	19
Length of Seepage Barrier (ft)	4,350
No. of 2-inch diameter low capacity un-pumped wells including cell wells	128
No. of 4-inch low capacity pumped wells	37

9.12.2.2 Open Channel Intake In-the-Dry with Invert at EL -20

9.12.2.2.1 High Capacity Well System

The high capacity well system is designed to depressurize the clean point bar sand stratum beneath the excavations, lowering the piezometric head below the excavation to at least 5 feet below planned subgrade (or to EL -35). See **Appendix G** for dewatering design calculations and sketches showing the design assumptions discussed below. Wells will be installed to 5 feet into the Pleistocene clay underlying the clean point bar sand, or to about EL -135. The well borehole diameter will be about 24 inches and the finished well diameter will be 10-inch pipe size, which will allow 300-gpm capacity pumps to be installed in them. Wells riverward of the MRL will be installed on the inboard side of the cellular or the combi-wall cofferdam. Each well will be screened completely through the clean point bar sand stratum. The total system flow was estimated to be 2,200 gpm using the equation for steady combined artesian-gravity flow to an equivalent well with a radius of 534 feet, as described in **Appendix G**, drawdown to EL -35, and the assumptions listed in Table 9-2. Using a uniform well spacing of 200 feet, 7 wells will be required, and for a total system flow of 2,200 gpm, the average required pump capacity per well is about 300 gpm. The required head capacity of each pump is about 93 feet, as calculated in the previous Section (9.12.2.1).

9.12.2.2.2 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum

The purpose of the low capacity 2-inch un-pumped well system, which would be screened in both the silt and in the underlying sand on a close (25-foot) center-to-center spacing around the perimeter of the intake excavation landward of the cellular cofferdam is to lower the phreatic surface in the point bar silts to at least the approximately planned subgrade level (EL -30). Because of the fineness of this formation, it is known from experience that the individual well flows and aggregate system flow will be very small and have not been estimated. As indicated in **Appendix G**, 121 of these wells are required at this spacing.

9.12.2.2.3 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum in Cofferdam Cells

The stability of the cofferdam cells requires that the phreatic surface in the individual cells be lowered to the elevation of the inboard stability berm, or to about EL -20. In the DT's judgment, the phreatic surface inside each cofferdam cell can be lowered and maintained at or below EL -20 using one 2-inch diameter un-pumped well in each cell that is screened through the cell backfill, natural levee, fine point bar, and extending 10 feet or more into the underlying clean point bar sand, in conjunction with pumping the high capacity well system to lower the head in the clean point bar sand to EL -55 or deeper. As indicated in **Appendix G** for this design case, 12 of these wells will be required. It has been assumed that it will be necessary to produce a small relative vacuum in these casings to induce drainage of the silts.

9.12.2.2.4 Low Capacity 4-inch Diameter Low Capacity Pumped Well Screened in Point Bar Silt to at Least 10 feet Below Planned Subgrade

Landward of the pinch-out at about baseline Station 38+00, the clean point bar sand does not exist below the point bar silt and lowering the phreatic surface will probably require installing and pumping closely spaced (25-foot) low capacity wells in the point bar silts. This design spacing will require approximately 29 low capacity pumped wells. Pumping the anticipated small flow from the silt and simultaneously producing a vacuum in the sealed well casings can be accomplished using either 4-inch diameter parallel pipe jet eductors, which will pump both air and water, or by installing small 4-inch diameter submersible pumps in the wells to pump water and using vacuum pumps to produce a small relative vacuum in the sealed well casings.

9.12.2.2.5 Length and Face Area of Seepage Barrier

As indicated in **Appendix G**, the length of the jet grouted seepage barrier will be about 3,370 feet, and for treatment between EL -60 and EL -135, the barrier face area is 252,750 square feet.

Table 9-5: Dewatering Summary for Open Channel Intake In-The-Dry with Invert at EL -20

Parameters	Value
Q_t=Flow in Clean Point Bar Sand (gpm)	2,200
r_e = radius of equivalent well (ft)	534
L = Distance in Feet to Line Source of Seepage (300 ft outboard of well system)	1,350
No. of 10-inch High Capacity Wells	7
Length of Seepage Barrier (ft)	3,250
No. of 2-inch diameter low capacity un-pumped wells (including cell wells)	133
No. of 4-inch low capacity pumped wells	29

9.12.2.3 Open Channel Intake In-the-Dry with Invert at EL -50

9.12.2.3.1 High Capacity Well System

The high capacity well system is designed to depressurize the clean point bar sand stratum beneath the excavations, lowering the piezometric head below the excavation to at least 5 feet below planned subgrade (or to EL -65). See **Appendix G** for dewatering design calculations and sketches showing the design assumptions discussed below. Wells will be installed to 5 feet into the Pleistocene clay underlying the clean point bar sand, or to about EL -135. The well borehole diameter will be about 24 inches and the finished well diameter will be 10-inch pipe size, which will allow 300-gpm capacity pumps to be installed in the wells. Wells riverward of the MRL will be installed on the inboard side of the cellular or combi-wall cofferdam. Each well will be screened completely through the clean point bar sand stratum. The total system flow was estimated to be 4,600 gpm using the equation for steady combined artesian-gravity flow to an equivalent well with a radius of 620 feet, as described in **Appendix G**, drawdown to EL -65, and the common assumptions listed in Table 9-2.

Using a 300-gpm well capacity, 15 wells will be required, and the calculated total system flow is 4,600 gpm. The required head capacity of each pump is estimated to be about 93 feet, as calculated in the previous Section (9.12.2.1).

9.12.2.3.2 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum

The purpose of the low capacity 2-inch un-pumped well system, which would be screened in both the silt and in the underlying sand on a close (25-foot) center-to-center spacing around the perimeter of the excavation landward of the cellular cofferdam is to lower the phreatic surface in the point bar silts to at least the approximately planned subgrade level (EL -50). Because of the fineness of this formation, it is known from experience that the individual well flows and aggregate system flow will be very small and have not been estimated. As indicated in **Appendix G**, 88 of these wells are required at this spacing.

9.12.2.3.3 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum in Cofferdam Cells

The stability of the cofferdam cells requires that the phreatic surface in the individual cells be lowered to the elevation of the inboard stability berm or to about EL -20. In the DT's judgment, the phreatic surface inside each cofferdam cell can be lowered and maintained at or below EL -20 using one 2-inch diameter un-pumped well in each cell that is screened through the cell backfill, natural levee, fine point bar, and 10 feet or more into the underlying clean point bar sand, in conjunction with pumping the high capacity well system to lower the head in the clean point bar sand to EL -65 or deeper. As indicated in **Appendix G** for this design case, 14 of these wells will be required. It has been assumed that it will also be necessary to produce a small relative vacuum in the casings to induce drainage of fine grained silts.

9.12.2.3.4 Low Capacity 4-inch Diameter Low Capacity Pumped Well Screened in Point Bar Silt to at Least 10 feet Below Planned Subgrade

Landward of the clean point bar sand pinch-out at about baseline Station 38+00, that stratum does not exist below the point bar silt and lowering the phreatic surface will probably require installing and pumping closely spaced (25-foot) low capacity wells in the point bar silts. This design spacing will require approximately 62 low capacity pumped wells. Pumping the anticipated small flow from the silt and simultaneously producing a vacuum in the sealed well casings can be accomplished using either 4-inch diameter parallel pipe jet eductors, which will pump both air and water, or by installing small 4-inch diameter submersible pumps in the wells to pump water and using vacuum pumps to produce a small relative vacuum in the sealed well casings.

9.12.2.3.5 Length and Face Area of Seepage Barrier

As indicated in **Appendix G**, the length of the jet grouted seepage barrier will be about 3,580 feet, and for treatment between EL -60 and -135, the barrier face area is 268,500 square feet.

Table 9-6: Dewatering Summary for Open Channel Intake In-the-dry with Invert at EL -50

Parameters	Value
Q_t=Flow in Clean Point Bar Sand (gpm)	4,600
r_e = radius of equivalent well (ft)	622
L = Distance to Line Source of Seepage	1,420
No. of 10-in. High Capacity Wells	15
Length of Seepage Barrier (ft)	3,580
No. of 2-in. dia. low capacity un-pumped wells (including cell wells)	102
No. of 4-in. low capacity pumped wells	62

9.12.2.4 Submerged Culvert Intake In-The-Dry with Invert at EL -50

9.12.2.4.1 High Capacity Well System

The high capacity well system is designed to depressurize the clean point bar sand stratum beneath the excavations, lowering the piezometric head below the excavation to at least 5 feet below planned subgrade (or to EL -65). See **Appendix G** for dewatering design calculations and sketches showing the design assumptions discussed below. Wells will be installed to 5 feet into the Pleistocene clay underlying the clean point bar sand, or to about EL -135. The well borehole diameter will be about 24 inch and the finished well diameter will be 10-inch pipe size, which will allow 300-gpm capacity pumps to be installed in the wells. Wells riverward of the MRL will be installed on the inboard side of the cellular or the combi-wall cofferdam. Each well will be screened completely through the clean point bar sand stratum. The total system flow was estimated to be 5,200 gpm using the equation for steady combined artesian-gravity flow to an equivalent well with a radius of 751 feet, as described in **Appendix G**, drawdown to EL -65, and the common assumptions listed in Table 9-2.

Using a 300-gpm well capacity, 17 wells will be required. The required head capacity of each pump is estimated to be about 93 feet, as calculated in the previous section (9.12.2-1).

9.12.2.4.2 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum

The purpose of the low capacity 2-inch un-pumped well system, which would be screened in both the silt and in the underlying sand on a close (25-foot) center-to-center spacing around the perimeter of the excavation landward of the cellular cofferdam is to lower the phreatic surface in the point bar silts to at least the approximately planned subgrade level (EL -50). Because of the fineness of this formation, it is known from experience that the individual well flows and aggregate system flow will be very small and have not been estimated. As indicated in **Appendix G**, 96 of these wells are required at this spacing.

9.12.2.4.3 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum in Cofferdam Cells

The stability of the cofferdam cells requires that the phreatic surface in the individual cells be lowered to the elevation of the inboard stability berm or to about EL -20. In the DT's judgment, the phreatic surface inside each cofferdam cell can be lowered and maintained at or below EL -20 using one 2-inch diameter un-pumped well in each cell that is screened through the cell backfill, natural levee, fine point bar, and 10 feet or more into the underlying clean point bar sand, in conjunction with pumping the high capacity well system to lower the head in the clean point bar sand to EL -65 or deeper. As indicated in **Appendix G** for this design case, 36 of these wells will be required. It has been assumed that it will also be necessary to produce a small relative vacuum in the casings to induce drainage of the point bar silts.

9.12.2.4.4 Low Capacity 4-inch Diameter Low Capacity Pumped Well Screened in Point Bar Silt to at Least 10 feet Below Planned Subgrade

Landward of the clean point bar sand pinch-out at about baseline Station 38+00, that stratum does not exist below the point bar silt and lowering the phreatic surface will probably require installing and pumping closely spaced (25-foot) low capacity wells in the point bar silts. This design spacing will require approximately 62 low capacity pumped wells. Pumping the anticipated small flow from the silt and simultaneously producing a vacuum in the sealed well casings can be accomplished using either 4-inch diameter parallel pipe jet eductors, which will pump both air and water, or by installing small 4-inch diameter submersible pumps in the wells to pump water and using vacuum pumps to produce a small relative vacuum in the sealed well casings.

9.12.2.4.5 Length and Face Area of Seepage Barrier

As indicated in **Appendix G**, the length of the jet grouted seepage barrier will be about 5,000 feet, and for treatment between EL -60 and -135, the barrier face area is 375,000 square feet.

Table 9-7: Dewatering Summary for Open Channel Intake In-the-Dry at EL -50

Parameters	Value
Q_t=Flow in Clean Point Bar Sand (gpm)	5,200
r_e = radius of equivalent well (ft)	751
L = Distance to Line Source of Seepage	1,300
No. of 10-inch High Capacity Wells	17
Length of Seepage Barrier (ft)	5,000
No. of 2-inch diameter low capacity un-pumped wells (including cell wells)	132
No. of 4-inch low capacity pumped wells	53

9.12.2.5 U-Frame Intake In-The-Wet with Invert at EL -40

9.12.2.5.1 High Capacity Well System

The high capacity well system is designed to depressurize the clean point bar sand stratum beneath the excavations, lowering the piezometric head below the excavation to at least 5 feet below planned subgrade (or to EL -55). See **Appendix G** for dewatering design calculations and sketches showing the design assumptions discussed below. Wells will be installed to 5 feet into the Pleistocene clay underlying the clean point bar sand, or to about EL -135. The well borehole diameter will be about 24 inches and the finished well diameter will be 10-inch pipe size, which will allow 300-gpm capacity pumps to be installed in the wells. Wells riverward of the MRL will be installed on the inboard side of the cellular or the combi-wall cofferdam. Each well will be screened completely through the clean point bar sand stratum. The total system flow was estimated to be 3,000 gpm using the equation for steady combined artesian-gravity flow to an equivalent well with a radius of 378 feet, as described in **Appendix G**, drawdown to EL -55, and the common assumptions listed in Table 9-2.

Using a 200-foot well spacing, 11 wells will be required. The required head capacity of each pump is estimated to be about 93 feet, as calculated in the previous Section (9.12.2.1).

9.12.2.5.2 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum

The purpose of the low capacity 2-inch un-pumped well system, which would be screened in both the silt and in the underlying sand on a close (25-foot) center-to-center spacing around the perimeter of the excavation landward of the cellular cofferdam is to lower the phreatic surface in the point bar silts to at least the approximately planned subgrade level (EL -50). Because of the fineness of this formation, it is known from experience that the individual well flows and aggregate system flow will be very small and have not been estimated. As indicated in **Appendix G**, 88 of these wells are required at this spacing.

9.12.2.5.3 Low Capacity 2-inch Un-pumped Well System Screened in Point Bar Silts and Underlying Clean Sand Stratum in Cofferdam Cells

The stability of the cofferdam cells requires that the phreatic surface in the individual cells be lowered to the excavation subgrade elevation (EL -50). In the DT's judgment, the phreatic surface inside each cofferdam cell can be lowered and maintained at or below EL -50 using one 2-inch diameter un-pumped well in each cell that is screened through the cell backfill, natural levee, fine point bar, and 10 feet or more into the underlying clean point bar sand, in conjunction with pumping the high capacity well system to lower the head in the clean point bar sand to EL -55 or deeper. As indicated in **Appendix G** for this design case, 13 of these wells will be required. It has been assumed that it will also be necessary to produce a small relative vacuum in the casings to induce drainage of the point bar silts.

9.12.2.5.4 Low Capacity 4-inch Diameter Low Capacity Pumped Well Screened in Point Bar Silt to at Least 10 feet Below Planned Subgrade

Landward of the clean point bar sand pinch-out at about baseline Station 38+00, that stratum does not exist below the point bar silt and lowering the phreatic surface will probably require installing and pumping closely spaced (25-foot) low capacity wells in the point bar silts. This design spacing will require approximately 122 low capacity pumped wells. Pumping the anticipated small flow from the silt and simultaneously producing a vacuum in the sealed well casings can be accomplished using either 4-inch diameter parallel pipe jet eductors, which will pump both air and water, or by installing small 4-inch diameter submersible pumps in the wells to pump water and using vacuum pumps to produce a small relative vacuum in the sealed well casings.

9.12.2.5.5 Length and Face Area of Seepage Barrier

As indicated in **Appendix G**, the length of the jet grouted seepage barrier will be about 2,230 feet, and for treatment between EL -60 and -135, the barrier face area is 167,250 square feet.

Table 9-8: Dewatering Summary for U-Frame Intake In-The-Wet with Invert at EL -40

Parameters	Value
Q_t=Flow in Clean Point Bar Sand (gpm)	3,000
r_e = radius of equivalent well (ft)	378
L = Distance in Ft to Line Source of Seepage (300 ft outboard of well system)	1,432
No. of 10-inch High Capacity Wells	11
Length of Seepage Barrier (ft)	2,230
No. of 2-inch dia. low capacity un-pumped wells (including cell wells)	101
No. of 4-inch low capacity pumped wells	122

9.12.3 Seepage Cutoff Evaluations

No seepage cutoff is required for the HW excavation, in the DT's opinion. Dewatering the fine point bar deposit by pre-drainage using widely-spaced deep wells tapping the underlying coarse point bar sand could well be all that is required for adequate groundwater control during construction. In the worst case, closely spaced wells that completely penetrate the fine point bar and are pumped with or without applied vacuum may be required in addition to the coarse point bar deepwell system. Installing a seepage barrier is not required for the success of this method of groundwater control. Although it is theoretically possible to dewater the fine point bar soils by open sumping, the excavation slopes would necessarily have to be very flat, even assuming that a fully penetrating seepage barrier were installed in advance of excavation. If the fine point bar deposit does not drain vertically because of layer of clay, there will also be seepage stability problems at such interfaces if open sumping with no pre-drainage is the dewatering method. The need for a seepage barrier will be evaluated again during the 30% design for the post-construction case of a high river stage in conjunction with a closed intake. It is likely that hydrostatic pressure relief may be necessary immediately landward of the intake gate for that case. There may be other cases requiring permanent seepage control measures in the HW area that will also be evaluated during the 30% design phase.

9.12.4 Combi Walls/Cellular Structures

9.12.4.1 Cellular Structures

Construction of the "in-the-dry" options involves cellular cofferdams extending from approximately 150 feet east of the MRL centerline into the river. The distances that the cells extend into the river differ between the EL -20 and EL -40 options as shown in **Appendix D**.

The DT sized the cofferdams for the 15 percent level of design initially based on published case histories for two riverine cofferdams of roughly similar height in similar soil conditions. These two case histories were the cofferdams for Locks and Dam 26 in the Mississippi River near St. Louis, Missouri[7] and cofferdam for Olmsted Lock on the Ohio River near Olmsted, Illinois.[8] Basic details of these cofferdams and the selected dimensions for the MBSD cofferdam are summarized in Table 9-9 below and drawings are provided in **Appendix D**.

Table 9-9: Attributes of Referenced Cofferdams and Those Selected for MBSD Cofferdam

Attribute	LD 26	Olmsted	Selected for MBSD
Diameter (ft)	63	62.7	63
Maximum height (Top of cofferdam elevation-mudline elevation)	60	69	65*
Maximum Head Difference (Design river elevation - dewatered design elevation on land side)	83	104	65*
Width of Top of Landside Berm (ft)	20	20	20
Slope of Landside Berm and Soil Type of Berm	5H:1V sand	3H:1V, sand	3:1 riprap
Distance from Top of Cofferdam to top of Berm (ft)	35	39	35*
Foundation Soil Below Cofferdam	Sand	Sand with stiff clay	Sand (east end) Sand, silt,clay (west end)
Cofferdam Dewatered	yes	yes	yes
Penetration of Sheet pile Below Mudline	35	40	35

*Assumed top of cofferdam at EL 15 which was later changed to EL 17.5

After sizing the MBSD cofferdam based on similar case histories, rough calculations were made to confirm that the size selected was reasonable. Calculations generally followed the USACE guidelines in EM 1110-2-2503, "Design of Sheet Pile Cellular Structures, Cofferdams, and Retaining Structures," 29 September 1989. This method differs slightly from others in the references cited below which were also reviewed.

---

[7] Clough, G.W.,Kuppusamy,Thangavelu,"Finite Element Analyses of Lock and Dam 26 Cofferdam", Journal of Geotechnical and Environmental Engineering, American Society of Civil Engineers, Volume 111, No. 4, April 1985, pages 521-540.

[8] Mansur, C.I., Durrett, S.G.,"Dewatering Cofferdam for Construction of Olmsted Locks", Journal of Geotechnical and Environmental Engineering, American Society of Civil Engineers, Volume 126, No. 6, June 1, 2002, pages 496-510.

Two different subsurface profiles are applicable to design of the cofferdams, one at the east end of the cofferdam furthest in the river, and a second near the MRL. For the invert EL -40 option, the cofferdam extends approximately 900 feet from the MRL into the river. Subsurface conditions there are represented by Reaches 1 and 2 conditions from the Soil Delineation Report. Subsurface conditions in Reaches 1 and 2 are essentially identical, i.e. coarse point bar deposits in the river bed to at least EL -85.

Further west, near the existing MRL, soil conditions are less well defined because no borings are located in the river near the levee. The closest borings in the river are about 650 feet east of the MRL. The closest profile is at Reach 3 along the center of the MRL which was used for evaluation. An east-west subsurface profile is shown in **Appendix D**. As noted in the figure, subsurface conditions at the east end of the cofferdam consist of sand fill within the cofferdam over a clean coarse sand foundation (coarse point bar deposits). These conditions are preferable to the west end of the cofferdam near the MRL where soil conditions consist of silts, silty sand, and clay (fine point bar deposits) from surface grade (EL -6) to about EL -90. Below EL -90 coarse point bar sands are generally present, similar to the east end of the cofferdam.

For the EL -20 invert, the entire cofferdam is within about 500 feet of the MRL and soil conditions for calculations were assumed to be represented by only Reach 3 conditions.

Analysis of Cells at the East End of the MRL (Reach 1 and 2 Soil Conditions), Invert EL -40.

Analysis of this cofferdam using only drained (S-case) conditions because foundation soils and cell fill will be sand. Additional assumptions for this case are:
Top of cofferdam is EL 17.5 with river at EL 17.5

Wall friction was ignored.

Seepage was assumed controlled to EL -50 on the protected side of the cofferdam and to EL -16.25 inside the cell (halfway between the river elevation and dewatered level inside the cofferdam) assuming the cells are dewatered.

The cofferdam was analyzed with no penetration below the mudline and therefore results calculated and summarized below are conservative since the actual cofferdam extends 35 feet below the mudline.

The preliminary calculations confirmed that key factors of safety were met. Therefore the dimensions of the cofferdam assumed based on published case histories was reasonable for Reach 1 and 2 soil conditions in the river near where boring information is available.

Analysis of Cells near the MRL, (Reach 3 Soil conditions), Invert EL -40

By inspection, bearing capacity at EL -85 for the Reach 3 soil conditions showed it was inadequate. Calculations indicated a Factor of Safety on bearing capacity less than 1.0 and well below the 3.0 value recommended by the USACE.

Therefore the cells were deepened to EL -100 to bear in the dense which is present near EL -90. Deepening the cofferdam will also help with sliding, overturning and vertical shear that would likely be an issue at shallower depth. Extending the cells to EL -100 would only be needed for cells within 300 feet of the levee where the depth of the coarse point bar sand deepens based on existing data.

When more detailed information is available it may show that a larger diameter cell is needed due to conditions near the MRL, but that should not have a major impact on the cost of steel, since the steel required is approximately independent of the cell diameter. The only additional cost for a larger cell would be for cell fill, but this should fall within the contingency in the budget.

The closer that cells are to the MRL, the greater the amount of existing sediment that might be left inside the cells. These clayey and silty materials result in higher interlock tension than the sandy cell fill further east. Therefore, it is recommended that sheets with high interlock strength (32 k/in) be assumed for cost estimating. For conservatism this is recommended for all sheets not just the ones within 300 feet of the MRL.

Cells for EL -20 Option

For this option, cells will be closer to the MRL. It is assumed that soil conditions for Reach 3 to be representative, although no borings are in the river in this area. The elevation of the cut in this area is to EL -30, or 20 feet above the grade for the EL -40 option. Soil at this elevation in Reach 3 are clays with undrained shear strength of about 400 to 600 psf which are worse than conditions for the EL -50 cut where stiffer soils are present. Again, by inspection bearing capacity in the clays above the coarse point bars sands would not be satisfactory.

Consequently, it is recommended that these cells within 300 feet of the levee also extend to EL -100 into the dense sands for bearing capacity. Extending to the sand will also improve sliding resistance and other modes of failure. Refer to Figure 9-1 for cells that should extend to EL -100.

Construction Considerations

Dewatering of the cofferdams is vital for stability and therefore Eustis recommends that dewatering be included in the estimates. The two referenced case histories also included active dewatering inside the cells.

If soft clays are present in the river, they should be removed within the cells down to the stiffer clays and sands near EL -40. Further analysis after exploration in this area will determine the need for this. Most of these soft clays will likely be removed for constructability.

Driving of sheets this long (117.5 feet) may be difficult, although sheets of roughly this length were successfully driven for the two case histories noted. Pre-excavation, jetting, impact driving or other measures may be needed. Sheets should have a minimum thickness of ½ inch to improve drivability.

The contractor will have to evaluate stability and interlock tension during various phases of construction. These depend on his method and sequence of construction and were not evaluated in these calculations. For example the highest interlock tension noted in the Locks and Dam 26 case history occurred during filling of the cells.

An instrumentation program to monitor cell movements and possibly interlock stresses during and after construction should be developed by the contractor to provide early warning of potential problems so that they can be mitigated before a serious problem could potentially develop.

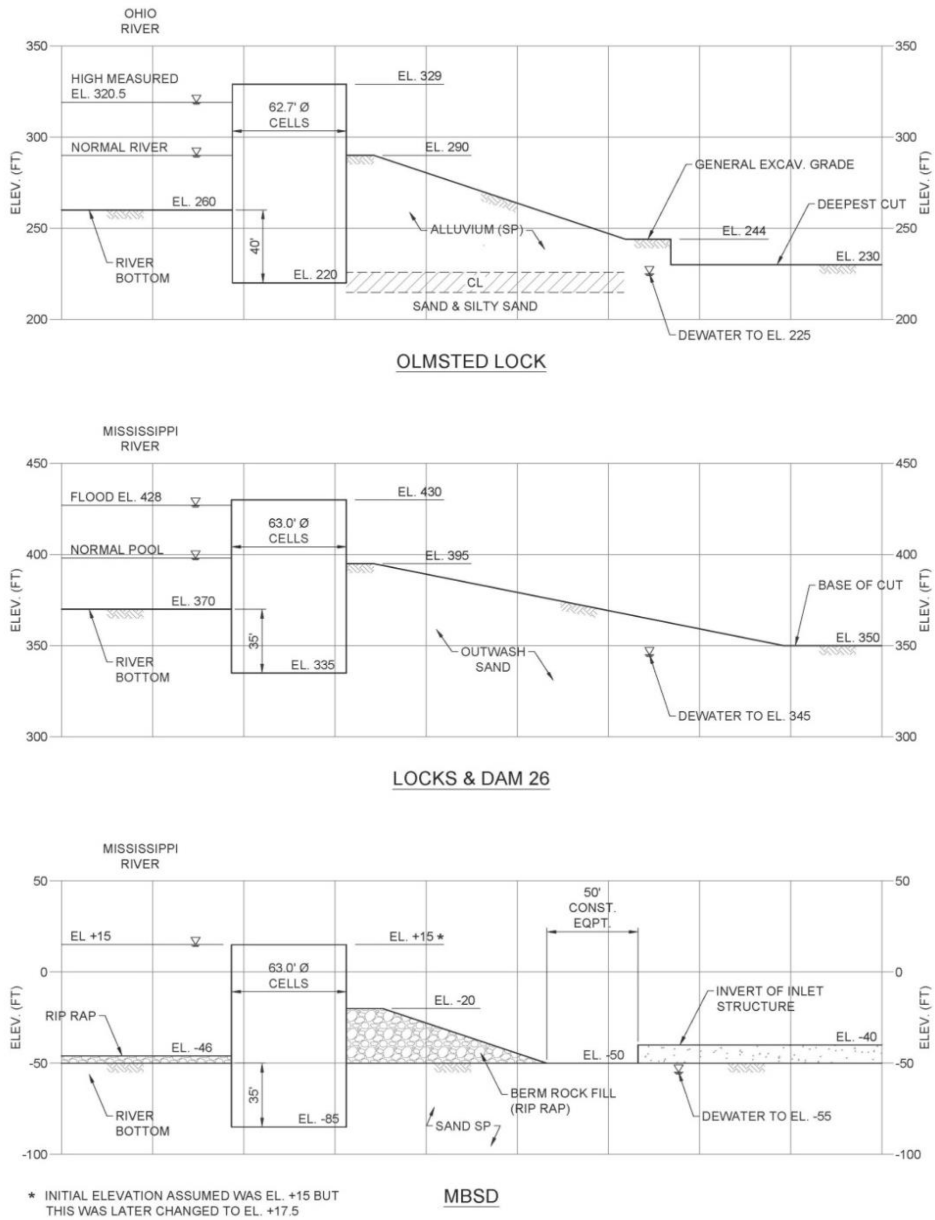


Figure 9-1: Comparison of Cofferdam Designs for Locks and Dam 26, Olmsted Lock and MBSD

9.13 Headworks Excavation Design and Dewatering (In-the-Wet)

9.13.1 Settlement of MRL Interim Levees during Construction

The DT performed settlement analyses for the MRL Interim levees using the settlement analysis program SETTLE 3D by RocScience, Inc. Vertical stresses were computed using Westergaard solutions. Soil consolidation parameters were based on soil consolidation and index tests presented in the Soil Delineation Report for Soil Reaches 4 and 5. The MRL Interim levees will serve as the Conveyance Channel levees after construction, therefore, the settlement analyses for the MRL Interim levee are applicable to the Conveyance Channel levees in Soil Reach 5. Total settlement estimates account for consolidation, immediate, and secondary settlement. It is assumed immediate settlement as approximately 30% of the total consolidation settlement. This assumption is based on DT's experience with levee construction in similar geologic conditions.

The DT used wick drains in the analyses to expedite consolidation settlement during construction. The DT considered wick drains installed through the Holocene deposits and terminating in the Point Bar deposits at approximate EL -50 in Soil Reach 4 and EL -37 in Soil Reach 5. The assumed triangular wick drain spacing for DT's analyses is 5 feet. The DT assumed the levee will be constructed in multiple 6-foot lifts with each lift occurring during a 6-month period. The DT performed iterative time-rate settlement analyses for determination of the levee overbuild required to maintain the levee design elevation for a period of 4 years after the end of levee construction. The DT provides a summary of the MRL Interim levee parameters in Table 9-10, the results of the settlement analyses in Table 9-11, and the settlement calculations in **Appendix G**.

Table 9-10: MRL Interim Levee Parameters

Soil Reach	Station Nos.	Existing Ground Surface EL (NAVD88)	Top of Levee Design EL (NAVD88)	Total Height of Fill Placement at Centerline of Levee (feet)	Number of Lifts during Construction	Construction Duration Assuming 6 Months per Lift
Reach 4	30+00 to 35+00	4	16.0	14.5	3	1.5
Reach 5	35+00 to 48+00	3	16.0	15	3	1.5

Table 9-11: Conveyance Channel Levee Settlement Summary

Soil Reach	Top of Levee Design EL (NAVD88)	Levee Overbuild EL at End of Construction (NAVD88)	Total Ground Surface Settlement at End of Construction (feet)	Total Ground Surface Settlement Occurring 4 Years after End of Construction (feet)
Reach 4	16.0	17.0	1.5	0.7
Reach 5	16.0	17.0	1.0	0.6

9.13.2 Mississippi River Interim Levee Stability

The DT performed stability analysis of the Conveyance Channel to evaluate potential modes of failure and establish critical water levels in the channel. All stability analyses use undrained shear strength (Q-case) parameters for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices. This method satisfies moment/force equilibrium and include non-circular and circular searches. Analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical circular and non-circular slip surfaces were optimized, and tension cracks filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. The DT's stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The proposed bottom of the head works excavation is at EL -25 for the graving site (Soil Reach 5) and EL -50 at the gated structure and intake areas (Soil Reach 4). Design grade for the interim MRL is at EL 16.0 with 4H:1V side slopes. To determine the allowable slopes for the HW excavation, the assumed the interim MRL will be constructed while the excavation is dewatered to approximately 5 feet below the bottom of the excavation. A minimum required Factor of Safety of 1.30 was considered during construction. The allowable excavation slopes for each component of the HW excavation are shown in Table 9-12.

Table 9-12: Stability Results of Headworks Excavation Slopes

SOIL DESIGN REACH	GROUND SURFACE EL (NAVD88)	EXCAVATION BOTTOM EL (NAVD88)	EXCAVATION SLOPE	FACTOR OF SAFETY
4[1]	4	-50	6.5H:1V	1.34
5	3	-25	5.5H:1V	1.31

[1] DT considered both ML and CL soil types between el -10 and -34 for stability analyses.

The DT performed global stability analyses of the interim MRL with respect to the dewatered HW excavation considering a levee construction overbuild at EL 17. Strength gain of the foundation soils due to consolidation was considered for the interim MRL stability analyses. The DT discusses the methodology used for strength gain computations in Section 9.16.3. The results of the global stability analyses for the interim MRL and dewatered HW excavation are presented in Table 9-13.

Table 9-13: Interim MRL Global Stability Results –Excavation Dewatered

SOIL DESIGN REACH	EXCAVATION BOTTOM EL (NAVD88)	EXCAVATION SLOPE	INTERIM MRL CENTERLINE OFFSET FROM EXCAVATION TOE IN (FEET)	INTERIM MRL FLOODISDE TOE OFFSET FROM TOP OF EXCAVATION	FACTOR OF SAFETY
4[1]	-50	6.5H:1V	401	50	1.32
5	-25	5.5H:1V	252	103	1.32

[1] DT considered both ML and CL soil types between EL-10 and -34 for the analyses.

Part of the interim MRL constructed within Reach 5 will serve as the Conveyance Channel levee following the completion of HW construction. Deep soil mixing will be required to allow the interim MRL centerline to align with the future Conveyance Channel centerline while maintaining a 300-foot wide excavation bottom. The DT used deep soil mixing to EL -37 and extending 20 feet from the proposed top of HW excavation towards the interim MRL to provide a minimum Factor of Safety of 1.30 during construction. The DT assumed an improved shear strength value of 1,500 psf for soil mixing. The DT considered the global stability of the interim MRL as well as the local stability of the excavation. The results of the stability analyses of the interim MRL and dewatered HW excavation with soil mixing are presented in Table 9-14.

Table 9-14: Interim MRL Global Stability Results – Soil Mixing

SOIL DESIGN REACH	EXCAVATION BOTTOM EL (NAVD88)	EXCAVATION SLOPE	INTERIM MRL CENTERLINE OFFSET FROM EXCAVATION TOE (FEET)	INTERIM MRL FLOODISDE TOE OFFSET FROM TOP OF EXCAVATION (FEET)	FACTOR OF SAFETY	
					GLOBAL	LOCAL
5	-25	5.0H:1V	199	64	1.34	1.31

The HW excavation will be flooded after construction of the Intake Structure components and the interim MRL will need to meet a minimum stability Factor of Safety of 1.40 for flood protection. Eustis performed stability analyses for these cases considering water levels at EL 0.0(LWL) and EL 17 (top of levee). Deep soil mixing was used for the flood side analysis at Soil Reach 5. Below is a summary of these analyses on Table 9-15.

Table 9-15: Interim MRL Stability Results for Low Water Level and Flood Conditions

SOIL DESIGN REACH	SLIP SURFACE DIRECTION	EXCAVATION SLOPE	FLOOD SIDE WATER EL (NAVD88)	FACTOR OF SAFETY
4	Flood Side	6.5H:1V	0	2.08
	Protected Side	6.5H:1V	17	1.65
5	Flood Side	5.0H:1V	0	1.76
	Protected Side	5.0H:1V	17	1.52

9.13.3 Cellular Structure Buttress of Levee

The DT provided a concept for using cellular structures to serve as a buttress providing significant lateral support to the existing MRL during excavation for the gated structure. The 15% design did not further this concept from the geotechnical perspective. Cell design would include evaluations of MRL global stability, cell overturning and sliding to ensure the sheetpile tip penetrations below the bottom of excavations are adequate. Internal stability calculations such as vertical shear and hopp stress evaluations would also be required to size the sheeting and cell diameters. When site specific geotechnical data are obtained, the DT will discuss this concept with the CMAR.

9.13.4 Guide Levee Stability during Construction

The DT analyses performed for the interim MRL presented in Section 9.13.2 are applicable to the guide levees for the Conveyance Channel because the interim MRL design grade (EL 16) is approximately 0.4 feet higher than the Conveyance Channel levee design grade (EL 15.6). The DT has performed separate analyses to evaluate the Conveyance Channel levees for Soil Reach 5 which are presented in Section 9.16.

9.13.5 Seepage Analyses

Seepage analyses were performed for the interim MRL using Lane's Weighted Creep Ratio (LWCR). Soil Reach 4 was considered the critical case for seepage due to the presence of silt at approximately EL -10. The DT assumed water the interim MRL design grade (EL 16) for DT's analyses and analyzed multiple seepage paths through the foundation subsoils. Recommended minimum values for LWCR are 3 for clayey soils and 8.5 for silt[9]. Blanket theory performed in accordance with Appendix B of EM 1110-2-1913, Design and Construction of Levees and DIVR 1110-1-400, was used where the computed LWCR did not meet the recommended minimum values. Hydraulic conductivity data gathered from the previous exploration within Soil Reach 4 was used to develop parameters for blanket theory analysis. The results of DT's seepage analyses indicate that the interim MRL meets the required factors of safety for seepage and minimum recommended LWCR values. The DT provided seepage calculations in **Appendix G**.

[9] Lane, E. W., "Security from Under-seepage: Masonry Dams on Earth Foundations," Trans. Am. Soc. Civil Eng., vol. 100, p. 1257, 1935

Refined seepage analyses using blanket theory including SEEP/W (modeling) will be performed for the next phase and will include additional soil data collected during the field exploration.

9.14 Headworks (HW)

9.14.1 Piles Selection and Capacities

The DT computed allowable pile load capacities for the HW using methods outlined in Section 3.4.3 of the Project DCD. The DT analyzed 24, 36, and 48-inch open end pipe piles, 24-inch square precast concrete piles, and 14-inch H-piles using design parameters for Soil Reach 4. The top of piles and ground surface were assumed to be at EL -50. The DT provided the estimates of allowable pile load capacities and supporting calculations in **Appendix G**.

9.14.2 Global Stability

Global stability through the HW structure (i.e., pile supported gated structure) will be provided by the shear strength within the foundation soils underlying the structure. With an invert at EL -40, the likely bottom of concrete is at EL -50. The underlying soils are medium stiff to stiff clays interspersed with silts, sandy silt and silty sands. Once constructed, the HW structure will be designed to withstand the differential water across the structure that would be anticipated by a high river event. At this time, the DT's opinion is that the HW structure will satisfy the global stability requirements for Factor of Safety. This is considering the foundation soil types, the width of the structure (floodside to protected side dimension), and the supporting foundation piles. Once the additional geotechnical data is obtained and soil design parameters are verified, then a global stability analysis will be performed to verify that lateral forces do not need to be carried by the foundation piles to provide the required safety factors (often termed an "unbalanced load").

9.14.3 Underseepage Assessment/Permanent Cutoff

The DT's opinion is that the most severe seepage problems will be at the intake gate when the intake is closed and the river is at the design flood stage. The lowest completed grade will be EL -40, and the top of the coarse point bar below the gate is at about EL -90, or 50 feet below the finished concrete grade at the gate. For the same reasons stated in previous Section 9.12.3, The DT's opinion is that a seepage cutoff will probably be unnecessary for permanent seepage control. Seepage can be probably effectively controlled for the critical permanent construction case using a combination of permanent relief wells and aggregate drains. This design aspect will be carefully evaluated in the next phase of design after the results of current subsurface explorations and laboratory testing currently underway are known, together with the results of the pumping test contemplated on the coarse point bar sand stratum near the proposed gate. For this (future) evaluation, a 2D plan view numerical seepage model will be developed to evaluate seepage stability and to design seepage control measures for pressure relief. If these design analyses indicate that a seepage cutoff is necessary, a cutoff will be designed at that time. An advantage of using a pressure relief well system for permanent seepage control is that it can be designed to be pumped temporarily to provide the pressure relief needed for unwatering during gate maintenance.

9.14.4 Wing Walls at Transition Channel

The wing walls at the transition channel will consist of 14 T-Wall monoliths (T-1 through T-14) on each side of the channel. The T-Walls will start at Station 36+15 (T-1) and end at Station 42+00 (T-14) where it will tie into the Conveyance Channel levee. The protected side of the T-Walls will be backfilled to EL 2.

Design ground surface elevations on the flood side of the T-Walls vary from EL -40 at Monolith T-1 to EL 2 at Monolith T-14. Top of wall grade is EL 15.6.

The DT performed stability and seepage analyses on select T-Wall monoliths to evaluate unbalanced loads and required sheetpile tip elevations using methods outlined in Section 3.5.11 of the Project DCD using design parameters for Soil Reach 5. Due to the differential fill height between the protected side and flood side, the critical failure case occurs towards the flood side at Monolith T-1 when the water level in the channel is at EL 0.0. The DT computed an unbalanced load of 58.7 kips/ft for this case. To negate the unbalanced loads on Monolith T-1 and maintain a stability Factor of Safety of 1.40, the DT used deep soil mixing on the protected side of the T-Wall to EL -55 and extending 55 feet from the protected side edge of the T-Wall base. The DT assumed an improved shear strength value of 1,500 psf for soil mixing. Soil mixing will need to extend from Monolith T-1 through T-3 to reduce unbalanced loads to a practicable value. The DT provides a summary of the stability and seepage results performed for the T-Walls in Table 9-16 and the supporting calculations are provided in **Appendix G**.

Stability of the T-Walls should be considered for the various constructions stages to ensure the T-Walls are not subjected to excessive unbalanced loads during soil mixing operations and prior to flooding of the Conveyance Channel. This can be achieved by utilizing temporary stability berms on the flood side and/or braced excavations. Analyses will be performed to evaluate T-Wall stability throughout construction after the T-Wall construction sequence is developed with the contractor.

Table 9-16: Stability and Seepage Results for Transition T-Walls

Monolith No(s).	Flood Side Ground Surface EL (NAVD88)	Base Width (feet)	Bottom of Base Design EL [with 2-foot Working Pad] (NAVD88)	Required Sheetpile Tip EL for Seepage (NAVD88)	Stability Factor of Safety	Unbalanced Load (lbs)	Comments
T-1	-37	31	-49	-107	1.40	58700	Stability towards Flood Side
					1.44	0	Soil mix to 55 feet from T-Wall to EL -55
					1.30	0	Construction Case: 3,000 psf surcharge load required on flood side to negate unbalanced loads during soil mixing.
T-2	-30	31	-44	Not performed	1.40	23500	Stability towards Flood Side
T-3	-25	31	-39	Not performed	1.40	3000	Stability towards Flood Side
T-4	-18	31	-34	-92	1.87	0	Stability towards Flood Side
T-6	-11	24	-22	-62	Not performed	0	
T-9	-1	15	-14	-52	Not performed	0	
T-11 to T-14	2	15	-7	-52	2.14	0	Stability towards Protected Side

The DT computed allowable pile load capacities for the wing walls using methods outlined in Section 3.4.3 of the Project DCD. The DT analyzed various sizes of open end pipe piles with the top of piles at EL -3 and EL -49 using design parameters for Soil Reach 5. The DT provides estimates of allowable pile load capacities and supporting calculations in **Appendix G**.

9.15 Conveyance Channel Slope Stability

The DT performed stability analysis of the Conveyance Channel to evaluate potential modes of failure and establish critical water levels in the channel. All stability analyses use undrained shear strength (Q-case) parameters for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices. This method satisfies moment/force equilibrium and includes non-circular and circular searches. Analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical circular and non-circular slip surfaces were optimized, and tension cracks filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. DT's stability analyses were performed in accordance with Section 3.5.1 of the Project DCD.

The DT performed stability analyses of the proposed 4H:1V channel side slopes using soil design parameters presented in the Soil Delineation Report for Soil Reaches 5 through 8. The bottom of channel is at EL -25. The stability analyses considered two water levels within the Conveyance Channel: water at EL 0.0 simulates in-the-wet excavation of the channel and water at EL -25 simulates in-the-dry excavation of the channel. Porewater pressures were defined using a piezometric line in the Slope/W program. Seepage forces and transient pore water pressures were not considered for these analyses. The table below summarizes the results of the stability analyses of the Conveyance Channel side slope excavations. The DT provides stability calculations in **Appendix G**.

Table 9-17: Conveyance Channel Excavation Stability Results

WATER ELEVATION IN CHANNEL (NAVD 88)	CHANNEL SIDE SLOPES	MINIMUM FACTOR OF SAFETY					
		REACH 5	REACH 6	REACH 7	REACH 7A	REACH 7B	REACH 8
0	4H:1V	2.16	2.81	1.99	2.43	2.61	1.24
-25	4H:1V	1.09	1.38	0.99	1.22	1.17	0.58

Staged excavations and stability berms are necessary to maintain a minimum Factor of Safety 1.30 during excavation of the Conveyance Channel with 4H:1V side slopes. The DT performed analyses for three excavation stages as outlined below using design parameters for Soil Reaches 5, 7, and 8. The staged excavation results for Soil Reach 7 are considered applicable to Soil Reaches 6, 7A, and 7B.

Stage 1: The Conveyance Channel is excavated in-the-dry to the depth which maintains a minimum Factor of Safety 1.30 with a 4H:1V side slope. Stage 1 excavation details and results are provided on Figure 9-2 and Table 9-18.

Stage 2 The Conveyance Channel is excavated in-the-dry to EL -25 utilizing a stability berm at the toe of the Conveyance Channel slope to maintain a minimum Factor of Safety of 1.30 for global and local stability. Stage 2 excavation details and results are provided on Figure 9-3 and Table 9-19.

Stage 3 The Conveyance Channel is flooded prior to excavating the stability berm. Stage 3 excavation details and results are provided on Figure 9-4 and Table 9-20.

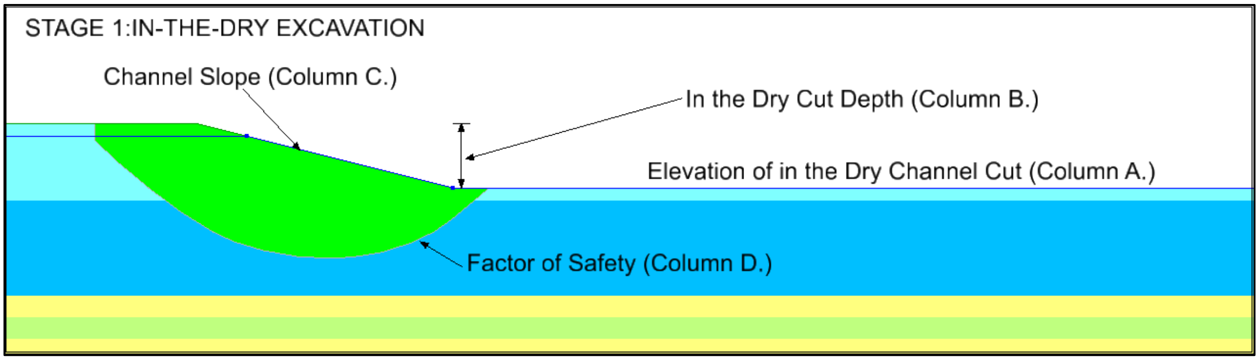


Figure 9-2: Stage 1 Conveyance Channel Excavation

Table 9-18: Stage 1 Conveyance Channel Excavation Results

SOIL DESIGN REACH	GROUND SURFACE EL (NAVD88)	A IN-THE-DRY EXCAVATION EL (NAVD88)	B IN-THE DRY DEPTH OF EXCAVATION (FEET)	C IN-THE-DRY CHANNEL SIDE SLOPES	D FACTOR OF SAFETY
5	3	-12	15	4H:1V	1.34
7	0	-14	14	4H:1V	1.32
8	-3	-10	7	4H:1V	1.32

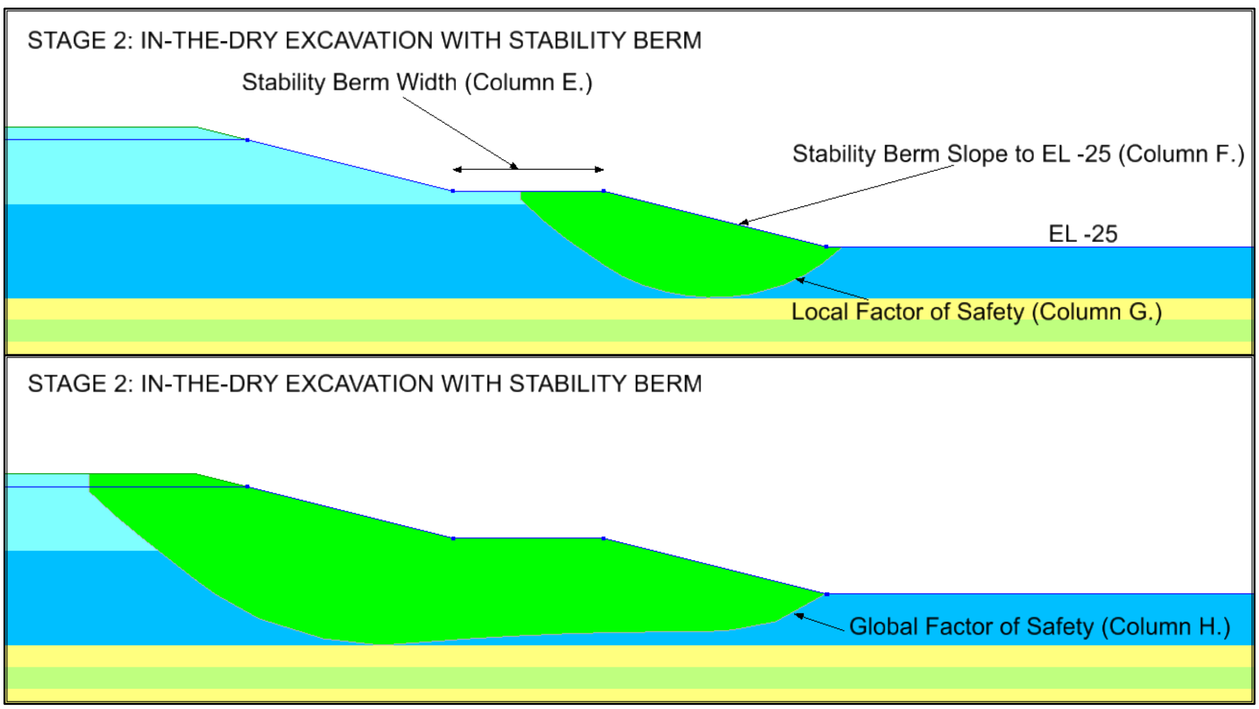


Figure 9-3: Stage 2 Conveyance Channel Excavation

Table 9-19: Stage 2 Conveyance Channel Excavation Results

SOIL DESIGN REACH	GROUND SURFACE EL (NAVD 88)	E	F	G	H
		STABILITY BERM WIDTH IN FEET	STABILITY BERM SLOPES	FACTOR OF SAFETY	
				LOCAL	GLOBAL
5	3	35	4H:1V	1.99	1.34
7	0	20	4H:1V	1.94	1.34
8	-3	65	7H:1V	1.32	1.32

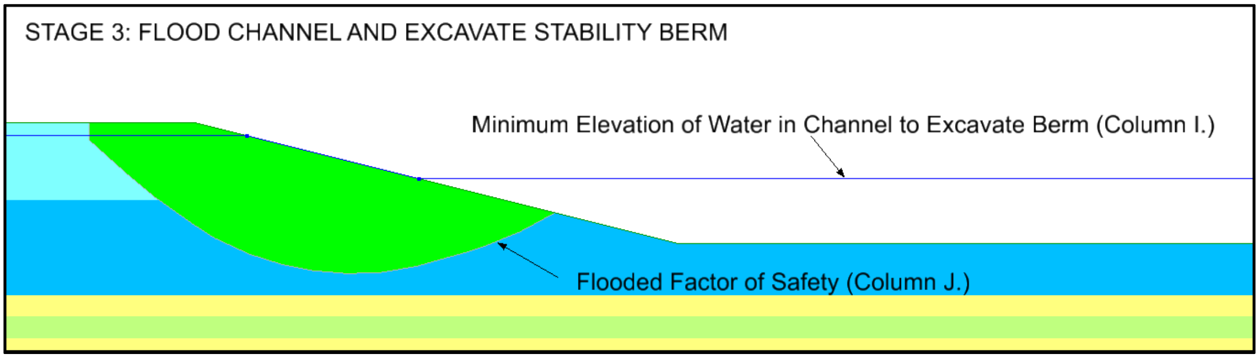


Figure 9-4: Stage 3 Conveyance Channel Excavation

Table 9-20: Stage 3 Conveyance Channel Excavation Results

SOIL DESIGN REACH	GROUND SURFACE EL (NAVD88)	I	J
		MIN. WATER EL IN CHANNEL TO EXCAVATE STABILITY BERM (NAVD88)	FACTOR OF SAFETY
5	3	-10	1.31
7	0	-12	1.31
8	-3	-7.5	1.31

9.16 Conveyance Channel 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. Settlement, slope stability, and seepage analyses for the CCL were performed. CCL design grades at EL 13 and EL 15.6 with side slopes of 4H:1V were analyzed. The centerline of the CCL will be offset approximately 150 feet from the edge of the Conveyance Channel. Iterative analyses to compute settlement, strength gain of the foundation soils, overbuild elevation of the levee, and evaluate stability were performed.

9.16.1 Slope Stability Analyses

The DT performed stability analysis of the CCL to evaluate potential modes of failure and establish critical water levels in the channel. All stability analyses use undrained shear strength (Q-case) parameters for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices. This method satisfies moment and force equilibrium, and include non-circular and circular searches. Analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical circular and non-circular slip surfaces were optimized and tension cracks filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. DT's stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The DT performed stability analyses for the CCL at Soil Reaches 5, 7, and 8. The results for Soil Reach 7 should be considered applicable to Soil Reaches 6, 7A, and 7B. The CCL overbuild crown elevations were used in the analyses for levee design grades at EL 15.6. The DT did not consider resistance contribution from geosynthetic fabric or slope armoring. In addition, seepage forces and transient porewater pressures were not considered. The stability cases analyzed and results of the analyses are summarized in Table 9-21. The DT analyzed these cases for end-of-construction (EOC) conditions considering a minimum Factor of Safety of 1.30.

Table 9-21: Conveyance Channel Levee Stability Results

SOIL DESIGN REACH	STABILITY CASE	CCL CROWN EL (NAVD88)	WATER EL IN CHANNEL (NAVD88)	FACTOR OF SAFETY
5	Levee failure towards flood side with water at EL 0.0 (LWL)	16.6	0	1.59
	Bank failure towards flood side with water at EL 0.0 (LWL)	N/A	0	2.16
	Levee failure towards protected side with water at top of levee	16.6	16.6	1.60
7	Levee failure towards flood side with water at EL 0.0 (LWL)	17.7	0	1.51
	Bank failure towards flood side with water at EL 0.0 (LWL)	N/A	0	1.99
	Levee failure towards protected side with water at top of levee	17.7	17.7	1.85
8	Levee failure towards flood side with water at EL 0.0 (LWL)	18.9	0	1.32
	Bank failure towards flood side with water at EL 0.0 (LWL)	N/A	0	1.40
	Levee failure towards protected side with water at top of levee	18.9	18.9	1.32

The DT performed stability analyses based on the settlement and strength gain estimates to maintain a minimum Factor of Safety for stability of 1.30 during construction. When necessary, the DT extended the duration of the stage prior to placing the final lift to allow additional strength gain and meet minimum Factor of Safety requirements for stability. This iterative procedure was used to develop the stage loading times presented in Table 9-22. The DT anticipates strength gain induced by the final lift will increase stability factors of safety to the minimum value required during operations and flood events. Refined analyses will be performed during the next stage of the project using soil data obtained during the field exploration.

Note, strength gain calculations for the MRL Interim Levee at Soil Reach 5 were used for the stability analyses of the CCL at Soil Reach 5. The MRL Interim Levee design grade is at EL 16 which may be considered applicable for the CCL design grade at EL 15.6. The DT considered this reasonably equivalent for the 15% level of design.

9.16.2 Settlement Analyses

The DT performed settlement analyses for the CCL using the settlement analysis program SETTLE 3D by RocScience, Inc. Vertical stresses were computed using Westergaard solutions. Soil consolidation parameters were based on soil consolidation and index tests presented in the Soil Delineation Report for Reaches 7 and 8. The DT has assumed the settlement analyses for Reach 7 are applicable for the Conveyance Channel Reaches 6, 7A, and 7B. Total settlement estimates account for consolidation, immediate, and secondary settlement. The DT assumed immediate settlement due to lateral spread as approximately 30% of the total consolidation settlement. The DT based this assumption on the experience with levees being constructed on soft marsh deposits in Louisiana. The DT modeled immediate settlement in SETTLE 3D using typical elastic modulus (Es) values for each soil type and consistency in the model.

The DT used wick drains in the DT's analyses to expedite consolidation settlement during construction. The DT considered wick drains installed through the Holocene deposits to approximate EL -115 using a triangular wick drain spacing of 5 feet. The DT assumed the levee will be constructed in multiple 6-foot lifts with each lift occurring during a 6-month period. The DT performed iterative time-rate settlement analyses for determination of the levee overbuild required to maintain the levee design elevation for a period of 10 years after the end of levee construction. The DT provides a summary of the CCL parameters in Table 9-22 and the results of the settlement analyses Table 9-23.

Table 9-22: Conveyance Channel Levee (CCL) Parameters

Soil Reach	Station Nos.	Existing Ground Surface EL (NAVD88)	Top of Levee Design EL (NAVD88)	Total Height of Fill Placement at Centerline of Levee (feet)	Number of Lifts during Construction	Construction Duration (Years) Assuming 6 Months per Lift
Reach 7	48+00 to 85+00	0	13.0	19	4	2
			15.6	22	5	2.5
Reach 8	85+00 to 140+00	-3	13.0	30	5	2.5
			15.6	34	6	4 (1)

(1) Time between the final two lifts was extended to 1.5 years to satisfy slope stability requirements.

Table 9-23: Conveyance Channel Levee Settlement Summary

Soil Reach	Top of Levee Design EL (NAVD88)	Levee Overbuild EL at End of Construction (NAVD88)	Total Ground Surface Settlement at End of Construction (feet)(1)	Total Ground Surface Settlement Occurring 10 Years after End of Construction (feet)(2)
Reach 7	13.0	15.8	3.2	2.2
	15.6	17.7	4.3	1.9
Reach 8	13.0	18.5	8.5	5.2
	15.6	18.9	12.1	3.0

(1) These values represent the total settlement experienced over the construction period from the fill placed above the existing ground surface to achieve the constructed top-of-levee grade.
(2) These values represent the settlement predicted at the levee crown that will be experienced 10 years after the end of construction. These values do not include any settlement experienced during construction.

9.16.3 SHANSEP/Stage Construction

The consolidation-induced gain in strength of foundation soils due to the placement of the CCL following the Stress History and Normalized Soil Engineering Properties (SHANSEP) method outlined by Ladd and Foott, 1974 was estimated. The relationship between the undrained shear strength of a soil, the effective vertical stress, and the over-consolidation ratio of the soil is defined as,

$$\frac{S_u}{\sigma'_v} = S(OCR)^m \qquad (9.17.3\text{-}1)$$

where
S_u = undrained shear strength
σ'_v = effective vertical stress
S = undrained strength ratio
OCR = over consolidation ratio
m = empirical exponent

The over consolidation ratio is defined as the ratio of the maximum vertical stress a soil has experienced to the current vertical stress,

$$\frac{\sigma'_p}{\sigma'_v} = OCR \qquad (9.17.3\text{-}2)$$

where
σ'_p = past maximum vertical effective stress

The undrained strength ratio for the soils encountered at the MSBD site is estimated as 0.22, as discussed in the Delineation of Soil Parameters report. The empirical exponent is 0.8 for the DT's analyses. The SHANSEP equation may be rearranged to solve for OCR. Using the rearranged SHANSEP equation and the parameters discussed above, the DT computed the initial OCRs for each soil strata using the shear strengths presented in the Delineation of Soil Parameter report and the in-situ vertical effective stress.

The DT used the RocScience Inc. program, SETTLE 3D to compute changes in effective vertical stress due to consolidation at the center of each foundation soil strata. Vertical stress distributions were computed using the Westergaard solution within the SETTLE 3D program. The improved undrained shear strength estimated for each foundation soil sublayer was computed by applying the increase in vertical effective stress beneath the CCL and change in OCR due to newly induced loads.

As discussed in Section 9.17.2 of this report, the DT assumed staged construction of the CCL in 6-foot soil lifts. The allowable thickness of each lift will be estimated in the next phase of the project. Six-foot lifts were assumed to simplify the settlement analyses. The DT anticipates the actual lift thickness will vary for each Soil Design Reach based on the bearing capacity of the foundation soils.

The critical stability case for the staged construction process is immediately after the placement of the final lift. In this instant, the levee crown is at its highest elevation prior to foundation soils experiencing consolidation-induced strength gain from the final lift. The DT estimates strength gain based on consolidation of the foundation soils during the second-to-last construction lift with the CCL crown at its respective overbuild elevation for each soil reach. The DT evaluated strength gain at various locations within the CCL cross-section which include the edge of the stability berms, the center of the stability berms, the center of the CCL slopes, and the center of the CCL crown. In-situ soil strengths were assumed at the edge of the stability berm. Strength gain calculations are provided in **Appendix G**.

9.16.4 Wick Drain Assessment

Settlement analyses considers the use of wick drains to accelerate consolidation to induce strength gains of the Holocene-Era deposits beneath the CCL. Wick drains may experience folding or kinking due to the anticipated vertical and lateral settlement of the CCL within Reaches 7 and 8. An instrumentation program should be implemented to monitor pore pressures and settlement during CCL construction. If excess pore pressures are unable to dissipate as estimated, it may become necessary to install wick drains for the CCL a second time to allow remaining excess pore pressures to dissipate. Instrumentation observations will determine if this secondary wicking of the foundation soils will be necessary.

9.16.5 Borrow Pit Excavations

The DT has performed slope stability analyses to optimize proposed borrow pit geometries with respect to the CCL at Soil Reaches 7 and 8. The CCL at Soil Reaches 7 and 8 requires the most fill, therefore, the DT has only performed the borrow pit analyses for these soil reaches. The results for Soil Reach 7 may be conservatively applied to Soil Reach 5 and 6.

The excavations of on-site borrow pits are expected to occur during the construction of the CCL. Local stability of the borrow pit excavations were assessed to provide recommended excavation slopes and bottom elevations for the borrow pit. The DT provides multiple safe excavation slopes that correspond with different bottom elevations. Depending on right-of-way requirements, there may be a borrow pit geometry that is more desirable in terms of total volume of fill available. Note, the side slopes and excavation bottom elevation that the DT presents are not dependent on the CCL, and therefore may be applied to any excavation performed within the limits of Soil Reaches 7 and 8 that meet the minimum offset requirements from the CCL as discussed herein. The DT considered a minimum required Factor of Safety of 1.30 for the local stability analyses of the borrow assuming this is a temporary condition during construction. The results of the analyses are presented in Table 9-24.

Table 9-24: Local Stability Results for Borrow Pit Excavation

SOIL REACH	EXCAVATION SLOPE	EXISTING GROUND SURFACE EL (NAVD88)	BOTTOM OF EXCAVATION EL (NAVD88)	DEPTH OF EXCAVATION (FEET)	FACTOR OF SAFETY
7	4H:1V	0	-14	14	1.3
	6H:1V	0	-30	30	1.34
8	4H:1V	-3	-10	7	1.31
	7H:1V	-3	-12	9	1.31
	10H:1V	-3	-30	27	1.35

During construction of the CCL, the DT has assumed the borrow pit will remain fully dewatered and the Conveyance Channel will not be subject to flood loading until after construction is complete. The critical global stability case for the CCL with respect to the borrow pit occurs at the end of construction when the CCL crown is at the final overbuild elevation. The DT provides minimum offset distances for the borrow pit from the toe of the protected side CCL stability berm which is governed by the critical global stability case at the end of construction. The DT considered a minimum required Factor of Safety of 1.30

for global stability of the CCL and borrow pit because this is a temporary condition during construction. The results of the analyses are presented in Table 9-25.

Table 9-25: Global Stability Results for Borrow Pit Excavation at End of Construction

SOIL REACH	CCL CROWN EL AT END OF CONSTRUCTION (NAVD 88)	WATER EL IN CHANNEL (NAVD 88)	BORROW PIT OFFSET DISTANCE FROM BERM TOE (FEET)	FACTOR OF SAFETY
7	17.7	0	30	1.32
	17.7	0	30	1.34
8	18.9	0	70	1.3
	18.9	0	80	1.32
	18.9	0	130	1.36

The DT assumed borrow pit excavations will be flooded after construction of the CCL. During the operational life of the channel, the critical global stability analysis for the CCL with respect to the borrow pit occurs when the Conveyance Channel water level is at the top of levee crown. The DT considered a minimum Factor of Safety of 1.40 for global stability of the CCL failing towards the borrow pit during highwater conditions. The results of the analyses are presented in Table 9-26.

Table 9-26: Global Stability Results for Borrow Pit Excavation during Highwater Conditions

SOIL REACH	WATER EL IN CHANNEL (NAVD 88)	FACTOR OF SAFETY
7	15.6	1.82
	15.6	1.90
8	15.6	1.72
	15.6	1.73
	15.6	1.83

9.16.6 Seepage Analyses

Seepage analyses were performed for the CCL using Lane's Weighted Creep Ratio (LWCR). Soil Design Reaches 7 and 8 were considered due to the presence of silt at approximately EL -15 for Reach 7 and EL -27 for Reach 8. The DT assumed water at the top of the project grade of the levee, EL 15.6, for the DT's analyses and analyzed multiple seepage paths through the foundation subsoils. Recommended minimum values for LWCR are 3 for clayey soils and 8.5 for silt[10]. Blanket theory performed in accordance with **Appendix** B of EM 1110-2-1913, Design and Construction of Levees and DIVR 1110-1-400, was used where the computed LWCR did not meet the recommended minimum values. Hydraulic conductivity data gathered from the previous exploration for ML soils was used to develop parameters for blanket theory analysis. The results of the seepage analyses indicate that the CCL meets the required factors of safety for seepage and minimum recommended LWCR values. Seepage calculations are included in **Appendix G**. Refined seepage analyses using blanket theory will be performed for the next phase and will include additional soil data collected during the field exploration.

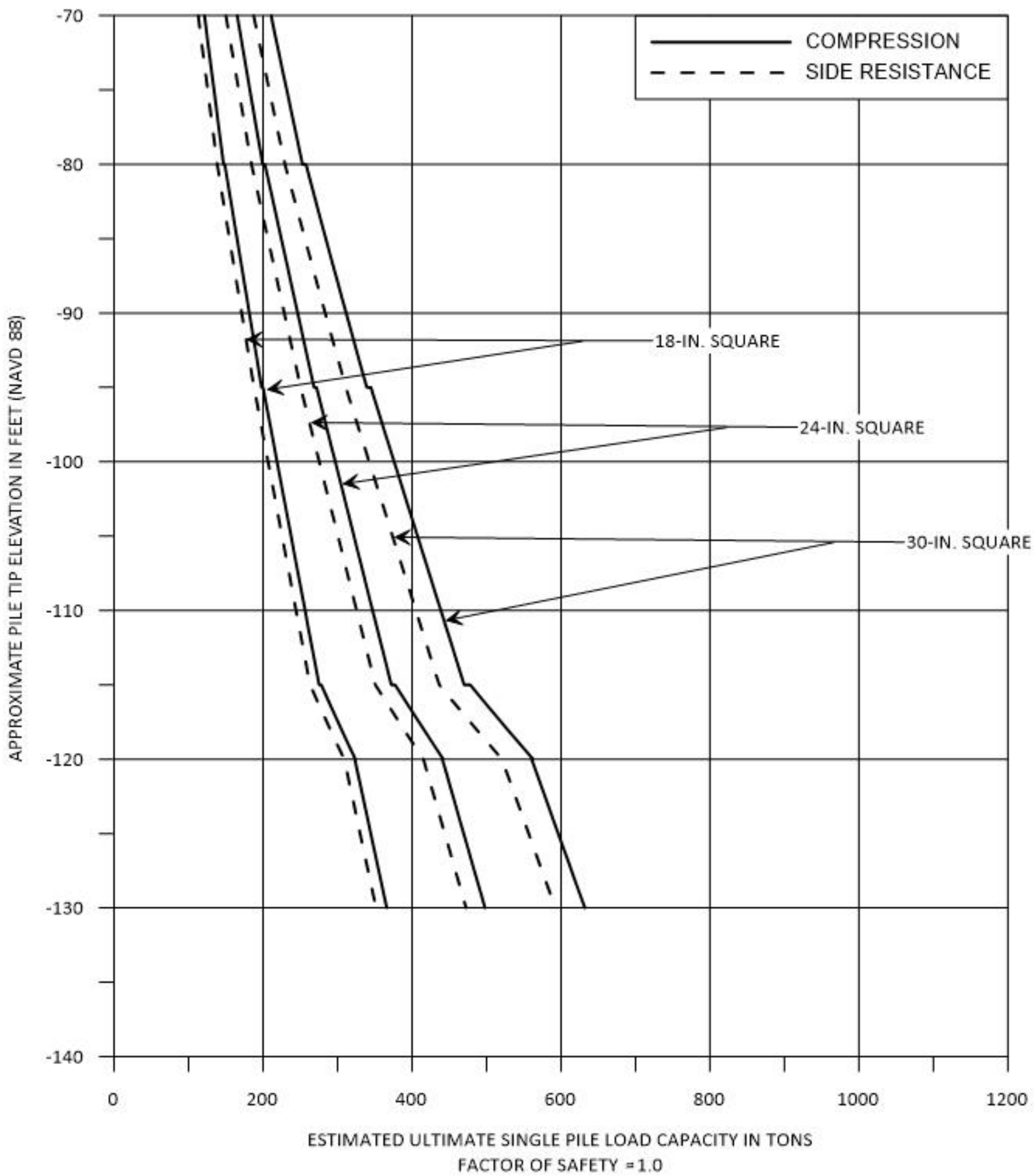
[10] Lane, E. W., "Security from Under-seepage: Masonry Dams on Earth Foundations," Trans. Am. Soc. Civil Eng., vol. 100, p. 1257, 1935

9.17 Hwy 23 Bridge

The Hwy 23 Bridge will be located at approximate Station 65+00. The bridge will span the guide levees and Conveyance Channel. Currently 16 bents spaced on 128-foot centers are envisioned for support of the bridge. We anticipate abutment settlement will require pile support transition slabs to limit settlement. Design and construction of the bridge will conform to standard requirements of the Louisiana Department of Transportation and Development. The bridge lies entirely in generalized Soil Reach 7 and parameters developed for this reach as described in the Delineation of Soil Parameters Report were used for analyses.

9.17.1 Piles Capacities

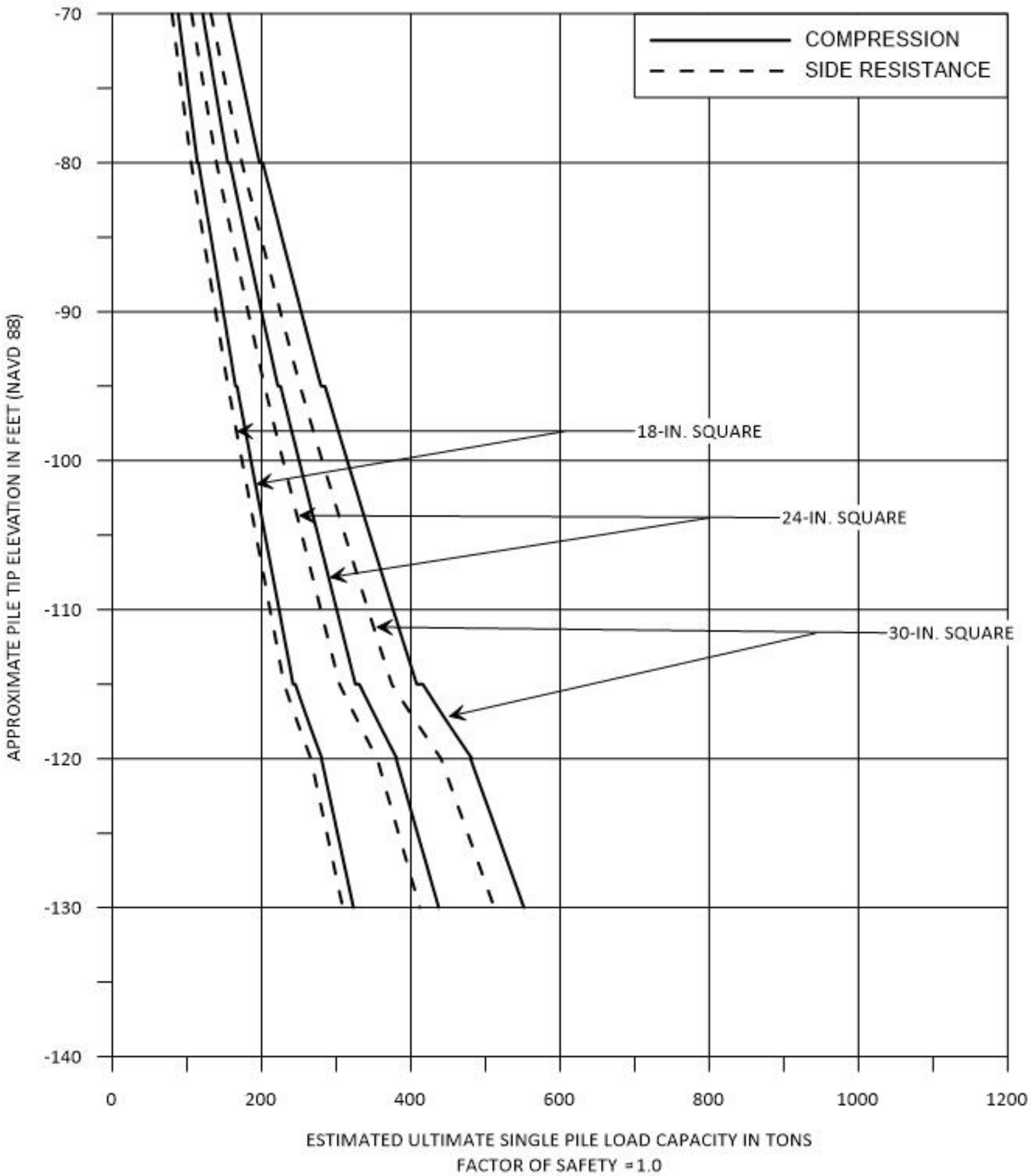
We computed ultimate pile load capacities for the bridge using methods outlined in Section 3.4.3 of the Project Design Criteria using LRFD procedures. Specifically, we used the computer program DrivenPiles 1.3.6. Factors of safety will be established considering the laboratory test data, static pile test program, and dynamic tests performed during construction as indicated by the LRFD requirements. Unfactored capacities were computed for 18, 24, and 30-inch square precast concrete (SPC) piles for support of the bridge structure and treated ASTM D25 timber piles for the transition approach slabs. Pile capacities for SPC piles are shown on Figure 9-5 for piles located at existing grade (el 0) and on Figure 9-6 for piles located in the Conveyance Channel at EL -25. Table 9-28 tabulates timber pile capacities for the abutments at EL 0.0. The calculations are presented in **Appendix G**.



NOTES:

1. ULTIMATE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.
2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.
3. ULTIMATE PILE LOAD CAPACITIES COMPUTED USING DRIVENPILES 1.3.6 SOFTWARE.
4. APPROPRIATE LRFD RESISTANCE FACTORS SHOULD BE APPLIED TO THE ULTIMATE PILE CAPACITIES.
5. ULTIMATE PILE LOAD CAPACITIES DO NOT ACCOUNT FOR SCOUR.

Figure 9-5: Hwy 23 Bridge Ultimate Pile Load Capacities for SPC Piles with Ground Surface at EL 0.0



NOTES:

1. ULTIMATE PILE LOAD CAPACITIES PRESENTED ON THIS FIGURE DO NOT INCLUDE THE WEIGHT OF THE PILES.
2. PILES ASSUMED TO BE INSTALLED BY IMPACT DRIVING EQUIPMENT WITHOUT ASSISTANCE FROM VIBRATORY EQUIPMENT.
3. ULTIMATE PILE LOAD CAPACITIES COMPUTED USING DRIVENPILES 1.3.6 SOFTWARE.
4. APPROPRIATE LRFD RESISTANCE FACTORS SHOULD BE APPLIED TO THE ULTIMATE PILE CAPACITIES.
5. ULTIMATE PILE LOAD CAPACITIES ON THIS FIGURE DO NOT ACCOUNT FOR SCOUR.

Figure 9-6: Hwy 23 Bridge Ultimate Pile Load Capacities for SPC Piles with Ground Surface at EL -25

Table 9-27: Hwy 23 Ultimate Pile Load Capacities for Treated Timber Piles with Ground Surface at EL 0

TAPERED PILE DIAMETERS	PILE TIP EMBEDMENT BELOW GROUND SURFACE IN FEET (1) (2)	PILE TIP EL (NAVD 88)	ESTIMATED ULTIMATE SINGLE PILE LOAD CAPACITIES IN TONS (3) (4) (5) (6) (7)	
			COMPRESSION	SIDE RESISTANCE
8-Inch Tip 12-Inch Butt Timber	30	-30	16 ½	16
7-Inch Tip 12-Inch Butt Timber	40 50	-40 -50	22 ½ 29	22 29 ½
7-Inch Tip 13-Inch Butt Timber	60 70	-60 -70	39 48 ½	38 47 ½

Notes:
1. Selection of pile tip embedment should also consider settlement potential.
2. Ground surface assumed to be at EL 0.0.
3. These estimated capacities do not include limitations on structural capacity as imposed by some building codes.
4. Piles assumed to be installed by impact driving equipment without assistance from vibratory equipment.
5. Ultimate pile capacities computed using DrivenPiles 1.3.6 software.
6. Appropriate LRFD resistance factors should be applied to the ultimate pile capacities.
7. Ultimate pile load capacities do not account for scour.

9.17.2 Scour requirements

Scour protection will be provided throughout the Conveyance Channel, between the top of the channel and the levee toe, and up the levee slope. Therefore, scour was not considered for the piles located within the confines of this scour protection.

9.17.3 Abutment Settlement

Preliminary calculations estimate settlement at the Conveyance Channel edge of the transition ramps to be 12 to 16 inches. The results of these settlement calculations are included in **Appendix G**. Therefore, we will consider utilizing a pile supported transition slab, a preload surcharge, or a combination of these options. A surcharge will likely be a viable, economical option because wick drains will be used for the guide levees in this area.

9.17.4 Pavement Recommendations

The pavement section for Hwy 23 is anticipated to be 2 inches of Superpave asphaltic concrete wearing course on 2 inches of Superpave binder course, with 9 inches of asphaltic concrete base course on 12 inches of Class II base course. Shoulders are anticipated to be 4 inches of Superpave asphaltic concrete. Pavement components for the north and south haul roads and levee access roads will be 2 inches of Superpave asphaltic concrete wearing course on 2 inches of Superpave asphaltic concrete binder course. Embankments for Hwy 23, the bridge ramps, the levee access roads, and the haul roads will meet the material and construction standards describe in Section 203 of the Louisiana Standard Specifications for Roads and Bridges (LSSRB), 2016 Edition.

9.17.5 T-Wall Design

The T-Walls at the Hwy 23 Bridge will consist of five monoliths (T-1 through T-5) on each side of the Conveyance Channel at Station 65+00. The ground surface on the protected is at EL 0.0 and the ground surface on the flood side is at EL 3. Top of wall grade is EL 15.6. Braced excavations will be used to construct the T-Walls and these stability analyses will be performed after the T-Wall construction sequence is developed with the contractor.

The DT performed stability and seepage analyses for the T-Wall monoliths with the flood side water level at EL 15.6 to evaluate unbalanced loads and required sheetpile tip elevations using methods outlined in Section 3.5.11 of the Project DCD using design parameters for Soil Reach 7. A summary of the stability and seepage results performed for the T-Walls in Table 9-28 and the supporting calculations are provided in **Appendix G**.

Table 9-28: Stability and Seepage Results for Hwy 23 T-Walls

Monolith No(s).	Protected / Flood Side Ground Surface EL (NAVD 88)	Base Width (feet)	Bottom of Base Design EL [with 2-foot Working Pad] (NAVD 88)	Required Sheetpile Tip EL for Seepage (NAVD 88)	Stability Factor of Safety	Unbalanced Load (lbs)
T-1 to T-5	0 / 3	15	-10	-37	1.66	0

The DT computed allowable pile load capacities for the Hwy 23 T-Walls using methods outlined in Section 3.4.3 of the Project DCD. The DT analyzed various sizes of open end pipe piles with the top of piles at EL 10 using design parameters for Soil Reach 7. Estimates of allowable pile load capacities and supporting calculations in **Appendix G**.

9.18 Conveyance Channel Guide Levee Closures of Canals

The final configuration of the CCL will require closures of Timber Canal at approximate Station 113+50 and the NOV Back-levee Canal at approximate Station 140+00. The NOV Back-levee Canal has an invert elevation at approximate EL -10 which is deeper than the Timber Canal invert at approximate EL -7. The DT performed stability analyses for the critical case at the NOV Back-levee Canal.

9.18.1 Settlement Considerations

The CCL at the canal closures will have similar cross-section dimensions as the Soil Reach 8 CCL. Therefore, the settlement analyses provided for the CCL at Soil Reach 8 are considered applicable for the canal closures. Detailed settlement analyses considering the filling of the canals will be performed for the next phase and will include additional soil data collected during the field exploration.

9.18.2 Stability

The DT performed stability analysis of the Conveyance Channel to evaluate potential modes of failure and establish critical water levels in the channel. All stability analyses use undrained shear strength (Q-case) parameters for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices. This method satisfies moment/force equilibrium and include non-circular and circular searches. Analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical circular and non-circular slip surfaces were optimized, and tension cracks filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. Stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The DT evaluated stability of the levee into the canal considering the canal invert at EL -10. The canal closure requires the placement of 300 feet of geosynthetic reinforcement placed beneath a sand working pad. The DT recommended a woven geosynthetic with a minimum tensile strength of 5,000 lbs/ft at 5% strain. The DT also recommended 200-ft wide sand working pad be placed at the levee centerline with the top of sand at EL -3. The DT assumed uncompacted clay fill will be placed beyond the sand fill in the canal to EL -3. Uncompacted clay fill placed to EL -3 is required to extend an additional 100 feet beyond the protected side stability berm to meet a minimum Factor of Safety of 1.30 during construction. Strength gain parameters computed for the Soil Reach 8 CCL and the deformed shape of the levee based on settlement results for the Soil Reach 8 CCL were modeled in the analyses. Results of the stability analyses in Table 9-29 and calculations are provided in **Appendix G**.

Table 9-29: Stability Results of Canal Closure Levee Section during Construction

Description of Analyses	Water Level in Conveyance Channel (NAVD88)	Minimum Factor of Safety Computed
Levee failure towards flood side at LWL	0	1.31
Bank failure towards flood side at LWL	0	1.34
Levee failure towards protected side with water at top of levee	18.9	1.34
Levee failure towards protected side with water at levee design grade	15.6	1.41

The DT anticipates strength gain induced by the final levee lift will increase stability factors of safety to the minimum value required during operations and flood events. Refined analyses will be performed during the next stage of the project using soil data obtained during the field exploration.

9.18.3 Seepage Considerations

The canal closure levee cross-section will have a sand core at the levee bottom for the canal closure case. The DT performed seepage analyses for the CCL at Soil Reach 8 which are presented in Section 9-16. Based on the Soil Reach 8 CCL seepage results, the DT does not anticipate an underseepage issue due to the addition of the sand core in the levee section. Detailed seepage analyses using blanket theory will be performed for the next phase and will include additional soil data collected during the field exploration.

9.19 Siphon

The inverted Siphon at Timber Canal will be located at Station 113+50. Construction of the inverted Siphon will precede construction of the Conveyance Channel levee to maintain water flow of Timber Canal during construction of the Conveyance Channel levee.

9.19.1 TRS Requirements for Installation

The DT performed analyses to evaluate temporary braced excavations for the pipes at approximate Station 113+50 using design parameters for Soil Reach 8. The DT assumed the Conveyance Channel excavation will be completed prior to performing excavations for the Siphon pipes. The stability berms at the toe of the Conveyance Channel excavation should remain in place during the installation of the sheet piling for the retaining system. The DT's analyses were performed in accordance with Section 3.5.8 of the DCD using the software CWALSHT from the US Army Corps of Engineers Waterways Experiment Station Information Technology Laboratory version date 2003/05/02. To limit sheet pile lengths to 90 feet, the DT used anchor supports and degrading near the Siphon excavation. Eustis presents the results of the analyses in Table 9-31 and the calculations are in **Appendix G**.

Table 9-30: Results of Temporary Retaining Structure for Siphon Excavation

Anchored Condition	Top of Wall EL (NAVD 88)	Ground Surface EL (NAVD 88)	Excavation EL (NAVD 88)	Required Sheetpile Tip EL (NAVD88)	Anchor EL (NAVD 88)	Anchor Force (Lbs/ft)	Maximum Moment (Lbs-ft/ft)
Anchored	-15	-15	-33	-94	-17	14,853	172,050
Anchored	-25	-25	-42	-81	-25	7,766	65,715
Cantilever	-25	-25	-42	-112	N/A	N/A	418,810

The anchored case with the ground surface at EL -15 is applicable from the edge of the Conveyance Channel to the intake and outlet structures of the Siphon and requires a 12-foot cut to EL -15 to reduce the height of retained soil. The cut to EL -15 should extend 50 feet from the sheetpile wall then slope to the existing ground surface at a 1V:7H slope. The anchored and cantilever cases the ground surface at EL -25 are applicable within the Conveyance Channel excavated to EL -25. The DT does not anticipate seepage being an issue for the TRS because the required sheetpile tip elevations are deeper than interbedded silt layer at Soil Reach 8. A detailed seepage assessment of the excavation and TRS will be performed for the next phase of the project and will include additional soil data collected during the field exploration. The DT recommends hot-rolled sheet piles be specified for seepage control through the interlocks.

The structural engineer should review the estimated anchor force and maximum applied moment when selecting a sheet pile section. The DT considered one anchor location for the TRS analyses for the BOD Phase of the project. The DT will refine the analyses for the next phase to include multiple anchor points which may reduce the required tip elevation, anchor forces, and applied moment for the TRS sheet piles.

9.19.2 Settlement at Guide Levees

The DT evaluated settlement of the Siphon Intake Structure and pipe located beneath the Conveyance Channel levee assuming the Intake Structure is supported by 70-ft long timber piles. The Conveyance Channel settlement analyses for Reach 8 was used for evaluation of the Siphon structures. The DT estimated pile downdrag settlement at the channel side of the pile-supported Intake Structure to be more than 4 feet and differential settlement across the Intake Structure to be more than 3 feet. The DT estimated settlement of the pipe beneath the Conveyance Channel levee to be approximately 12 feet due to the subsoils consolidating from the load induced by the levee. The DT will provide settlement calculations for the Siphon structures in **Appendix G**.

The DT recommends implementing a T-Wall system near the Siphon structure due to the large settlement estimates of the Siphon structures induced by construction of the levee. In addition, void spaces may develop beneath the Intake Structure due to near surface soils consolidating beneath a pile-supported structure. A T-Wall system will reduce settlement of the Siphon structures to a tolerable level.

9.19.3 Pile Capacities

The DT computed allowable pile load capacities for the Siphon using methods outlined in Section 3.4.3 of the Project DCD. The DT analyzed various sizes of Class B timber piles with the top of pile at EL -11. Estimates have been provided of allowable pile load capacities and supporting calculations in **Appendix G**.

9.19.4 Seepage Assessment

The DT performed seepage analyses for the proposed T-Wall sheetpile cutoff using LWCR to provide a minimum sheet pile tip elevation for seepage cutoff. The DT assumed flood water at the top of the T-Wall at EL 15.6 and tail water at EL -10 within the Siphon Intake and outlet structures and considered multiple seepage paths through the foundation subsoils. The DT recommends a minimum sheetpile tip at EL -40 to provide a minimum LWCR of 3 and adequate seepage cutoff of the potential interbedded silt between EL -27 and EL -33 encountered in CPT NL-1C. The DT will provide seepage calculations in **Appendix G**. The DT will perform detailed blanket theory analyses for the next phase of the project and will include additional soil data collected during the field exploration.

9.20 Outfall Transition Feature

9.20.1 General

The Outfall Transition Feature or Outfall Channel is considered the area on the basin side of the existing NOV 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.

9.20.2 Stability of Excavated Slopes

The DT performed stability analyses of the Outfall Channel to evaluate the potential failure of the channel excavation. All stability analyses use undrained shear strength (Q-case) conditions for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices, which satisfies moment and force equilibriums. The DT's analyses include non-circular and circular searches. These analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical slip surfaces were optimized, and tension crack lines filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. Porewater pressures were defined using a piezometric line in the Slope/W program. Seepage forces and transient porewater pressures were not considered for these analyses. The DT's stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The DT evaluated the stability of the Outfall Channel slope into the Conveyance Channel with invert at EL -25. The DT analyses considered water levels within the excavation and Conveyance Channel at EL -25 to simulate the in-the-dry excavation of the channel and water at EL -3 (existing grade in the Outfall Area) to simulate the in-the-wet condition of the channel. The DT developed an allowable slope of 1V:10H for the Outfall Channel. The results of the analyses are presented in Table 9-31.

Table 9-31: Outfall Channel Stability Results

Outfall Channel Slopes	Water Level in Conveyance Channel (NAVD88)	Minimum Computed Factor of Safety	Required Factor of Safety
1V:10H	-25	1.40	1.30 (During Construction)
	-3	4.00	1.40 (During Operation)

The DT will refine the analyses to model the proposed rip-rap base of the channel for the next phase of the project and will include additional soil data collected during the field exploration program. The DT anticipates that future analyses will yield adequate factors of safety based on the BOD Phase results. Calculations for the Outfall Channel stability are presented in **Appendix G**.

The DT performed stability analyses of the Outfall Channel to evaluate the potential failure of the channel excavation. All stability analyses use undrained shear strength (Q-case) conditions for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices, which satisfies moment and force equilibriums. The DT's analyses include non-circular and circular searches. These analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical slip surfaces were optimized, and tension crack lines filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. Porewater pressures were defined using a piezometric line in the Slope/W program. Seepage forces and transient porewater pressures were not considered for these analyses. The DT's stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The DT evaluated the stability of the Outfall Channel slope into the Conveyance Channel with invert at EL -25. The DT's analyses considered water levels within the excavation and Conveyance Channel at EL -25 to simulate the in-the-dry excavation of the channel and water at EL -3 (existing grade in the Outfall Area) to simulate the in-the-wet condition of the channel. The DT developed an allowable slope of 1V:10H for the Outfall Channel. The results of the analyses are presented in Table 9-32.

Table 9-32: Outfall Channel Stability Results

Outfall Channel Slopes	Water Level in Conveyance Channel	Minimum Computed Factor of Safety	Required Factor of Safety
1V:10H	-25	1.40	1.30 (During Construction)
	-3	4.00	1.40 (During Operation)

The analyses to model the proposed rip-rap base of the channel for the next phase of the project and will include additional soil data collected during the field exploration program will be refined. It is anticipate that future analyses will yield adequate factors of safety based on the BOD Phase results. Calculations for the outfall channel stability are presented in **Appendix G**.

Stability analyses of the Outfall Channel to evaluate the potential failure of the channel excavation was performed. All stability analyses use undrained shear strength (Q-case) conditions for cohesive materials and drained (S-Case) parameters for cohesionless materials following Spencer's Method of Slices, which satisfies moment and force equilibriums. The analyses include non-circular and circular searches. These analyses were performed with GEO-SLOPE International, Ltd.'s program SLOPE/W 2016, Version 8.16, using root-method searches (i.e., non-linear algorithms). All critical slip surfaces were optimized, and tension crack lines filled with water were modeled to eliminate negative interslice forces (i.e., base and shear) and negative normal forces at the base of individual slices. Porewater pressures were defined using a piezometric line in the Slope/W program. Seepage forces and transient porewater pressures were not considered for these analyses. Stability analyses were performed in accordance with Section 3.5.1 of the Design Criteria Report.

The stability of the Outfall Channel slope into the Conveyance Channel with invert at EL -25. Analyses considered water levels within the excavation and Conveyance Channel at EL -25 to simulate the in-the-dry excavation of the channel and water at EL -3 (existing grade in the Outfall Area) to simulate the in-the-wet condition of the channel will be evaluated. An allowable slope of 1V:10H for the Outfall Channel was developed. The results of analyses are presented in Table 9-33.

Table 9-33: Outfall Channel Stability Results

Outfall Channel Slopes	Water Level in Conveyance Channel	Minimum Computed Factor of Safety	Required Factor of Safety
1V:10H	-25	1.40	1.30 (During Construction)
	-3	4.00	1.40 (During Operation)

Analyses to model the proposed riprap base of the channel for the next phase of the project will be refined and will include additional soil data collected during the field exploration program. Future analyses will yield adequate factors of safety based on the BOD Phase results will be anticipated. Calculations for the outfall channel stability are presented in **Appendix G**.

9.21 Development of Erodibility Flume Testing Program

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 engaged Prof. Kehui Xu, PhD of LSU because of his local expertise in this research area. The DT has begun discussions with Professor Xu to develop a flume test program that is versatile and mobile. We are considering topics such as 1) reviewing the adequacy of existing borings planned in the basin (Borings OF-1 to OF-6) for use in the testing; 2) considering soil erodibility of existing, in-situ basin sediments versus sediments that will be deposited once the MBSD project is operational; and 3) developing a SedFlume testing apparatus that covers the appropriate range of flows and corresponding shear stresses. This flume testing program will be used for MBSD and could potentially be a resource for other diversion projects.

9.22 Risk of Faulting

The DT reviewed a document published by members of the New Orleans Geological Society that discusses the presence of faulting in the vicinity of the MBSD project area, notably the Ironton Fault. According to this document, “Episodic or slow fault creep may occur without the induction by or the creation of seismicity”. Due to the limited evidence based on historical mapping, this document advocates for 3D seismic surveys or high resolution 2D imaging to refine our understanding of faults. According to the researchers, additional seismic mapping will enhance our ability to quantify the risk associated with faulting on the planned infrastructure for the MBSD project, in addition to a better understanding of the subsidence impacts in the outfall basin. The DT is currently in communication with several potential SMEs and will engage one SME to serve as the principal investigator for this task in the 30% design. Another goal of the testing program is to establish a correlation between the flume test results and standard geotechnical testing (moisture content, Atterberg limits, sieve/hydrometer tests, undrained shear strength testing) such that these standard test results can be used as a proxy for basin erodibility.

10. STRUCTURAL ENGINEERING OF HYDRAULIC STRUCTURES

10.1 General

The DT performed a preliminary design of many structural components identified in the 2014 Basis of Design provided by CPRA for the BOD Phase. The individual Design Team members, their design tasks, and status of those tasks are provided in Table 10-1 below. Alternatives beyond the 2014 BOD were developed in an attempt to further improve sediment delivery, address possible cost savings (construction and life-cycle), and adjust to concurrent hydraulic modeling. Throughout the BOD Phase, designs were evaluated jointly by the DT and CPRA to focus attention on specific alternatives and put others on hold.

Table 10-1: Structural DT, Tasks and Status

Structure	Design Firm	Status
Intake Structure In-the-dry	WSP	Selected and progressing
Intake Structures In the-wet	WSP	On hold per Workshop No. 2
Intake Training Walls	AECOM	Selected and progressing
Gated Diversion Structure	AECOM	Tainter gated selected per Diversion Gate Study, structure progressing
MRL Tie-in Walls	TBS	EL 16.4 primary, EL 20.1 alternative
Transition Walls	TBS	EL 15.6 progressing, EL 12.1 alternative
Floodwalls at Hwy 23	TBS	EL 15.6 progressing, EL 12.1 alternative
Siphon	Principal	Selected and progressing
Back Structure	GISE	Eliminated at Workshop No. 1
Pump Station at Bayou Chenier		Eliminated by Siphon Alternative based on Cost.

10.2 References and Publications

All references used are listed in Section 5.1 of the Project DCD (**Appendix U**).

10.3 Design Approach

Hydraulic structures are designed in accordance with USACE Engineering Manuals and HSDRRS design guidance. The Headworks, all River tie-in options, floodwalls and the Siphon are all considered to be hydraulic structures. LRFD methods are applied to concrete structures in accordance with ACI 318-14 and load factors and detailing are per EM 1110-2-2104. The use of the EM criteria, as stated in Paragraph 3.6 of the EM 2104, precludes the need to check crack control. However, to assure durability, in the next design phase tension stresses shall be calculated to assure cracking is minimal. LRFD steel design is in accordance with AISC Manual of Steel Construction, 14th Ed., and load factors are per ETL 1110-2-584. All hydraulic structures are pile founded. At this time pile designs use the ASD method; however, future designs may use a LRFD process. The DT has chosen a small number of load cases to examine for each structure's preliminary design based on engineering judgement of typical governing conditions.

It is anticipated that the structure geometry may be revised to meet future requirements and changes to design water elevations but the structure type and design approach likely will not change.

Concept designs for Interim Structures are provided by the DT but final designs shall be the responsibility of the CMAR. All CMAR designs shall be reviewed by the DT.

10.4 River Intake Designs

10.4.1 Intake U-Frame Design Alternatives

The intake structures are designed to guide sediment flows into the diversion. Of the four alternative designs investigated, the open channel exhibited the best hydraulic properties. The Open Channel option consists of two primary structures. The primary structure is a reinforced concrete U-Frame with invert at EL -40 and a top-of-wall EL 16.4. This U-shaped structure extends from the Gated Intake into the river approximately 550 feet. The second feature is a set of flared training walls that continue towards the river centerline. These walls are inverted pile-founded T-Walls that step up in elevation gradually to follow the contour of the MRL. An alternative with Top Wall elevations increased to EL 20.1 was included for just the Open Channel Intake. This change only affected structures that form the line of protection along the River.

The selection of this structure was driven primarily by the hydraulic characteristics of the intake geometry and bolstered by lower anticipated costs, adaptability of the system, and the robustness of the structure. Because the selected intake does not extend significantly into the Mississippi River, it is likely that the site can be dewatered within a cofferdam (See Section 9) and therefore, conventional in-the-dry construction methods can be used.

10.4.2 Codes and Standards

See **Appendix U** – DCD, for Codes and standards used on the MBSD Project.

10.4.3 Options Investigated

Several options were evaluated from a conceptual standpoint. Prior to this BOD phase the dimensions and elevation of the intake were hydraulically unproven, the type of construction was unknown, and the viability of constructing a cofferdam that could extend a significant distance into the Mississippi River was in question. As such, conceptual options were evaluated for a variety of construction methodologies, sizes, configurations and elevations. The four primary concepts investigated included the following:

- Open Channel at EL -40: This is the chosen option described above.
- U-Frame at EL -40 With Interior Walls: In this option the U-Frame is constructed to the existing EL -40 contour in the riverbed, resulting in a significantly longer primary structure. The 1,150 foot long intake channel was divided into three bays of equal width and stepped down the wall height towards the river end of the structure.
- U-Frame at EL -40 Without Interior Walls: This option matches the previous U-Frame geometry but excludes the center walls (only one center bay).
- Submerged Culvert at EL -40: This option is a 1,150 foot long closed culvert extending from the river opening to the gate structure. The culvert geometry varies in height and width to maintain a constant opening area; the height at the river end was limited by navigation clearance concerns.

For the primary options summarized above, in-the-wet construction methods were evaluated for the U-Frame and the Submerged Culvert. The driver for evaluating the in-the-wet methods was the viability of constructing and dewatering a cofferdam extending far into the Mississippi River. For the Open Channel Option in-the-wet was not considered because the required cofferdam did not protrude significantly out into the River. It has since been determined that constructing and dewatering a cofferdam sufficient for the in-the-dry construction of the Open Approach is viable, the training walls that extend riverward will be constructed within a braced excavation or use the lift-in method of construction.

Variations of the above were modeled and structural plans developed. Other intake options evaluated included more rudimentary, pro-rated designs for the following:

- Open Channel at EL -50
- U-Frame at EL -20 without interior walls
- Submerged Culvert at EL -50
- Open Channel at EL-20
- Open Approach at EL -40 with top of wall at EL 20.1

10.4.4 In-the-dry Methodology

When a cofferdam can be properly constructed and dewatered without excessive difficulty, permitting issues, negative impact on navigation, or possible increases in flood levels, it is believed that in-the-dry construction methods will provide the most cost-effective and robust intake structure. The design and construction are conventional and there is a very knowledgeable labor force able to perform the work. Beyond the cofferdam construction the work will primarily consist of pile driving and reinforced concrete construction. Protective structures will also be required to prevent vessel impact to the cofferdam.

10.4.4.1 Critical Loadings

There are three primary loading cases that will govern the design of the in-the-dry structures, summarized in the table below.

Table 10-2: In-the-Dry Methodology Critical Loading Stages Summary

Construction Stage/Load Case	Design Considerations
Construction	• Mass concrete applications and thermal crack control • Maximum pile loads due to the absence of uplift effects
Service	• Hydrostatic forces from riverine and hurricane flood events • Ship / Barge / Debris impact • Abrasion of flowing water • Backfill soil loading • Scour of foundation soils
Maintenance Dewatering	• Unbalanced hydrostatic and backfill forces • Uplift due to buoyancy

In sections that are to be dewatered it should be noted that the structure will essentially transform from a net downward loading with relatively balanced water pressure to a net upward loading with high unbalanced water pressure that cause tensile forces on piles and upward moments on structural members. As such, the final design will carefully evaluate the necessity of dewatering the structure and which parts of the structure require dewatering. For the preliminary designs it was assumed that for the Open Channel and U-Frame the sections inboard of the MRL would need to be dewatered while those outboard would not. For the submerged culvert, it was assumed that the entire structure would need to be dewatered.

10.4.4.2 Cofferdam

A large cellular cofferdam would be required for U-Frame or Submerged Culvert construction extending out into the river. Discussion of this retaining system is found in Section 9.12. Structural items that require design beyond the cofferdam itself are dolphins and protection cells to keep vessels away from the structure and a combiwall tie-in to the MRL.

The Open Channel option also requires a cellular cofferdam but it will enclose a significantly smaller area and protrude less into the river. The training walls can be constructed within a braced excavation (described in **Section 10.4.4.4.2**) or built using lift-in construction.

10.4.4.3 Toe Sheeting

The possible requirement of additional seepage cutoff protection within the cofferdam system is described in **Section 9.12**.

10.4.4.4 Open Channel Alternative

The Open Channel in-the-dry alternative uses a constant invert elevation at EL -40 and a consistent top of wall elevation at EL 16.4 for the U-Frame reach and stepped elevations for the Training T-Walls. The Training walls are not part of the flood protection system. The river end monoliths will be submerged.

The wall and slab thicknesses are governed by shear and flexural requirements in accordance with EM 2104 ; the pile design is governed by the construction case (axial compression) and the dewatered state (axial tension). In the dewatered state, the pile connections are required to resist uplift, which is achieved using pile embedment with tension hooks rebar configurations. A minimum pile embedment of one-foot is recommended so that there is sufficient tolerance in the top of pile elevation. While bending stress in the piles is not expected to govern the pile design, the pile stresses will be checked during final design assuming a fixed and pinned head condition. The potential lateral movement of the structure under any lateral loading will be checked assuming a pinned pile connection.

High capacity 48-inch diameter pipe piles, which derive their capacity from a combination of end bearing and skin friction, are used as the current foundation scheme for the Open Channel alternative. For the 15% level design, downdrag is not assumed to act on the piles but shall be included once the final grades are established. Additional pile types shall be investigated once the CMAR is under contract and the results of the recent soil borings are available.

Because the walls are cantilevered and resist relatively high loading, 3 layers of reinforcement are required in the current design. The U-Frame structures are currently designed to resist the dewatered condition, this load case governs wall design. The pile foundation was laid out to control the level of reinforcement in the base.
The primary benefits of this option are the use of conventional construction methods, length of piling and volume of concrete is significantly lower than other alternatives, and the option for maintenance

dewatering provides an increase in overall structure robustness and the possibility of increased longevity.

10.4.4.4.1 Training Wall Design

Inverted T-shaped retaining walls are designed for the Open Channel alternative using in-the-dry construction methods and channel EL -40. These walls are not floodwalls, rather, their purpose is to guide water towards the intake structure and restrict riverbank soils from filling in the channel. Two cross-sections are analyzed: one near the intake structure and one near the river end of the alignment. These two cross-sections are a representative sample of the highest and lowest backfill levels. Wall sections in between are interpolated between these two extremes. The primary load case for this level of design is an extreme high soil condition where siltation is assumed to have occurred behind the wall (on top of what is already the high soil side). The level of silting is set equal to the Normal River Stage of EL 5 as it seems possible for silt to build up to this level under normal operation conditions.

Thirty-six (36) inch diameter x ½ inch wall steel pipe piles are analyzed using CPGA with no allowable overstress. Both the pinned and fixed connection conditions are examined to ensure geotechnical pile capacity, structural pile capacity, and overall monolith deflection are within acceptable limits. Pile batters are limited to 6V:1H or steeper for large diameter piles to ensure piles do not interfere with the surrounding cofferdam system.

Concrete walls and slabs are sized for shear and moment forces. Reinforcement is not fully designed in this level of design but member size is checked to ensure required flexural reinforcement does not exceed the USACE limit of $0.25\rho_{bal}$ set in EM 1110-2-2104. The load case is considered Usual as per EM 1110-2-2104 resulting in an applied uniform load factor of 2.2. As per the Design Criteria, K_0 for all horizontal soil loads including the accumulated silt is 0.95. To reduce the amount of concrete used the wall stems are tapered, the walls are spot-checked at multiple elevations to ensure the taper does not result in a localized overstress.

10.4.4.4.2 Training Wall Cofferdam Design

Beyond the limits of the U-Frame structure the walls are constructed in isolated, braced excavations. The width of the braced excavation shall be sufficient to permit the driving of battered piles. It is assumed that the piles shall be driven in the wet with sufficient water pressure to eliminate heave. Prior to dewatering the braced excavation a seal slab will be placed to counter heave and minimize seepage. The seal slab shall also serve as a work platform. A cofferdam system will be built around the proposed open channel tie-in walls to dewater the area during construction. Two rectangular cells will be built with one around each wall alignment in order to limit the area being dewatered. The assumed construction process of both cofferdams is as follows:

1) AZ-46 sheets will be driven to approximately EL -64.
2) Soil areas within the cells will be excavated with water to remain inside the cells.
3) Piles are driven using the braced excavation as a template
4) A slurry seal will be installed at the bottom of the excavation to cut off seepage into the cell.
5) Water will be pumped out in stages. As the water is removed, W27x161 wales and 24-inch diameter x ½-in wall pipe struts will be installed. In total, there will be three vertical layers of waters and struts in each cofferdam.

The system has been designed using the soil information from Reach 3 and for an external top of soil elevation of EL -20. The water level outside of the cell is set at EL 8. It is assumed that the cell is dewatered to just below each strut level before the waler and strut is installed. The end monoliths could also use precast, lift-in units and avoid the braced excavation. The T-Wall base footprint would be excavated and piles driven in the low water season. A short sheet pile wall maybe required to minimize siltation of the prepared excavation.

10.4.4.5 U-Frame Alternative

The U-Frame in-the-dry alternative uses step-down walls ranging in top elevation from EL -20 at the outboard end to EL 16.4 at the Gated Structure. The top-of-wall elevations are determined by hydraulics analysis and consider the potential for vessel impact. Top-of-wall elevations can be adjusted during final design without significant additional structural analysis or study. The table below summarizes the sections that are assumed to be dewatered for maintenance. The design and purpose of the walls do not change except that the outboard sections are not designed for the dewatering load condition.

Table 10-3: Maintenance Dewatering Assumptions

Station From	Station To	Top of Wall EL	Design Assumption
22+10	24+20	EL -20	Not dewatered
24+20	26+30	EL -6	Not dewatered
26+30	28+40	EL 8	Not dewatered
28+40	33+50	EL 16.4	Dewatered

The design of the walls and invert of the outboard sections is governed by shear and flexural capacity by the soil backfill and structure self-weight. For the inboard sections that will be dewatered, their design is nearly identical to that of the Open Channel except for sections that encroach significantly beyond the MRL may need to be analyzed for scour in their final condition if not armored. A scour protection study will need to be performed during future design stages if this option is decided upon. The pile design is governed by the construction case where the structure is in-the-dry and not yet subjected to submerged buoyancy forces. The scour analysis may impact the length of the embedded piles but is not anticipated to change the loading on the base.

The piles are 48 inch diameter steel piles that derive their capacity from a combination of end bearing and skin friction are shown as the current foundation scheme for the U-Frame alternative. For the 15% level design, downdrag is not assumed to act on the piles. Alternative piles shall be investigated when new soil boring results are available and the CMAR is under contract.

The primary benefits of this option are the use of conventional construction methods and the ability to dewater the majority of the intake structure.

10.4.4.6 Submerged Culvert Alternative

The Submerged Culvert Alternative has a consistent EL -40 invert and constantly tapering roof and walls. Prior to performing the hydraulic modeling for this structure it was presumed that it would yield superior sediment-to-water ratios than the open approach methods and was therefore considered even though the roof would add weight and some construction complication. However, as outlined in Section 3, the Submerged Culvert did not present the expected sediment transport results and has a risk of becoming clogged. For these reasons it is excluded from further study. That being said, the concept was advanced to a level where a layout was created and approximate quantities could be estimated.

The outboard end width of the U-Frame is approximately 300 feet, which results in roof and floor spans of approximately 100 feet. This span length is infeasible when considering a dewatered condition that causes large hogging moments and shear forces on the spans. In order to make the spans work, it would need to be assumed that either no maintenance dewatering would occur or that intermediate walls would be acceptable from a hydraulics standpoint. It was determined that neither of these options are acceptable because there is a high risk of clogging, dewatering should be accounted for and the intermediate walls add an unacceptable amount of head loss to the system. Interior walls were included in the BODR to account for additional cost increases.

The following table summarizes the stations where additional intermediate walls would be required. These assumed wall layouts are used in the quantity calculations for this alternative.

Table 10-4: Submerged Culvert Layout Summary

Station From	Station To	No. Interior Walls	Overall Width (ft)	Interior Height (ft)
22+10	24+90	5	314 to 270	20 to 27
24+90	27+80	5	270 to 227	27 to 34
27+80	30+65	2	227 to 184	34 to 41
30+65	33+50	2	184 to 140	41 to 48

Beyond the negative aspects described previously, the wall and slab layout in the previous table show that significantly more concrete will be needed for this option. More concrete results in more weight, and therefore increases the foundation demand. Similar to the other options, a pile foundation consisting of 48 inch diameter steel piles will be used. For the 15% level design, downdrag is not assumed to act on the piles.

One difference in loading that the Submerged Culvert will experience is that of the soil backfill over the tunnel required for the MRL and the addition of a rail loading on top of the culvert. The train loading is not analyzed in this design phase for the Submerged Culvert but from preliminary analyses it should be assumed that minor structural modifications could accommodate this loading. If this alternative is revived in later design stages, the train loading should be assessed and accounted for.

10.4.5 In-the-Wet Methodology

In-the-wet construction methods provide an alternative means of construction. The deep riverward cofferdam can be avoided by constructing the concrete intake structure in a graving site and then floating the monoliths into place and submerging them on pre-driven piles. For the most outward elements that cannot be floated due to their limited wall height, they can be cast off-site, towed into place on a barge, and picked up and placed with a barge-mounted crane. Details and considerations for these methods and alternatives are explained below. ACI 350 and ACI 357 shall be used to design the float in sections. As such, crack control will most likely govern the base design.

10.4.5.1 Critical Loadings

The in-the-wet method requires that the monoliths be buoyant during transport, which will require the interior of each element be dry (i.e., to have a “bathtub” configuration). During this load case there will be unbalanced hydrostatic pressure on the outside of the element, causing significant hogging moment and shear forces on the invert, roof and exterior walls. Similarly, because there are such drastic differences in loading conditions during fabrication and transport compared to the final in-service condition, each construction stage needs to be carefully coordinated with the CMAR to ensure that each stage has a load case developed for which the elements can be designed accordingly. The following table presents several of the expected governing load cases.

Table 10-5: In-the-Wet Methodology Critical Loading Stages Summary

Construction Stage/Load Case	Design Considerations
Construction	• Mass concrete applications and thermal crack control • Stripping strength and creep of concrete after stripping
Transport	• Dynamic loading from waves and transports • Design of appurtenances for towing, lifting and temporary works • Buoyancy and draft • Water-tightness of bulkhead and temporary walls
Immersion	• Effect of bulkhead connections on structure • Maximum unbalanced hydrostatic forces on structure and longitudinal transference • Negative buoyancy and ballasting • Precision of placement and tolerances • Water-tightness of bulkhead and temporary walls
Joining	• End frame planar tolerances • Unbalanced hydrostatic force transferred longitudinally • Water-tightness of bulkhead and temporary walls • Water-tightness of gina-type seal
Service	• Hydrostatic forces from riverine and hurricane flood events • Ship / Barge / Debris impact • Abrasion of flowing water • Backfill soil loading • Scour of foundation soils • Maximum axial pile compression
Maintenance Dewatering	• Unbalanced hydrostatic and backfill forces • Uplift due to buoyancy

10.4.5.2 Flotation, Transport and Ballasting

After casting and installation of the temporary walls and bulkheads, the floating monoliths be buoyed with a draft sufficient to safely clear the shallowest water along the transport route and in the graving site. In addition, ballast (typically in the form of water tanks) will be required to immerse the elements by making them slightly negatively buoyant. With a slight negative buoyancy, the elements can be lowered and maneuvered into a precise location in a controlled manner. The Culvert and U-Frames can be made into a buoyant “bathtub” using end bulkheads that seal the water out during float-in and immersion. Their design is explained in subsequent sections.

For the U-Frame methods that have step-down walls, the use of temporary walls on lower wall sections (i.e., with final top of wall elevations below the construction WSE) is an economical way to place the majority of the U-Frame sections using the Floating Monolith concept. The Floating Monolith concept utilizes large pre-cast sections and only minimal in-water work, which should provide a robust means of delivering a quality product in challenging conditions. Where the wall sections are so low that the temporary walls become burdensome and costly (in this design, below EL -6), drop-in modular sections barged into place and assembled in-place will be used.

As noted previously, the condition where elements are immersed to their final location yet still dewatered inside is likely to govern many of the structural designs. In this condition, there is significant unbalanced hydrostatic force on the outside of the structure and virtually no interior pressure to counteract the force. For the purposes of flotation, ballasting, and buoyancy, the unit weights considered should be conservative enough such that a slight variation in concrete unit weight or in-situ water unit weight will not affect the floating, transport, and immersion process. Therefore, for example, the water unit weights for the ballast calculation required for immersion should consider mud-laden water with a higher unit weight, while the calculation of draft should consider relatively fresh water with a low unit weight. Field investigations should be undertaken during final design to confirm both concrete and water unit weights as shown in the following table:

Table 10-6: Unit Weights for Buoyancy

Material	Minimum Unit Weight (pcf)	Maximum Unit Weight (pcf)
Concrete	142	154
River Water	62.4	64.5

A careful accounting of weight balance including all structural steel and concrete as well as any temporary construction appurtenances (i.e. push-pull knees, bollards, etc.) shall be kept during final design and construction so that transport draft and ballasting needs can be accurately calculated. A multi-beam sonar survey of the final location and transport route should be performed prior to construction to confirm that a minimum clear depth of at least 2 to 3 feet can be maintained and that no debris accumulation will impede the transport, immersion and placement of the elements.

Although the transport distance will be relatively short from the graving site to the final location, towing and transport forces will be accounted for in the design and sufficient freeboard will be provided so that any waves, wakes or turbulence will not pose undue risk to the structures. The minimum recommended draft and freeboard requirements are tabulated below.

Table 10-7: Transportation Requirements for Floating Structures

Variable	Minimum Draft/Freeboard (ft)
Controlled Waters Draft Clearance	2.0
Uncontrolled Waters Draft Clearance	4.0
Calm Water Freeboard (U-Frame)	4.0
Calm Water Freeboard (Culvert)	2.0
Intermittent Wave Height	2.0
Unbalanced Water Height	4.0

In its final in-service condition the structures will be immersed in water with interior and exterior water levels essentially even. Designing for buoyancy in the final condition will only be necessary where the structure is expected to be dewatered. In this case, the pile-to-structure interface should be designed in accordance with EM 1110-2-2102 where the connection provides adequate resistance against the tensile forces induced by buoyancy. Conversely, for parts of the structure not designed to be dewatered, negative buoyancy during construction can be provided by ballast or through the pile-to-structure connection. During the construction condition, the Factor of Safety against buoyancy shall be as follows:

Table 10-8: Buoyancy Safety Factors

Condition	Minimum Factor of Safety
Ballast and Gravity Loads Only	1.05
Ballast plus Structural Connections	1.10

Based on the selected configuration of the graving site, transport channel, and structure there is a potential that some amount of secondary pour concrete will be required. In this case, the structural elements and buoyancy will need to be checked at each stage of concrete placement to ensure sufficient buoyancy, structural integrity and draft.

The floating structures will be moved into placed with either a custom-built catamaran barge or using flotation tanks, barges, and anchored lines. The design of these features shall be performed by the selected contractor and coordinated with the DT. The final structural design will need to accommodate the contractor’s means and methods during final float-in.

10.4.5.3 Temporary Walls

As previously noted, some shorter structures will require temporary walls to create the “bathtub” buoyancy effect required for flotation. These walls need to be essentially watertight, with performance characteristics similar to that of a sheet pile cofferdam.

In the current U-Frame configuration, the float-in construction accounts for approximately 80 percent of the intake structure length. Approximately 210 feet of that will require temporary walls. The following table presents a summary of the floating sections and their temporary walls.

Table 10-9: Temporary Wall Requirements

Section	Station From	Station To	Temp Wall Height (ft)
F-1	33+50	30+95	n/a
F-2	30+95	28+40	n/a
F-3	28+40	26+30	n/a
F-4	26+30	24+20	12

The temporary walls consist of steel plates bolted and welded to vertical W-sections which are in turn connected to the concrete monoliths using cast-in-place studs. Studs are designed for shear and tension/compression from the overturning moment. Pullout resistance of the studs dictate a thicker top of wall than would otherwise be required. The current 15% level concrete monolith design includes loads induced by the temporary walls.

The walls will be built concurrent with the concrete monoliths and will be removed after final placement. The removal and demolition of these walls can either use divers or mechanized equipment to cut or otherwise detach (i.e. through a bolted connection) the wall just above the permanent structure and lift out each wall section. Temporary walls are currently designed to EL 6, which should be confirmed during final design based on the time of year and water levels at the anticipated floating, transport and immersion time.

10.4.5.4 End Bulkheads

The end bulkheads, while a temporary structure, are a critical item of the float-in system because they close in the "bathtub" and are a relatively major work item. Bulkhead walls have historically been constructed with both concrete and steel designs. For the depth and height of these hydraulic structures, a steel wall will likely provide a more economical solution that is also easier to demolish and remove.

Due to the large horizontal and vertical spans, significant bracing and stiffening of the bulkheads is required. The current concept uses steel plates connected to vertical stiffeners and horizontal walers that transfer the hydrostatic loads to rows of vertical and horizontal kickers connected to the concrete monolith. The bulkheads are provided with access doors to allow passage into the annular spaces between bulkheads to check the Gina compression prior to final concrete placement.

It is standard practice that at least two bulkheads are in place at all times between the river and any construction activity being done within the buoyant structure. With less than two bulkheads in place in front of workers, limited operations should take place. Because the last bulkhead will be directly against the river in the final condition, it is envisioned that this bulkhead could be demolished by divers when the structure is flooded or a floating-type gate similar to those used in dry-dock operations could be used. This detail has not been designed at this time.

10.4.5.5 Connections

At this time, it is envisioned that the elements will be connected using a Gina-type seal between elements. Using this method, the Gina-type seal on one element is mated against a plain steel end frame of an adjacent element. An initial seal is made, at which point the space between elements, bounded by the Gina-type seal, can be dewatered causing an unbalanced water pressure that pushes the elements together. From this point, the structural connection between elements can be made in-the-dry with rebar couplers and a second concrete pour. This will allow the connections between elements to be as smooth as a finished concrete surface with limited imperfections or grade differences that could cause head loss or sediment build-up. If it is determined from a hydraulic perspective that the connections between elements do not require this level of precision, the elements could simply be connected using tremie concrete. This will be determined at a later date by the DT after hydraulic modeling is complete and with input from the CMAR.

10.4.5.6 U-Frame Alternative

The U-Frame Alternative in-the-wet has similar dimensions to that of the in-the-dry alternative except that temporary walls are required and the entire floating structure is designed to resist hogging moments and shears caused by the unbalanced external water pressure. In addition, only steel pipe piles are considered since they will be driven through the water column with a follower or underwater hammer instead of constructed in-the-dry.

The most outboard sections of the structure will need to be constructed using modular lift-in methods since the wall elevation is too low to practically design and build temporary walls for float-in and immersion. The following table presents the limits of the float-in construction vs. the lift-in construction.

Table 10-10: Proposed In-the-Wet Construction Methodology

Section	Station From	Station To	Construction Methodology
F-1	33+50	30+95	Float-in
F-2	30+95	28+40	Float-in
F-3	28+40	26+30	Float-in
F-4	26+30	24+20	Float-in
L-1	24+20	23+50	Lift-in
L-2	23+50	22+80	Lift-in
L-3	22+80	22+10	Lift-in

One potential lift-in sequence involves casting the wall sections off-site in the lengths shown above, barging them into place, and setting them on pre-driven piles. The lengths of the walls are defined herein assuming a maximum pick weight of 2,000 tons. In between the lift-in walls the slab could be either lift-in as well or could be constructed using tremie methods with the reinforcement cage placed underwater onto a screeded gravel bed. If this method is revived during later design stages, the CMAR should provide input on their desired methods of slab construction.

10.4.5.7 Submerged Culvert Alternative

The Submerged Culvert alternative is not feasible in that some elements cannot be designed for buoyant transport and immersion without significant allowances and accommodations. When the culvert concept was broached, the outboard dimensions were not known and it was considered feasible. Since that time it has been determined that for outboard elements to float, they would need to use lightweight concrete and additional flotation devices such as barges attached structurally to the elements. The inboard sections could potentially be floated into place but the outboard sections are where the in-the-wet methodology has the most tangible benefits. Therefore, without significant dimensional modifications, this alternative is not recommended for further study.

10.4.6 Intake Armoring

10.4.6.1 General

Armoring analysis for the intake channel commenced after Workshop No. 2 selection of the Open Cut Intake configuration, constructed in the wet, to EL -40. For armoring analysis, this single intake alternative was considered. To proportion the armoring, 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.

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)

“Classic” design of channel armoring can be simplified for discussion as follows:

1) Project cross sections, geometry, and limits are established
2) Hydraulic analysis provides water velocities or tractive forces
3) Riprap stone or ACB depth is sized

4) Riprap gradation or ACB dimension is established
5) Layer thickness is established (riprap)
6) Filter layer gradations/thicknesses are selected
7) Transitions and special features detailed
8) Iterate/adjust design as necessary

10.4.6.2 Pre-Analysis Alternative Screening

In order to eliminate unnecessary effort, armoring alternatives were qualitatively screened in the context of MBSD project conditions. Feasible armoring alternatives for screening were judged to be stone riprap and articulated concrete blocks or mattresses (ACBs or ACMs). Feasible filter layer alternatives for screening were judged to be geotextile fabric (fixed to underside of ACB), fines contained within compartmented flexible mattresses, and coarser-graded loose filter material. Geotextile filter or loose filter material underlying riprap, constructed in-the-wet, was not considered constructible. Given the in-the-wet construction method selected for the intake channel, practical armoring placement considerations were rapidly found to dominate. These considerations, which alter the approach from classic armoring design, are as follows:

- Reliable placement of light riprap in flowing water, at the MBSD project depths (up to 45-50 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.
- Placement of fine material at MBSD project depths could potentially be achieved with a clamshell lowered to river bottom prior to opening, or with slurry pumping through a tremie pipe and diffuser. If technically feasible, these methods may prove tedious when wide coverage is required, and favorable economics must be established prior to implementation.
- Predicted Mississippi River current velocity during construction will strongly influence the choice of armoring scheme and details of design. Currents may alter the un-armored banks between dredging and armoring, and may disperse any insufficiently heavy intermediate filter or stone material prior to final armoring layer placement.
- While the USACE District successfully places ACMs as revetment on the Mississippi River bank, and even places irregular “pocket” revetment ACMs successfully, this placement uses specially constructed barges for such a purpose. To date, no similar case history demonstrating successful placement of commercially available ACB matting, with interlocked segments comparable to the District ACMs, placed in comparable water depth, has been discovered. The HDR Report suggests that for an ACB alternative to be successful, use of divers would be required to guarantee coverage and to interlock the commercial ACB segments while underwater; and that such a method would be difficult and expensive.

The USACE District approach to Mississippi River bank revetment in this region, exhibited in the Myrtle Grove revetment surrounding the intake channel, is to place anchored and interlocked ACMs without a filter layer directly on the underlying bank, and tolerate subsequent local movement resulting from scour between individual ACM block units. A similar approach is applied to rock dikes and riprap, where heavy stone is placed directly on river banks. 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.

Riprap armoring alone was chosen to advance for analysis and proportioning in this 15% BOD stage. Until conclusion of the next design stage, the ACB constructability and case history investigation is recommended to remain open, as satisfactory methods may demonstrate ACBs a viable alternative to riprap.

Filter layers, rigorously proportioned by particle diameters according to EM 1110-2-2300, were eliminated from further analysis during 15% BOD due to uncertain constructability, and due to the feasible alternative approach employed by the USACE District as described above. An underlying stone layer of 4-inch diameter screening would be a minimum filter or foundation material size for use with in-the-wet placement from the surface. Similarly to ACBs, investigation of construction methodology facilitating cost effective and satisfactory filter placement is recommended to remain open through the next design stage. Rigorously proportioned filter layers, if feasible, may permit reduced stone layer thicknesses and improve durability of the intake channel armoring, reducing construction and O&M cost, respectively.

Following selection of riprap and assuming elimination of filter, the armoring design process, simplified for discussion, generally becomes:

1) Project cross sections, geometry, and limits are established
2) Hydraulic analysis provides water velocities or tractive forces
3) Riprap stone is sized for tractive forces
4) Riprap stone size validated/adjusted against practical in-the-wet minimum
5) Riprap gradation is established
6) Layer thickness is established from EM method
7) Layer thickness is adjusted based on EM and local judgement for O&M serviceability
8) Transitions and special features detailed
9) Iterate/adjust design as necessary

Should ACBs and filter stone layers prove to be constructible alternatives at competitive cost prior to final selection of an armoring design, the process will be modified.

10.4.6.3 Armor Stone Sizing

A color-coded depth-average velocity figure was provided from hydraulic model output for a 75,000 cfs diversion channel flow during Mississippi River discharge at 1,000,000 cfs. This figure visually displays the velocities at various locations along the intake channel and within the river cross-section in the area where the diversion currents approach the intake channel. This model represents only the case used for intake screening, and not the entire design envelope; additional model outputs across the full envelope of flows will be provided as completed. The controlling case for armoring is anticipated to be the Corps of Engineers design discharge, referred to as the Mississippi River and Tributaries Project Flood discharge, for the Mississippi River at the proposed location of the intake channel, a value of 1,250,000 cfs.

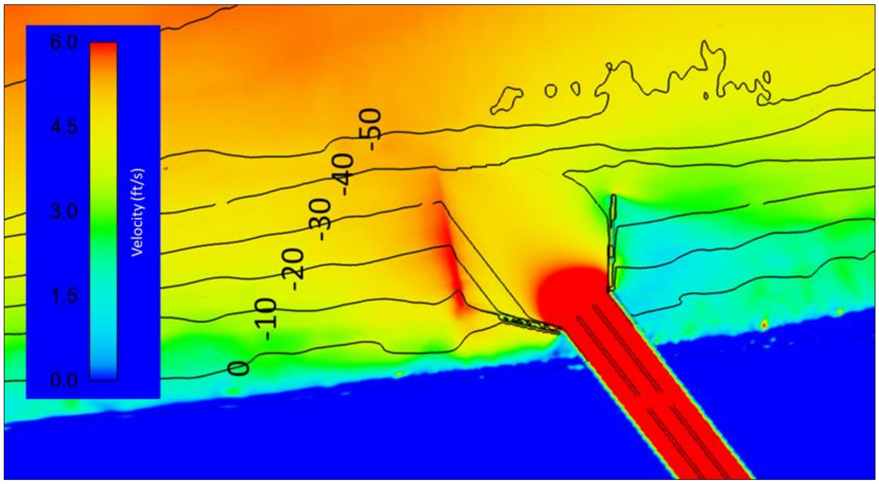


Figure 10-1: Depth-Average Velocity Distribution, 75k cfs MBSD / 1.0M cfs MR, from Intake Alternative Screening Model

The hydraulic model output revealed highest velocities where the currents veer towards the southwest to enter the intake channel from the river, and where the cross sectional area constricts into the intake structure. For highest design confidence in the armoring solution at late design stages, depth-average velocity figures should be calculated and shown to the maximum level of detail possible so that high localized velocities, particularly where the currents veer towards the southwest to enter the intake channel from the river, in excess of the average flow velocity, can be detected.

For this analysis, the maximum modeled value of velocity, exhibited at the northern edge of intake channel cut slope, and in the proximity of the U-frame entrance, will be used to select a single size of armoring stone. The value read is 6.0 ft/s (red coloration in the figure).

According to EM 1110-2-1601, from selected armoring design velocity, the minimum W_{50} of a gradation required to resist the tractive force was determined using the graph on Plate B-29. See clipped and annotated Plate in the figure following.

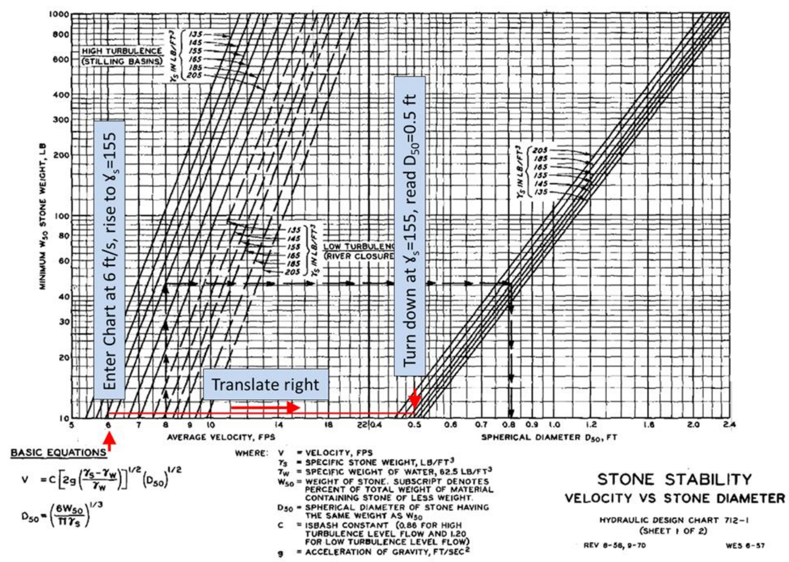


Figure 10-2: Stone D_{50} Selection by Depth-Average Velocity

The 6.0 ft/s modeled velocity, at a selected stone unit weight of 155 lb/ft^3, yields D_{50}=0.5ft (W_{50}=10.2 lb). From the USACE Lower Mississippi Valley Division (LMVD) Report on Standard riprap gradations appended to EM 1110-2-1601 as page F-18, the lightest commonly produced gradation with W_{50} no less than a calculated 10.2 lb minimum likely yields a W_{100} range of 40 – 90 lb. For cross-reference to an independently defined gradation, this roughly corresponds to the LADOTD definition of 30-lb Class Riprap. See the annotated LMVD report chart following.

LMVD 12 November 81

STANDARD RIPRAP GRADATIONS
(Design Sepcific Weight 155 pounds per cubic feet)

	GRADATION NORMALLY PRODUCED MECHANICALLY								GRADATIONS NORMALLY REQUIRNG SPECIAL HANDLING				
Layer Thickness in Inches High Turbulent Flow	12	15	18	21	24	30	36	42	48	54	63	72	81
Layer Thickness in Inches Low Turbulent Flow			12	14	16	20	24	28	32	36	42	48	54
Percent Lighter by Weight													
100	25 10	50 20	90 40	140 60	200 80	400 160	650 260	1000 400	1500 600	2200 900	3500 1400	5000 2000	7400 3000
50	10 5	20 10	40 20	60 30	80 40	160 80	280 130	430 200	650 300	930 440	1500 700	2200 1000	3100 1500
15	5 2	10 5	20 5	30 10	40 10	60 30	130 40	210 60	330 100	460 130	700 200	1100 300	1500 500

Minimum W_{50} Exceeding 10.2 lb.

Figure 10-3: Standard Riprap Gradation with W_{50} > 10.2 lb.

The gradation above was identified on the basis of the modeled 6.0 ft/s velocity generated in an early alternative screening model, and is likely to be adjusted as modeling advances.

On the Mississippi River, the USACE District uses a heavier riprap for revetment repair where the existing ACMs are disrupted, or where revetment by ACM is not feasible. The gradation is annotated in the following figure, taken from the 2015 USACE stone placement contract, and titled “Grade Stone B without Fines”.

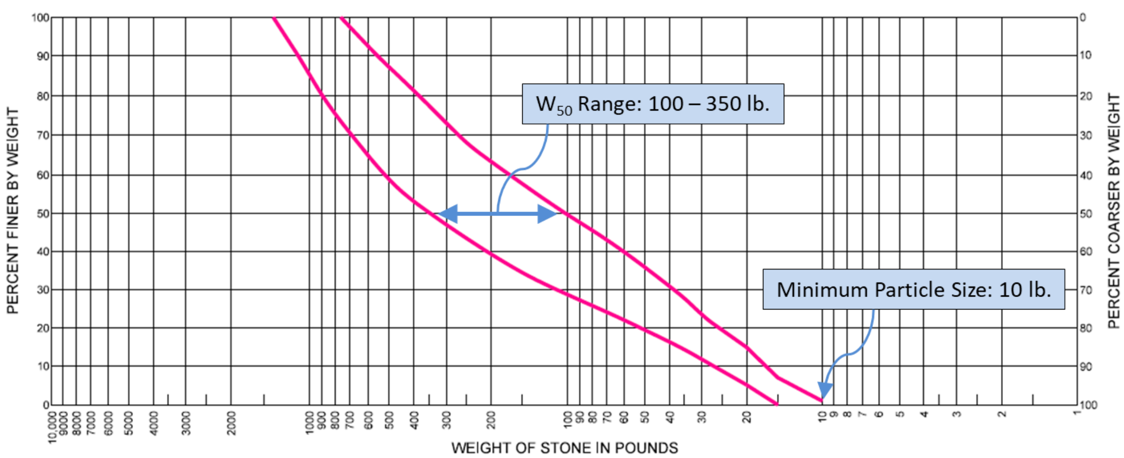


Figure 10-4: Mississippi River Revetment Riprap Gradation Specified by USACE District

The two riprap gradations resulting from a calculated W_{50}, and from USACE District specifications, are significantly different as shown in the following table.

Table 10-11: Riprap Gradation Comparison

% Lighter by Weight	Calculated by EM	Grade Stone B
W_{100}	40 – 90 lb	750 – 1200 lb
W_{50}	20 – 40 lb	100 – 350 lb
W_{15}	5 – 20 lb	20 – 35 lb

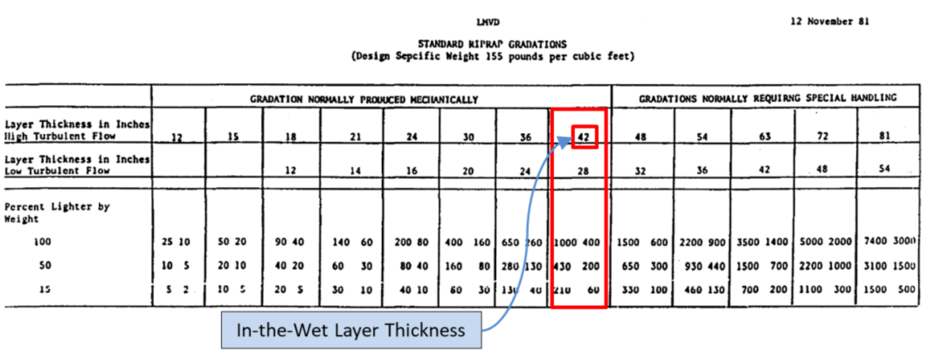
At this stage, the heavier riprap gradation, Grade Stone B (B-Stone), was selected for advancement. Lighter or differently distributed gradation(s) may be ultimately selected following more detailed hydraulic and construction method analysis. Use of B-Stone is not explicitly required by published document; however, it represents the following advantages over the calculated gradation:

- Greater placement reliability from the surface in river currents.
- Conservatism in stone gradation to account for uncertainty in localized velocities at structures, or unforeseen turbulence.
- Larger riprap gradations exhibit better stability on steep slopes, lending better revetment resistance to stone layer slump resulting from toe scour or migration of underlying fines.

10.4.6.4 Layer Thickness

By EM, the selected gradation upper limit D_{100} is 2.45 feet, from a W_{100} of 1,200 lb. 1.5 times the upper limit D_{50} is 2.45 ft (1.5 x 1.63 feet), from a W_{50} of 350 lb., which established the minimum layer thickness. For in-the-wet construction, the thickness was increased by 50% to 1.5 x 2.45 = 3.68 feet. Transverse bank slopes are modest in this vicinity, generally 1v:10h or flatter, suggesting that further adjustment of stone layer thickness for sideslope stability is not warranted; though local intake channel slopes may require, as geometry is developed, upward thickness adjustment.

Alternately, from the LMVD Standard Riprap Gradations chart in the EM, a gradation most near B-Stone was selected, with a recommended "Layer Thickness in Inches – <u>Low</u> Turbulent Flow" value of 28 inches, appropriate for in-the-dry placement. The "Layer Thickness in Inches – <u>High</u> Turbulent Flow" value of 42 inches is commonly used for in-the-wet placement in low turbulence applications such as anticipated at the MBSD. The 42 inch value (3.5 feet) is essentially equal to the 3.68 feet value calculated above.


LMVD 12 November 81

STANDARD RIPRAP GRADATIONS
(Design Sepcific Weight 155 pounds per cubic feet)

	GRADATION NORMALLY PRODUCED MECHANICALLY								GRADATIONS NORMALLY REQUIRNG SPECIAL HANDLING				
Layer Thickness in Inches High Turbulent Flow	12	15	18	21	24	30	36	42	48	54	63	72	81
Layer Thickness in Inches Low Turbulent Flow			12	14	16	20	24	28	32	36	42	48	54
Percent Lighter by Weight													
100	25 10	50 20	90 40	140 60	200 80	400 160	650 260	1000 400	1500 600	2200 900	3500 1400	5000 2000	7400 3000
50	10 5	20 10	40 20	60 30	80 40	160 80	280 130	430 200	650 300	930 440	1500 700	2200 1000	3100 1500
15	5 2	10 5	20 5	30 10	40 10	60 30	130 40	210 60	330 100	460 130	700 200	1100 300	1500 500

Figure 10-5: Riprap Layer Thickness from LMVD Standard Gradation Chart

Construction tolerances for underwater stone layer thickness are commonly written at 6 inches over/under; however, the 12 inch vertical range may be shifted to all "over" or all "under" as required. Difficult placement conditions at grade breaks, transitions, or boundaries may warrant relaxed tolerances and/or a thicker layer; such conditions being identified as channel geometry and diversion structures are refined.

The intake channel will be constructed into the predominant fine point bar deposits. At the time of the 2013 geotechnical investigation, the bank constituent soils at the uppermost elevations of borings (project Station 20+00 to Station 25+00) are generally sands with silt, and silts, SP and ML, respectively. A single boring at Station 21+50 reflects a very soft clay deposit of 18” at the surface, overlaying the same SP layer. Surface deposits of soft clay may be dispersed or displaced by stone placement, resulting in depressed revetment surface.

The uniform layer thickness required, as of 15% BOD, was established at the minimum rounded value of 4.0 feet. For conservatism, pending further analysis, the riprap revetment has been drawn at 5.0 feet. Depending on the final construction specifications for under/over thickness tolerances, level of concern for isolated soft bank deposits, and particular geometric or hydraulic constraints, the layer thickness may be adjusted to meet scour protection or other non-armoring criteria.

Upon more detailed hydraulic analysis, review of USACE District O&M experience at the Myrtle Grove revetment, and establishment of CPRA preferred O&M approach; detailing of the following armoring features may be accomplished:

- Toe scour protection, likely as a heavily thickened launchable stone layer toward the river-most extent of the revetment. The launchable layer is located above the expected zone of attack and anticipated to fill scour voids over time, creating a trenched-in thickened toe.
- End scour protection, likely as thickened stone layer placed on top of existing ACM revetment. The required ACM/riprap lap distance required by the USACE District may be upwards of 80 ft., based on experience. Transitions between measures in erosive zones merit particular attention.
- Rock dikes of significantly greater depth than the greater 5.0 ft layer thickness, adjacent to structures, may be provided to either provide stability and toe protection or to increase bathymetric elevation following excavation for structure placement.

10.4.6.5 Further Considerations

Design will advance according the basis established above, and may incorporate the following tasks:

- Use predicted and/or measured normal Mississippi River current velocities at the location of revetment construction, at varying river stages, for the purpose of establishing threshold(s) for suspension of construction activity, by phase and/or task.
- Analyze predicted water velocities at progressive phases of construction, informed by feasible construction methods, to establish the lower limit of filter or stone particle size practical.
- Coordinate construction specification tolerances, gradation, and finished grade requirements across technical disciplines to ensure no negative impact on sediment transport or hydraulic performance is caused by intake armoring.

10.5 Gated Diversion Structure

10.5.1 Design Approach

The concrete gated structure houses steel gates which allow river water to pass through the MRL into the Conveyance Channel. Tainter gates are used based on the recommendation of the Diversion Gate Study found in **Appendix O**. The initial design layout has set the intake entrance at Station 33+50; however, several locations have also been investigated. Based on decisions made in Workshop No. 2 (see **Appendix R**), the primary design effort uses a sill elevation of EL -40 with three 45 foot wide gates.

Prorated designs for invert levels EL -20 and EL -50 have also been developed to determine approximate size and cost differences between these levels and EL -40.

Concrete monolith and steel gate designs are described below. The intake structure construction is assumed to use a conventional in-the-dry method. All intake structures will be pile founded and all concrete is traditional reinforced concrete with the exception of the tainter gate trunnion connection, which is post-tensioned.

The Diversion Gated Structure is loaded from both sides. The river side ties into the MRL and is controlled by riverine flood conditions. The design top of wall elevation based on riverine flood is currently EL 16.4. The design flood condition for the basin side is based on a 50-Year hurricane event with water EL 15.6. These are the river and basin flood elevations used in the primary intake structure design.

Although not authorized in this reach of the MRL, CPRA is considering incorporating the hurricane criterion that is used for flood protection on MR levees. For this submittal, a 50-Year hurricane event with flood EL 20.1 is used as a secondary design case. The pile foundation for the gate monolith is designed equally for EL 16.4 and 20.1. All concrete components are prorated to determine EL 20.1 quantities.

The following table is a summary of all intake options examined for this submittal:

Table 10-12: Intake Options Summary

Alternative	Sill Invert	River Side TOW	Basin Side TOW	Design Level
1	-40	16.4	15.6	Designed
2	-40	20.1	15.6	Prorated
3	-50	16.4	15.6	Prorated
4	-50	20.1	15.6	Prorated
5	-20	16.4	15.6	Prorated
6	-20	20.1	15.6	Prorated

Two alternative gate locations were investigated, one moving the structure closer to the MRL to Station 32+00, a 150 foot shift towards the river, and a second location that shifted the gate 2,100 feet towards the Basin to Station 50+00. The Station 50+00 was investigated when it was unknown if the sands at the riverward locations could be dewatered for construction. Geotechnical analysis has since proven that the sand strata can be dewatered using conventional means. The added cost and increased head loss on Conveyance eliminated the 2,100 setback from consideration. The slight move towards the river has no apparent negative impacts with respect to MRL stability construction or conveyance. The shift will be made in future designs once hydraulic conditions with partial gate openings are modeled and found to not to be detrimental. The shift will be limited to 100 feet, leaving an approximate 200-foot space between the Railroad Bridge and gated structure to allow maintenance floating enough room to operate.

10.5.2 Concrete Monolith Design Loadings

All load cases are defined in the Project Design Criteria found in **Appendix U**. The 15% designs included in this submission use only the load cases that the DT judges to be worst-case scenarios. A full analysis of all load combinations will be calculated in later design phases.

The critical load cases considered for concrete and pile designs are as follows:

- Construction: River Side water EL 0.0, Basin Side water EL 0.0
- Dewatering: River Side water EL 5, Basin Side water EL 1
- (MR flow line or flow at design SWL: River Side water EL 12.4, Basin Side water EL 1
- MR design grade (TOW): River Side water EL 16.4, Basin Side water EL 1
- Reverse MR design grade (TOW): River Side water EL 1, Basin Side water EL 15.6

Concrete design load combinations are based on EM 1110-2-2104 and are as follows:

Table 10-13: Concrete Design Load Combinations

No.	Load Case Name	Description	Factored Load Combinations	Load Category
1	Construction plus Wind	Dead, Lateral, Surcharge, Gate Equipment, Wind	1.6(D+EH+Ls+Q+W)	Unusual
2	Maintenance Dewatering, (Pervious)	Dead, Lateral, Surcharge, Gate Equipment, Pervious Cut-off	1.6(D+EH+ES+Q+Hs+Hu)	Unusual
3	Water at Design SWL or Flowline (Pervious)	Dead, Lateral, Surcharge, Gate Equipment, Pervious Cut-off	2.2(D+EH+ES+Q+Hs+Hu)	Usual
4	Water to Top of Wall (Pervious)	Dead, Lateral, Surcharge, Gate Equipment, Pervious Cut-off	1.6(D+EH+ES+Q+Hs+Hu)	Unusual
5	Reverse Water to Top of Wall (Pervious)	Dead, Lateral, Surcharge, Gate Equipment, Pervious Cut-off	1.6(D+EH+ES+Q+Hs+Hu)	Unusual

The Construction Load Case assumes vertical loads including dead load, uniformly distributed construction surcharge load over the foundation, gate weight and operation equipment load on walls. Lateral loads include wind load on walls and earth force due to an assumed 5-foot differential in bank grades.

The Maintenance Dewatering Load Case assumes vertical loads including dead load, uniformly distributed construction surcharge load over the foundation, gate weight and operation equipment load on walls in the gravity direction and uplift pressure on the slab in the opposite direction. Lateral loads include saturated earth pressure on both left and right banks, construction surcharge transmitted through the soil and hydrostatic load on the stoplogs.

The Water at Design SWL or Flowline, Water to Top of Wall and reverse Water to Top of Wall Load Cases assume vertical loads in the downward direction including dead load, water pressure on the channel slabs, gate weight (in closed position) and operation equipment load on walls. Uplift pressure is applied to the slab and is assumed to vary linearly between head pressures beneath the monolith. Lateral loads include saturated earth pressure on both left and right banks, soil surcharge and hydrostatic load on walls.

10.5.3 Pile Foundation

The pile foundation is analyzed in accordance with USACE EM 1110-2-2906. The software used is CPGA which uses the stiffness method and assumes the pile cap is rigid. More advanced software which utilizes P-Y and T-Z springs and a flexible base will be used in subsequent designs. Given the large footprint of the structure an all-vertical pile geometry is used. Downdrag effects are not considered and unbalanced loads are not present. Two pile types are considered: a 24-inch diameter pipe pile and a 36-inch diameter pipe pile. Construction cost and driving preference of the CMAR will ultimately influence the final design pile type. Because a static pile load test program will be conducted in advance of structure construction, a Factor of Safety of 2.0 is used in the design. For this submission, piles are structurally designed using the allowable stress design (ASD) method, though future structural analyses may use the load resistance factor design (LRFD) method. The pile size, pile tip elevation and pile capacities can be found in the drawings and calculations, **Appendix D** and **Appendix J**, respectively. The pile capacities are calculated using the Eustis Engineering Capacity Curves described in Section 9.

10.5.4 Finite Element Model

The Gated Structure is modeled in SAP2000 structural analysis program as a 190-foot long monolith with top of wall elevations of EL 16.4 and EL 15.6 at river and basin sides (RS & BS) respectively. The monolith includes three, 45-foot bays that are divided by 8-foot thick walls. Elevations used in this report are not final and may be subject to change. Also, construction, maintenance, equipment, gate and wind loads are estimated and may be subject to change. The wall and slab thicknesses are defined based on the shear and moment capacity of the sufficiently reinforced sections and checked with the model's results to make sure that the structure can effectively carry the lateral and vertical loads.

To model the piles in SAP2000, the pile stiffness matrix for round pinned piles derived by the CPGA program is applied in the SAP2000 model as joint springs. Though both 24-inch and 36-inch diameter piles are possible foundation options, the SAP2000 design model used assumes 36-inch piles. Compression, tension and displacement values of the piles are then evaluated and checked against the pile capacity including allowable overstresses. Piles diameter, depth and number are defined based on the pile capacity report that provides the compression and tension capacity for given pile sizes and tip elevations. In choosing the pile tip elevations, the design aims to keep the tip a sufficient distance above the weak layer that occurs around EL -130 in this reach.

10.5.5 Gate Structure Prorated

For the gate monolith, vertical and lateral forces related to TOW EL 16.4 and invert EL -40 are analyzed in spreadsheet format using forces similar to those described for the SAP2000 finite element model. The loads generated from this case are analyzed in CPGA using 36-inch diameter x ½-in wall pipe pile in a 11.75 foot spacing pile grid. The CPGA output data shows that the greatest pile forces are generated by the construction case (Load Case No. 1 in the above table) therefore, this case is used as the basis for all prorated foundation designs.

The total vertical force from the construction case is divided by the number of piles in the grid to develop an average force per pile. Next, the vertical loads of the sill EL -50 and EL -20 cases are calculated using the same spreadsheet. The total vertical force for each option is divided by the EL -40 average pile force to prorate the number of piles required to support the structure. This process is also used to determine the number of piles required when the TOW is extended to EL 20.1.

The walls of the concrete monolith are prorated using a lateral soil load from top of grade EL 4 with no water inside the channel which is the worst case condition for the cantilever walls. Thicknesses are determined by verifying the wall has adequate shear capacity and enough thickness to conform to the USACE maximum reinforcement limit of $0.25\rho_{bal}$ set in EM 1110-2-2104.

10.5.6 Gate Type and Design

Per the recommendation of the Diversion Gate Study, steel tainter gates are chosen for preliminary design and uses a top of wall EL 16.4 and sill EL -40. The analysis follows the LRFD design procedure described in ETL 1110-2-584, Design of Hydraulic Steel Structures, Appendix D Spillway Tainter Gates. Two load cases are applied for the 15% level design:

1) High River Condition:
    a. Water to Top of Gate EL 16.4 on river side (top of wall for river side)
    b. Water to EL 1.0 on basin side (lowest design elevation)
2) High Basin Condition:
    a. Water to EL 1.0 river side (lowest design elevation)
    b. Water to EL 15.6 on basin side (top of wall for Conveyance Channel)

Although these load cases are relatively simplistic, they will accurately determine the required member sizes because they represent the most extreme head differentials in each direction. Other cases that would include temporary loads such as wave, impact, or the more extreme gate friction forces would use lower water levels or would be considered an extreme limit state.

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. Load factors applied to dead and water loads conform to ETL 1110-2-584 Appendix D, Table D-1 ($1.2D + 1.4H_s$).

A 2D 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) and the ETL's recommendation of a 3/8 inch minimum thickness is followed. The ribs are analyzed as simple beams spanning between horizontal girders assuming the skin and rolled ST-shape rib section act compositely.

Horizontal girders, vertical and diagonal girder bracing, end frames, and end frame bracing is all sized using a SAP2000 3D model. 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 loads are applied to the front and back faces of the skin plate. Pinned restraints are applied at the trunnion pin and the bottom edge of the gate is supported by rollers that closely mimic how the gate will rest on the bottom seal plate (per guidance from EM 1110-2-2702, no longer in production, which provided additional details on how boundary conditions should be modeled in tainter gate design). The load cases above are input in SAP2000 with applicable load factors and the program's steel design process checked all members using the ϕ*α*nominal resistance limit.

Multiple preliminary gate designs are developed for the various alternatives described in Section 10.5.1. Because the majority of steel weight is based on primary member sizing it was decided that fully running the preliminary gate design for each option would more accurately reflect cost differences than assuming prorated steel shape sizing. Some portions of the gate such as the skin and rib sizing are the same regardless of the sill elevation because they are primarily dependent on maximum head differential and not overall water height. In total four gate options are modeled:

1) TOW & River EL 16.4, Sill EL -40
2) TOW & River EL 20.1, Sill EL -40
3) TOW & River EL 17.5, Sill EL -50 (17.5 is an earlier iteration of the max. river elevation)
4) TOW & River EL 17.5, Sill EL -20

10.5.7 Maintenance Bulkheads

Truss type bulkheads are included for both emergency and maintenance conditions. Bulkheads shall be designed in accordance with ETL 1110-2-584 with a load factor of 1.6. Two types of bulkheads shall be included. One set shall be designed with casters to allow installation in flowing waters in an emergency closure. The bulkhead dam for this emergency and maintenance condition shall be 50-foot tall. The second set shall be designed without rollers and will be installed in a steady state condition. Fracture critical members shall be designed using redundant connections where possible. Welding shall be done in accordance with AWS D1.5.

10.6 Mississippi River Levee Tie-Ins

10.6.1 Interim Line of Protection

The interim flood protection system is discussed in Geotechnical Section 9.

10.6.2 Permanent Line of Protection

10.6.2.1 In-the-Wet Tie-Ins

The U-Frame Intake Structure is enclosed on both the north and south sides with inverted T-Wall monoliths that form the MRL tie-in. Since the T-Walls are within the open excavation for the U-Frame and Gated Diversion Structure, the nearest MRL T-Walls will match their bottom elevations and step upward as they embed further into the levee.

There are three alternate U-Frame alignments based on the proposed sill options: EL -40, EL -50, and EL -20; EL -40 is the primary design and the remaining two are prorated designs. The top of wall for the primary design is set at EL 16.4 to match the current MRL Riverine Design Grade. The MRL tie-in walls are also analyzed with the top of wall at EL 20.1 to investigate the costs associated with adopting the USACE NOV Hurricane Protection 50-Year Event Design Grade. All calculated and prorated designs and their associated drawings are found in **Appendix J** and **Appendix D**, respectively.

10.6.2.1.1 Geometry

As stated previously, all alternatives start with a TOS elevation to match the adjacent U-Frame Structure and step upwards to TOS EL 0.0 before embedding in the MRL. T-Walls are backfilled with sand to EL 2 on both sides of the stem wall regardless of slab depth. All monoliths contain a PZ-22 sheet pile cutoff wall, are built on pile foundations, and terminate in a sheet pile transition which connects the T-Walls to the levee embankment.

The primary design with sill EL -40 and TOW EL 16.4 extends 290 feet from the U-Frame into the levee and is comprised of six T-Wall monoliths. Slab depths step up in 10 foot increments from T-1 at TOS EL -40 to T-5 at TOS EL 0; the T-6 slab matches that of T-5. All monoliths are 50 feet long with the exception of T-6, which is 40 feet long. The walls for monoliths T-1 through T-4 are 3 feet thick at the top and thicken at a 1:12 slope on the land side. Walls for monoliths T-5 and T-6 are a constant 3 foot thickness. Slabs vary from 32 feet wide and 7 feet thick at the lowest TOS elevation to 15 feet wide and 5 feet thick at the highest.

The EL -50 alternative extends the tie-in alignment to 350 feet and contains seven 50-foot long monoliths. Slabs again step up in 10-foot increments with the most exterior monolith, T-7, maintaining TOS EL 0.0. Walls for monoliths T-1 through T-4 are 3 feet thick at the top and thicken at a 1:12 slope on the land side and T-5 through T-7 walls are a consistent 3 feet thick. Slabs vary in size from 40 feet wide and 7 feet thick at EL -50 to 15 feet wide and 5 feet thick at EL 0.0.

The alternative for sill EL -20 is the most simplistic design. A 200-foot length of T-Wall is broken into four 50 foot monoliths with two stepped transitions. Only T-1 has a sloping wall similar to those described above and the slab measures 24 feet wide by 5 feet thick. The remaining three monoliths have consistent 3 feet thick walls and 15 feet wide by 5 feet thick slabs.

The last alternative examined matches the primary design but extends the wall height to EL 20.1. The general T-Wall layout remains the same in terms of step height and monolith length. T-1 through T-3 again have 1:12 sloped walls with 3 feet top thickness; however, these will be thicker at their bases because of the increased wall height. T-4 through T-6 walls are unchanged. The primary difference between this and the TOW EL 16.4 design is an increase in slab width and thickness because an extra row of piles is required. The largest slab is 40 feet wide and 8 feet thick but again transitions to a 15 feet by 5 feet slab.

10.6.2.1.2 Design Approach

The Sill EL -40 and TOW EL 16.4 option base slabs, foundations, and stem walls are analyzed as the primary design. From the EL -40 design, EL -50 and EL -20 alternates are prorated to determine number of piles, pile sizes, pile tips, length, wall thickness, and base slab dimensions. The T-Wall top of slab elevations start at EL -40 (or -50/-20 depending on alternative) at the U-Frame and end at EL 0.0 at the levee tie-in.

Calculations following the Ultimate Strength Design method described in ACI 318-14 and EM 1110-2-2104 are performed to determine allowable shear and flexure acting on the stem and slab of the inverted T-Wall monoliths. Serviceability requirements are not checked at this time for concrete structures; however, in using EM 1110-2-2104 criteria, serviceability requirements are met. A more comprehensive check will be performed during the next phase of the design.

Piles are designed using soil parameters found within **Appendix J**. According to the geotech information provided by Eustis, there are no unbalanced loads or significant down drag at the fill areas near levee tie-ins. A detailed analysis of downdrag and settlement will be investigated by the geotechnical team in the future phases and those results will be included in tie-in pile designs.

Analysis of the 3-dimensional structure is performed using a combination of hand calculations and Excel spread sheets. The hand calculations consider the self-weight of the T-Wall monolith, water weight and pressure, soil weight and pressure, and uplift forces. Totaling up a combination of these forces and moments, the result allows us to find the total forces and moments acting on the pile foundation. Hand calculations are also done to check the design of the stem wall and the base slab of the inverted T-Wall monolith, according to the MBSD Design Criteria factored loads for allowable shear calculations to find if the thickness of slab and stem are adequate.

10.6.2.1.3 In-the-Wet Tie-in Design Loadings

The load cases as described in the MBSD Design Criteria Table 5-5 (**Appendix U**) are the basis for the load cases evaluated in the analysis. Engineering judgment is used in selecting a limited number of preliminary design load cases by comparing the magnitude of the applied loads and the applicable Load Factor. Design resiliency checks will be evaluated in a later phase of the project. The analysis evaluated both the pervious and impervious cut-off wall uplift conditions. The following table shows the selected preliminary load cases.

Table 10-14: MRL T-Wall Concrete Design Load Combinations

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
1	Construction without soil, with surcharge	N/A	N/A	1.6(D+EH+EV+Ls)
4 & 5	Water at Design SWL	12.8	1.0	2.2(D+EH+EV+Hs+Hu)
17 & 18	Water to Top of Wall	16.4	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)
17 & 18 Alt.	Water to Top of Wall	20.1	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)
19	Maintenance Dewatering	5.0	3.0	1.6(D+EH+EV+Hs+Hu)

Notes: 1) Unbalanced loads not considered in this phase
2) 4, 17 & 19 impervious uplift, 5, 18 & 19 pervious uplift
3) DL= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Ls= Construction Surcharge, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

10.6.2.1.4 Pile Foundations

For the 15% phase the piles are designed for lateral and vertical loads only; the moment from vertical and lateral loads is not considered for analysis. During the next phase, all loads and moments will be included in the analysis for deflection and combined stress for piles. Assuming a static pile load test will be performed the allowable Factor of Safety for pile loads is 2.0. Pile tips are set to mitigate differential settlement between monoliths. Group analysis on piles is not done for this phase. Pile tips are set to mitigate differential settlement among monoliths. Group analysis on piles is not done for this phase.

All vertical and horizontal forces acting on the structure are summed for each load case. Vertical loads are assumed to be carried by all piles and lateral loads are assumed to be carried by only the piles battered against the direction of loading. This in itself is conservative as the lateral capacity provided by the opposing batters is ignored. Lateral loads are converted to axial pile forces by multiplying them by the pile batter. Once loads are distributed two maximum pile forces are calculated, one due to the lateral force and one due to the vertical. The required pile tip elevation is then found using the higher of these two values (compression, tension, or both) from the analysis results and plotting the points along the pile capacity curve for 24 inch and 30 inch diameter open-end steel pipe piles.

10.6.2.1.5 Cutoff Sheet Pile Wall

A PZ-22 sheet pile cutoff wall is included beneath all monoliths to limit seepage; the embedment criteria is specified in Section 9. Cutoff sheet pile will extend via a sheet pile transition wall into the levee embankment. The transition wall will be 30 feet long and the top of the sheet is matched with the levee crown.

10.6.2.1.6 Braced Excavations

A cofferdam (TRS) is designed for the EL -40 T-Wall monolith including the excavations for bottom and tremie seal slabs below. The sheet pile for the MRL cofferdam has a tip elevation of EL -64. The proposed width of the cofferdam is 47 feet to avoid the battered piles for the T-Wall foundations and the length is about 315 feet to provide adequate clearance around both ends of the wall alignment. The depth of the retaining systems will be reduced as the T-Wall base slab elevations go up towards the levee tie-ins. The wales and struts to support the sheet piles are sized using the ASD method found in the Steel Construction Manual (14^{th} Ed.). There is a 10-foot thick slurry seal below the 2 foot tremie seal slab to resist hydrostatic uplift and ensure a relatively dry work environment inside the cofferdam.

Cofferdams for the EL -50 and EL -20 alternatives are prorated based on the EL -40 design; a cofferdam for EL -40 with top of wall at EL 20.1 is also investigated using a similar methodology.

10.6.2.1.7 Future Analysis and Design Considerations

The following will be addressed in future submissions:

- Coordinate with the adjacent structures to identify and rectify any pile conflicts
- Coordinate and rectify interference with U-Frame and Gated Diversion Structure for selected alignments
- Evaluate all applicable load cases, including the design resiliency checks
- Verify the assumed construction sequence to determine its appropriateness
- Investigate pile foundation deformations and mitigation measures
- Determine the piles that require tension connections and design these items.
- Pile analysis using the Group pile program by Ensoft Corporation.
- Perform a more detailed design check for all of the structural members including rebar for shear and flexure
- Future Design grade (TOW EL 20.1) for hurricane is considered for this design phase as a prorated design based on TOW EL at 16.4 calculations. Actual calculations will be provided if this alternative is chosen.

10.6.2.2 In-the-Dry Tie-Ins

The U-Frame Intake Structure is enclosed on both the north and south sides with inverted T-Wall monoliths that form the MRL tie-in. The joint between the U-Frame and T-Wall monoliths will be sealed with water stops which can provide lateral movement between these two structures. The waterstop will be embedded into the U-Frame the full height of required seepage cutoff depth. For this construction method it is proposed that MRL T-Walls be built after backfilling the U-Frame and Gated Diversion Structure excavations. Unlike the MRL In-the-Wet Tie-Ins, the dry construction tie-in walls consider only one consistent TOS at EL 2. Overall two alternatives are considered, one for TOW at EL 16.4 and one for TOW at EL 20.1.

10.6.2.2.1 Geometry

Both alternatives set a TOS EL 2 and contain two identical 50-foot monoliths on either side of the U-Frame. The increase in wall height due to the changing TOW elevation does not affect any member sizing. All monoliths have a 3-foot thick uniform stem wall and a base slab that measures 15 feet wide and 5 feet thick. As with the In-the-Wet design, a PZ-22 sheet pile cutoff is located below the slab and all monoliths are pile-founded.

10.6.2.2.2 Design Approach

Calculations following the Ultimate Strength Design method described in ACI 318-14 and EM 1110-2-2104 are preformed to determine allowable shear and flexure acting on the stem and slab of the inverted T-Wall monoliths. Serviceability requirements are not checked at this time for concrete structures; however, in using EM 1110-2-2104 criteria, serviceability requirements are met. A more comprehensive check will be performed during the next phase of the design.

Piles are designed using soil parameters found within **Appendix J**. According to the geotech information provided by Eustis, there are no unbalanced loads or significant down drag at the fill areas near levee tie-ins. A detailed analysis of downdrag and settlement will be investigated by the geotechnical team in the future phases and those results will be included in tie-in pile designs.

Analysis of the 3-dimensional structure is performed using a combination of hand calculations and excel spread sheets. The hand calculations consider the self-weight of the T-Wall monolith, water weight and pressure, soil weight and pressure, and uplift forces. Totaling up a combination of these forces and moments, the result allows us to find the total forces and moments acting on the pile foundation. Hand calculations are also done to check the design of the stem wall and the base slab of the inverted T-Wall monolith, according to the MBSD Design Criteria factored loads for allowable shear calculations to find if the thickness of slab and stem are adequate.

10.6.2.2.3 In-the-Dry Tie-In Design Loadings

The load cases used for these monoliths are the same as those used for the In-the-Wet Tie-In design. See Table 10-14 for a discussion about the chosen design cases and a summary of the load combinations and applicable safety factors.

10.6.2.2.4 Pile Foundations

For the 15% phase the piles are designed for lateral and vertical loads only; the moment from vertical and lateral loads is not considered for analysis. During the next phase all loads and moments will be included in the analysis for deflection and combined stress for piles. Assuming a static pile load test will be performed the allowable Factor of Safety for pile loads is 2.0. Pile tips are set to mitigate differential settlement among monoliths. Group analysis on piles is not done for this phase.

All vertical and horizontal forces acting on the structure are summed for each load case. Vertical loads are assumed to be carried by all piles and lateral loads are assumed to be carried by only the piles battered against the direction of loading. Lateral loads are converted to axial pile forces by multiplying them by the pile batter. Once loads are distributed two maximum pile forces are calculated, one due to the lateral force and one due to the vertical. The required pile tip elevation is then found using the higher of these two values (compression, tension, or both) from the analysis results and plotting the points along the pile capacity curve for 24-inch and 30-inch diameter open-end steel pipe piles.

10.6.2.2.5 Cutoff Sheet Pile Wall

A PZ-22 sheet pile cutoff wall is included beneath all monoliths to limit seepage; the embedment criteria are specified in Section 9. Cutoff sheet pile will extend via a sheet pile transition wall into the levee embankment. The transition wall will be 30 feet long and the top of the sheet is matched with the levee crown.

10.6.2.2.6 Future Considerations

In the event this option is chosen as the alternative to be constructed, the same future considerations will be made for this design as are described for the In-the-Wet options in Section 10.6.2.1.7.

10.7 Transition Structures

10.7.1 Geometry

10.7.1.1 Transition Wing Wall In-the-Dry

The Transition T-Wall monoliths are located on both sides of the Conveyance Channel starting from the Gated Diversion Structure and span to the west. With In-the-Dry construction, there are three alternative sill elevations (based on the U-Frame channel elevations): EL -40, EL -50 and EL -20. The T-Walls on both sides of the Conveyance Channel are identical in all aspects and span from the U-Frame to the guide levee tie-ins. For all examined alternatives, the original top of wall elevation is EL 13. Top of wall elevations were increased to increase to EL 15.65 according to the 50-Year Future Hurricane Grade; however, effects of this change is not examined during the preliminary design phase. Additionally, walls were also designed using a lower level of flood protection at EL 12.1. There is no cofferdam proposed to construct the transition T-Walls; the walls will be constructed within the HW earthen cofferdam.

All alternatives contain a continuous PZ-22 sheet pile cutoff wall is beneath the monolith 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 2. An 8-foot clear roadway is also proposed regardless of alternative 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 DCD. Side mounted LADOTD guard rails are also proposed on both sides of the roadway.

10.7.1.2 Alternative EL -40

For this alternate, the lowest base slab is at EL -40 which matches the U-Frame channel elevation and the highest base slab elevation is at EL 0.0. There are fourteen T-Wall monoliths starting from base slab EL -40 (T-1) to base slab of EL 0.0 (T-14). The Conveyance Channel bottom grade slopes upward (in the west direction) from EL -40 to EL -25 over 150-feet and continues sloping up to EL 2; monoliths step up to mimic this slope. The total T-Wall length is approximately 720 feet.

The walls for monoliths T-1 through T-7 are 2 feet 6 inches thick at the top and thicken at a 1:12 slope on the land side. Monoliths T-8 through T-14 have uniform walls with a thickness of 2 feet 6 inches. The width of the base slab varies from 31 feet at the EL -40 level to 15 feet at the guide levee tie-in. Typical monolith length is 50 feet long. The T-Wall base slab thickness and width also vary according to the bottom slab elevations.

10.7.1.3 Alternative EL -50

For this alternate, the lowest base slab is at EL -50 which matches the U-Frame channel elevation and the highest base slab elevation is at EL 0.0. There are fourteen T-Wall monoliths starting from base slab of EL -50 (T-1) to base slab of EL 0.0 (T-14). The Conveyance Channel bottom grade slopes upward (in the west direction) from EL -50 to EL -25 over 250-feet and continues sloping up to EL 2; monoliths step up to mimic this slope. Total T-Wall length is approximately 700 feet.

The walls for monoliths T-1 through T-8 are 2 feet 6 inches thick at the top and thicken at a 1:12 slope on the land side. Monoliths T-9 through T-14 have uniform walls with a thickness of 2 feet 6 inches. The width of the base slab varies from 40 feet at the EL -50 level to 15 feet at EL 0. Typical monolith length is 50 feet long. The T-Wall base slab thickness and width also vary according to the bottom slab elevations.

10.7.1.4 Alternative EL -20

For this alternate, the lowest base slab is at EL -20 which matches the U-Frame channel elevation and the highest base slab elevation is at EL 0.0. There are thirteen T-Wall monoliths starting from base slab of EL -20 (T-1) to base slab of EL 0.0 (T-13). The Conveyance Channel bottom grade remains constant at EL -20 until T-6, then slopes upward to EL 2 at T-13. The total T-Wall monolith length is approximately 644 feet each side of the Conveyance Channel.

The walls for monoliths T-1 through T-7 are 2 feet 6 inches thick at the top and thicken at a 1:12 slope on the land side. Monoliths T-8 through T-13 have uniform walls with a thickness of 2 feet 6 inches. The width of the base slab varies from 24 feet at the EL -20 level to 15 feet at the EL 0.0 level. Typical monolith length is 50-feet long. The T-Wall base slab thickness and width also vary according to the bottom slab elevations.

10.7.2 Design Approach

For the 15% design phase there are three proposed alternatives for the transition T-Walls based on the U-Frame Channel EL -40, EL -50 and EL -20. The EL -40 alternate is the only alternative calculated and designed. From the EL -40 design, EL -50 and EL -20 alternates are prorated for quantities of the T-Wall including number of piles, pile sizes, pile tips, length and base slab dimensions.

For the transition T-Walls, hand calculations based on the Ultimate Strength Design using ACI 318-14 and EM 1110-2-2104 are preformed to determine allowable shear and flexure acting on the stem and slab of the inverted T-Wall monolith. Serviceability requirements are not checked at this level of design; however, in using EM1110-2-2104 criteria, serviceability requirements are met. A more comprehensive check will be performed during the next phase of the design.

The multiple sized T-Wall monoliths and pile foundations are designed based on hand calculations and Excel spreadsheets for the design of pipe piles, stem and base slab. The soil parameters provided by Eustis Engineering are used to calculate the soil pressures and pile capacities. All calculated and prorated designs and their associated drawings are found in **Appendix J** and **Appendix D**, respectively.

10.7.3 Pile Foundation

The piles are designed for lateral and vertical loads only for the 15% design phase. The moment from vertical and lateral loads is not considered for analysis. During the next phase, all the loads and moments will be included in the analysis for deflection and combined stress for piles. Assuming that a static pile load test will be performed, the allowable Factor of Safety for pile loads is 2. Group analysis on piles is not performed for the 15% design phase.

Vertical and horizontal forces acting on the structure are summed for each load case. Vertical loads are assumed to be carried by all piles and lateral loads are assumed to be carried by only the piles battered against the direction of loading. Lateral loads are converted to axial pile forces by multiplying them by the pile batter. Once loads are distributed two maximum pile forces are calculated, one due to the lateral force and one due to the vertical. The required pile tip elevation is then found using the higher of these two values (compression, tension, or both) from the analysis results and plotting the points along the pile capacity curve for 24-inch and 30-inch diameter open-end steel pipe piles.

10.7.4 T-Wall Design

Analysis of the 3-dimensional structure is performed using a combination of hand calculations and Excel spreadsheets. The hand calculations consider the self-weight of the T-Wall monolith, water weight and pressure, soil weight and pressure, and uplift forces. There are unbalanced loads shown in the geotechnical stability analysis at EL -40 to EL -25. The DT is proposing to eliminate the unbalanced loads by soil remediation with soil cement stabilization. Therefore, the unbalanced load for the 15% level design phase is not considered.

By summation of the applied forces and moments acting on the wall, the pile force and moments below the T-Wall are determined. Hand calculations are 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. Factored concrete design loads are used to confirm the adequacy of the stem wall and slab thickness.

To streamline the design process across each of the Alignments the T-Walls are designed identical. Design and analysis is performed for only the EL -40 base slab elevation and geometry for EL -50 and EL -20 alternates are prorated from the EL -40 alternate. All monoliths are pile supported with pile tips set to mitigate differential settlement among monoliths. Settlement calculations are not performed in the 15% level design but will be performed in the future phases.

10.7.4.1 Load Cases

The load cases as described in the MBSD Design Criteria Table 5-5 (**Appendix U**) are the basis for the load cases evaluated in the analysis. Engineering judgment is used in selecting a limited number of preliminary design load cases by comparing the magnitude of the applied loads and the applicable Load Factor. Design resiliency checks will be evaluated in a later phase of the project. The hydraulic grade and design grades are from the MBSD Design Criteria Table 2 and 3 of Section 2, Rev. 2, submittal draft no. 3 dated 4/27/2018. The basic load cases selected for the analysis are as stated in the table below.

The analysis evaluates both the pervious and impervious cut-off wall uplift conditions. The following table shows the selected load cases.

Table 10-15: Transition T-Wall Design Load Case Summary

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
1	Construction without soil, with surcharge	N/A	N/A	1.6(D+EH+EV+Ls)
4 & 5	Water at Design SWL	9.1	1.0	2.2(D+EH+EV+Hs+Hu)
14 & 15	Channel Low Water Reverse Head	1.0	1.0	2.2(D+EH+EV+Hs+Hu)
17 & 18	Water to Top of Wall	15.6	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)
17 & 18 Alt.	Water to Top of Wall	12.1	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)

Notes: 1) Unbalanced loads not considered in this phase
2) 4, 14 & 17 impervious uplift, 5, 15 & 18 pervious uplift
3) DL= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Ls= Construction Surcharge, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

10.7.5 Future Analysis & Design Considerations

The following will be addressed in future submissions:

- Coordinate with the adjacent structures to identify and rectify any pile conflicts.
- Coordinate and rectify interference from U Frame/Gate structures and the T-Wall for selected Alignments.
- Evaluate all applicable load cases, including the design resiliency checks.
- Verify the assumed construction sequence to determine its appropriateness. Investigate pile foundation deformations and mitigation measures.
- Perform detailed design checks for all structural members including sizing and detailing of rebar for shear and flexure.
- Determine piles that require tension connections.
- Perform pile analysis using advanced software such as Group by Ensoft Corporation. Use spring constants and structural software that account for the flexible base of the larger structures. Use pile curves based on the recent, extensive boring program. Maintain pile tips within the boring depths. ADT continues to recommend Pile Load Test on all major structure foundations to verify theoretical values.
- Perform alternative pile comparison to assure capacity and economy. Prestressed concrete piles will be considered where unbalanced loads and the effects of downdrag on battered piles are not factors.
- Future Design grade and hydraulic grade for hurricane is not considered for this design phase but will be considered in the future design phases.
- Determine piles that require moment connections.

10.7.6 Concrete Channel Base

Armoring options for the Channel are addressed in Section 11.

10.7.7 Riprap Channel Base

Armoring options for the Channel are addressed in **Section 11**.

10.8 Siphon

10.8.1 Design Approach

10.8.1.1 Structural Description and Design Criteria

The Inverted Siphon consists of three elements: the Intake Structure, the Inverted Siphon piping, and the Outlet Structure. The reinforced concrete Intake and Outlet Structures are essentially subdivided rectangular U-frame channels with partition walls subdividing the structures at each Inverted Siphon pipe. Additionally, the Intake Structure will have finger weirs for each Inverted Siphon pipe.

The Intake Structure will feature a 20 foot wide access deck across the width of the structure and steel bar screen. In a similar fashion, the Outlet Structure will have a 15 foot access deck. Both structures feature wing walls and sluice gates for each Inverted Siphon pipe. Both the Intake and Outlet Structures will be pile supported.

The Intake and Outlet Structures as rectangular U-Frame channels will be designed in accordance with EM 1110-2-2007, *Structural Design of Concrete Lined Channels,* and ACI 318-14, *Building Code Requirements for Structural Concrete.*

The Inverted Siphon piping will consist of two - 48” and four - 60” diameter reinforced concrete pipes and will be designed in accordance with EM 1110-2-2902, *Conduits, Culverts, and Pipes.* This Inverted Siphon piping configuration varies from that discussed in section 8.11.5 Conceptual Inverted Siphon Sizing by eliminating the single 48” and single 60” pipes that were added for redundancy. The redundant pipes are not shown in the drawings and were eliminated to reduce project costs by removing two lines of piping and narrowing the required excavation limits. This can be changed if the Owner decides to have redundant piping included in the project.

The pile foundations for the Intake and Outlet Structures will be designed in accordance with EM 1110-2-2906, *Design of Pile Foundations,* based on the allowable pile capacities specified by Eustis Engineering for the Inverted Siphon Headworks Structure. Tension connectors will need to be utilized on all piling for the Intake and Outlet Structures to counteract buoyance in the channel-dry maintenance condition.

10.8.1.2 Functional Characteristics

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

1. Influent stormwater flow is regulated by weirs to direct successive utilization of pipes. As water surface elevation increases in the Intake Structure, additional pipes are recruited, in order that desired minimum velocity is exceeded during the widest range of influent flow magnitudes. See Hydraulic Level Control below.

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.

5. Personnel and vehicular access is provided for operations and maintenance.

6. Operator safety and facility security are maintained.

10.8.1.3 Hydraulic Level Control

Stormwater influent flow magnitude, interior drainage basin headwater stages, intake stages, pipe number/diameter, and weir elevation increments will be iterated during the interior drainage modeling process in order to optimize pipe flow velocities, while meeting the broader interior drainage design criteria.

Intake stage increments between flow magnitudes, and individual pipe design flow in that stage increment will be used iteratively with the model to establish weir length. The sharp-crested weir equation will be used for this purpose.

Sharp-crested weir equation: $Q = \frac{2}{3} c_d \sqrt{2g} b H^{3/2}$

10.8.2 Excavation

Two alternate methods of excavation are presented for construction of the Inverted Siphon: Fully Sloped and Sloped-TRS. The Fully Sloped method utilizes a simple sloped excavation with 8H:1V side slopes. The existing grade elevation at the Inverted Siphon location is approximately EL -4. The cut would need to be excavated to minimum EL -39 at the Diversion Channel bottom. The required bottom width of the excavation is approximately 60 feet, with the current number and diameter of Inverted Siphon pipes.

The Sloped-TRS excavation method is a combination of a simple sloped excavation for the upper portion and 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 53.2 feet and the excavation would be dewatered. The design of the TRS is the responsibility of the Contractor.

The Sloped-TRS excavation method greatly reduces the amount of excavation required but with the length of the excavation and large width of the excavation to accommodate the six Inverted Siphon pipes the cost of TRS may be prohibitive making the Fully Sloped method the more economical alternate. Excavation costs will be determined, evaluated, and reviewed at the next plan stage.

10.8.3 Pipe Selection (Concrete and Steel)

With number and diameter of pipes provided by the completed interior drainage model, the Inverted Siphon piping shall be designed according to EM 1110-2-2902, as pipe through levees. Considerations include the following:

1. The alignment shall maintain minimum clear cover between diversion channel bottom and top of pipe. Five (5) foot clearance is the assumed lower limit.

2. Each individual pipe shall resist buoyant force when dewatered, during design flow of the diversion channel, by combination of pipe weight and buoyant weight of soil wedge above.

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.

Steel and concrete pipe alternates were investigated. During preliminary cost research, the estimated cost of the steel pipe alternate was found to be significantly higher than the cost for the concrete pipe alternative. Therefore, concrete pipe (AWWA C300) was selected for the Inverted Siphon piping.

10.8.4 Inverted Siphon Geometry

The Inverted Siphon profile is dictated by the Diversion Channel and levee as the Intake Structure is located at the protected side toe of the north levee and the Outlet Structure is located at the protected side toe of the south levee. The invert of the Inverted Siphon at the Intake Structure is -10. The Inverted Siphon pipe then descends at a 10H:1V slope crossing below bottom of the Diversion Channel with an invert EL -35. Once the Inverted Siphon pipe crosses the Diversion Channel it ascends at a 10H:1V slope reaching the Outlet Structure with an invert EL-10.

There are pile supported T-Walls with sheet pile cutoff providing the flood protection at the Intake and Outlet Structure locations. The steel sheet pile seepage cutoff will need to be driven prior to installation of the Inverted Siphon pipe and penetrations will be required for the Inverted Siphon piping.

In addition the location of T-Wall foundation piling, steel H-piles, will need to be coordinated with the Inverted Siphon piping to avoid damage to the Inverted Siphon piping during pile driving. Pre-drilling of the steel H-piles is recommended.

10.8.5 Headworks Design

The Intake Structure will be designed as a U-Frame channel with 20 degree wing walls at the structure’s entrance and a headwall at the end of the structure where the influent transfers to the Inverted Siphon piping. The length of the Intake Structure is 109’-6” not including wing walls. The width of the structure is 91’-10”. The height of the Intake Structure is 10 feet with top of U-Channel wall EL 0.0 and an invert EL -10.

The Intake Structure feeds four 60-inch and two 48-inch Inverted Siphon pipes. The channel is subdivided between each Inverted Siphon pipe location. All four 60” pipe subdivisions and the interior 48-inch pipe subdivision are flow controlled by finger weirs with EL -5.

Sluice gates are provided for each Inverted Siphon pipe at the headwall and are provided adjacent to the access deck at the front of the structure. The gates adjacent to the access deck are for maintenance dewatering purposes, and could be replaced with a manually-inserted stoplog or bulkhead system to reduce the O&M burden of mechanically operated gates. 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 Structure will be pile supported on timber piling and the design will look at the maintenance condition with the structure dewatered at maximum buoyancy. The piles will require tension connectors.

Sedimentation entering the Inverted Siphon piping is a major concern. While the width of the Intake Structure will slow velocities and cause some sediment to fall out prior to entering the Inverted Siphon piping, we believe that a proper Sedimentation Basin located immediately upstream designed to slow the canal velocity further than what is possible with the Intake Basin would be a beneficial addition to this project decreasing maintenance of the Inverted Siphon piping system and Inverted Siphon system performance between maintenance intervals.

The Outlet Structure will also be designed as a U-Frame channel. There are 30 degree wing walls at the structure's outlet and a headwall at the beginning of the structure where the influent transfers from the Inverted Siphon piping to the Outlet Structure. The length of the Outlet Structure is 30'-0" not including wing walls. The width of the structure is 47'-10". The height of the Outlet Structure is 10 feet with top of U-Channel wall elevation of 0.0 and an invert EL -10. The channel is subdivided between each Inverted Siphon pipe location.

Sluice gates are provided for each Inverted Siphon pipe and at the end of the structure. The same substitution of stoplog or bulkhead system in place of dewatering gates may be made at the outlet structure. There is a 15-foot access deck at the end of the Outlet Structure which will be designed for HS-20 loading.

The Outlet Structure will be pile supported on timber piling and the design will look at the maintenance condition with the structure dewatered at maximum buoyancy. The piles for the Outlet Structure will require tension connectors.

10.8.6 Gates and Trash Racks

The Intake Structure will have six (6) 10-foot sluice gates at the entrance as well as four (4) 5-foot and two (2) 4-foot sluice gates for each Inverted Siphon pipe at the rear headwall. All cast iron sluice gates will be rising stem, cast iron and meet AWWA C560. The sluice gates will be 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.

The Inlet Structure will also feature a steel bar screens with mechanical bar screen cleaners at the entrance to the structure preventing debris and trash in the canal from entering the structure and Inverted Siphon piping.

For the Outlet Structure, there will be four (4) 5-foot and two (2) 4-foot sluice gates for each Inverted Siphon pipe at the influent headwall. There will be six (6)-6 foot sluice gates at the exit of the structure. The cast iron sluice gates will be rising stem, cast iron and meet AWWA C560 specification. The sluice gates will be 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.

10.9 Marine Structures

TBD

10.10 Hwy 23 Bridge T-Walls

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 on both sides of the

Conveyance Channel. The T-Walls are located on both the north and south sides of the channel and are identical. The proposed T-Wall will connect to the guide levee tie-ins. 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.

10.10.1 Design Approach

For the 15% design phase, hand calculations based on the Ultimate Strength Design using ACI 318-14 and EM 1110-2-2104 as listed in the Design Criteria, Rev. 2 dated 4/27/2018 are performed to determine allowable shear and flexure acting on the stem and slab of the inverted T-Wall monoliths. Serviceability requirements are not checked at the 15% level design but will be checked in future phases.

For the 15% preliminary design phase, the quantities and size of T-Wall including number of piles, pile sizes, pile tips, length and base slab dimensions are calculated and can be found in **Appendix J**. The Conveyance Channel T-Walls under Hwy 23 Bridge start at EL 3 at the eastern guide levee tie-in and end at EL 3 at the western guide levee tie-in. The T-Wall monoliths and pile foundations are calculated based on hand calculations and excel spreadsheets for the design of components. Soil parameters provided by Eustis Engineering are used to calculate the soil pressures and pile capacities. All structural calculations and geotechnical information are included in **Appendix J**.

10.10.2 Pile Foundation

All monoliths are pile supported with pile tips set to mitigate differential settlement among monoliths. The piles are designed for lateral and vertical loads only for the 15% design phase. The moment from vertical and lateral loads are not considered for analysis. During the next phase, all the loads and moments will be included in the analysis for deflection and combined stress for piles. Pile axial capacity is calculated by multiplying the lateral load times the pile slope. Assuming that a static pile load test would be performed, the allowable Factor of Safety for pile loads is 2.0 in accordance with the MBSD Design Criteria. The required pile tip elevation is then found using the maximum tension and compression loads from the analysis results and plotting the points along the pile capacity curve for 24-inch diameter open-end steel pipe pile. Group analysis on piles is not performed for the 15% design phase. The pile size, pile tip elevation and pile capacities can be found in the drawings and calculations, **Appendix D** and **Appendix J**, respectively. The pile capacities are calculated by using the Eustis Engineering Design Soil Parameters and Pile Capacity data in Section 9.12.

10.10.3 Geometry

10.10.3.1 Base Slab and Stem

The base slab for the Conveyance Channel T-Walls is at EL 3 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. The T-Wall monoliths are identical on both sides. For this phase the top of slab for all Conveyance Channel T-Walls is at EL 3 and top of wall is EL 15.6. The T-Walls are back-filled with sand to EL 5 on both sides of the stem wall. Top of wall elevation is EL 15.6 for the 15% design phase. Wall stem height is 12-foot 6-inches and is the same for all monoliths. Base slab width and thickness is 15 feet and 5 feet, respectively. A continuous cut-off sheet pile curtain wall is installed beneath the monolith base slabs. All monoliths are pile supported with pile tips set to mitigate differential settlement among monoliths. Settlement calculations are not performed in the 15% level design but will be performed in the future phases. See **Appendix D** for pile layout, tip elevations, sizes and other pile features.

10.10.3.2 Cut-off Wall Sheet Pile

The cut-off wall of sheet piling is provided to limit seepage, and the embedment criteria are specified in the Geotechnical Report Section 9. Cutoff sheet pile will extend via a sheet pile transition wall into the levee embankment. The transition wall will be 30 feet long and the top of the sheet is matched with the levee crown. 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.

10.10.4 T-Wall Analysis

Analysis of the 3-dimensional structure was performed using a combination of hand calculations and Excel spreadsheets. The hand calculations, provided in the **Appendix J**, 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. The DT is proposing to eliminate the unbalanced loads by soil remediation with soil cement stabilization. Therefore, the unbalanced load for the 15% level design phase is not considered.

By summation of the applied forces and moments acting on the wall, the pile force and moments below the T-Wall were determined. 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. Factored concrete design loads are used to confirm the adequacy of the stem wall and slab thickness.

To streamline the design process across each of the Alignments, the T-Walls are designed identical. Design and analysis are performed for only the EL 3 base slab elevation. A continuous cut-off sheet pile curtain wall is installed beneath the monolith base slabs. All monoliths are pile supported with pile tips set to mitigate differential settlement among monoliths. Settlement calculations are not performed in the 15% level design but will be performed in the future phases. See **Appendix D** for pile layout, tip elevations, sizes and other pile features. Settlement calculations are not performed in the 15% level design but will be performed in the future phases. See **Appendix D** for pile layout, tip elevations, sizes and other pile features.

10.10.4.1 Load Cases

The load cases as described in the MBSD Design Criteria Table 5-5 (**Appendix U**) 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 design resiliency checks will be evaluated in a later phase of the project. 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 Table 2 & 3 of Section 2, Rev. 2, submittal draft No. 3 dated 4/27/2018.

Table 10-16: Hwy 23 T-Wall Design Load Case Summary

Load Case	Description	River Side Water EL	Land Side Water EL	Factored Load Combinations
1	Construction without soil, with surcharge	N/A	N/A	1.6(D+EH+EV+Ls)
4 & 5	Water at Design SWL	9.1	1.0	2.2(D+EH+EV+Hs+Hu)
14 & 15	Channel Low Water Reverse Head	1.0	1.0	2.2(D+EH+EV+Hs+Hu)
17 & 18	Water to Top of Wall	15.6	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)
17 & 18 Alt.	Water to Top of Wall	12.1	1.0	1.6(D+EH+EV+Hs+Hu+HW+W)

Notes: 1) Unbalanced loads not considered in this phase
2) 4, 14 & 17 impervious uplift, 5, 15 & 18 pervious uplift
3) DL= Dead Load, EH= Lateral Earth, EV= Vertical Earth, Ls= Construction Surcharge, Hs= Peak Hydrostatic, Hu= Uplift, HW= Wave, and W= Wind

10.10.5 Future Analysis and Design Considerations

The following will be addressed in future submissions:

- Coordinate with the adjacent structures to identify and rectify any pile conflicts.
- Combine the floodwall and siphon headwall as an alternative design.
- Evaluate all applicable load cases, including the design resiliency checks.
- Verify the assumed construction sequence to determine its appropriateness.
- Investigate pile foundation deformations and mitigation measures.
- Perform a more detailed design check for all of the structural members including rebar for shear and flexure.
- Determine the piles that requires tension connections.
- Piles analysis using group pile program by Ensoft Corporation.
- Future Design grade and hydraulic grade for hurricane is not considered for this design phase, but will be considered for the future design phase per DT.
- Determine piles that require moment connections.

11. CONVEYANCE CHANNEL AND LEVEES

11.1 General

The Conveyance Channel was designed to convey the sediment-laden river water from the Intake Structure to the Basin without overtopping the guide levees with enough velocity to prevent buildup of siltation in the channel and with protection against scour. At Workshop No. 2, a parallel Hurricane/Guide Levee alternative was compared to a Back Gate alternative, and the parallel Hurricane/Guide Levee alternative was selected. This is discussed in **Section 7**. The Guide Levees from the Diversion Gate Discharge Transition Segment to the new federal NOV-5a Levee Reach, which is located near the Timber Canal, will serve as hurricane flood protection. From the NOV-5a Levee Reach to the Diversion Outfall, the guide levees will serve only to convey the discharge flows.

With the decision at Workshop No. 2 to eliminate the Back Gate Structure, the channel guide levees must not only confine the diversion's discharge, but also serve as hurricane flood protection levees against hurricane storm surges.

11.2 Design Approach

The approach for the hydraulic design of the Conveyance Channel is discussed in **Section 8.6**; the geotechnical design approach for the levees is discussed in **Section 9.16**.

11.3 Conveyance Channel Geometry

The results of the numerical modeling of the Conveyance Channel are discussed in **Section 8.6**. The cross section of the channel includes a bottom width of 300 feet, with invert EL -25. Side slopes extend at 4H:1V until EL -2, where a berm extends 97 feet to EL 4. The total width of the Conveyance Channel is 734 feet.

11.4 Levee Design

Without a Back Gate Structure, the levees must provide hurricane coastal protection against storm surges. The DT investigated two design grades, one at EL 12.1 and the second at EL 15.6. The EL 12.1 grade provides a higher level of storm damage risk reduction than does the proposed USACE New Orleans to Venice (NOV) 5a levee project. The USACE project Design Grade is EL 9.6, which correlates to a 25-yearr event without overbuild for future Sea Level Rise (SLR). The EL 12.1 equates to a 25-year Storm with overbuild to account for 25 years of SLR and regional subsidence based on rates established by the USACE. The EL 15.6 grade is the USACE Design Grade for the Reach NOV-NF-W-05c, projected 50 years into the future (i.e., 2063), also accounting for SLR and regional subsidence. The DT recommends a Design Grade of EL 15.6 as further explained in **Section 11.5.9**.

The levee will be constructed with an overbuild of earthen materials, which will vary along the reaches of the channel as dictated by geotechnical analysis. A 10-foot wide levee crown will be topped with a 6-inch thick gravel access road. Side slopes will be constructed at 4H:1V with turf reinforcement and/or armoring; the levee slopes will extend to EL 4, then will slope to intersect existing ground on the protected side, and slope to form a berm to the top of the Conveyance Channel on the flood side. The wick drains will accelerate most of the predicted settlement to occur within the planned construction duration. One, 12-inch lift will be required at 20 years after construction to maintain EL 15.6. The DT estimates that sufficient quantities of suitable material required to construct the levee, including overbuild for settlement, is available from conveyance channel and Headworks excavation. The quantity calculations used the soil boring data obtained during BOD Phase, the latest topographical surveys, and a conservative 1.5 loss factor. Note that the unsuitable material will be used as Beneficial Use Material as described in **Section 24**.

Installation of a wick drain system will accelerate expected levee settlement. A grid layout of wick drains will be overlain by a granular drainage layer, and the levees will be constructed in stages so that the underlying soils incrementally gain strength as the levee is raised. A through-seepage cut-off will need to be installed through the drainage layer. The 15% drawings show a clay plug; however the final design will be decided in coordination with the CMAR. The DT briefly considered Deep Mixing Method (DMM) columns or panel to support the levee embankment, but did not develop a conceptual design because there is sufficient real estate to construct through wick-drain-aided staged construction, and by inspection this will be more economical than DMM columns/panels, provided there is sufficient on-site fill material available to complete embankment construction. The selection of the wick drains will be confirmed in coordination with the CMAR during the 30% Phase.

Other Conveyance Channel features of note include T-wall segments beneath the new Hwy-23 Bridge and at the new Inverted Siphon inlet and outlet. The Inverted Siphon will be located near the Timber Canal. Canal closures at the Timber Canal and Back Levee Canal will also be features of the Channel and Guide Levee System. T-walls are discussed in **Section 10**. Canal Closures are discussed in **Section 9**.

11.5 Armoring Design

11.5.1 General

The DT conducted an analysis of viable methods to protect the Conveyance Channel from erosion damage. The methodology and details of that analysis are presented in the Conveyance Channel Revetment Study included in **Appendix N**. The work involved the review of numerous guidance documents which are enumerated in the study and in the DCD in **Appendix U**.

The DT established four criteria to evaluate the protective armoring: 1) Maintain a stable bank configuration, 2) Protect against erosive forces, 3) Provide minimal frictional resistance, and 4) Be cost-effective. The team reviewed the plan and profile along with the cross-section of the Conveyance Channel in detail, dividing the channel into five sections: 1) Channel bottom, 2) Channel slope, 3) Stability berm, 4) Bottom half of levee slope, and 5) Top half of levee slope. The DT evaluated the conditions at each section and developed recommended armoring solutions. The team determined that three types of armoring were viable alternatives and reviewed each of them in detail: 1) Riprap revetment, 2) Articulated concrete blocks (ACBs), and 3) Turf reinforcement mats.

11.5.2 Geotechnical Considerations

The DT reviewed the main geotechnical considerations that would affect the performance of the various revetment alternatives. Based on the limited geotechnical data gathered to date, a major portion of the native material is comprised of highly dispersive clays that are readily subject to erosion without some form of protection. Geotechnical analyses of the available data indicate that substantial settlement will occur in the areas where the new levee and stability berm will be constructed **(Appendix G)**. The DT geotechnical analyses predict major settlement both during construction, as well as over 3-feet of additional long term settlement after completion of construction. The size and required layer thickness of the riprap will result in greater settlement than an ACB system due to the significant difference in weight. The heterogeneous nature of soils suggests that the settlement will not be uniform across all areas, thus the DT anticipates that there will be significant differential settlement. The differential settlement is problematic for the ACB system.

The small size of the clay soil particles, along with the seepage potential due to changing water levels inside and outside of the channel, will require a filter base to prevent the loss of fines. The DT recommends the use of a graded filter, along with a geotextile separator fabric, where feasible, to address this issue. Due to the lack of complete geotechnical information, the final filter design cannot be performed at this time. Based on the available data, the DT assumed that the riprap would have a filter base of 2-feet of sand plus 6 inches of No. 57 stone. A 6-inch bedding layer of sand was assumed for the ACB system, along with a geotextile, which can be attached to the bottom of the ACB mats, enabling its installation either in-the-wet or in-the-dry. Since an accurate method of installing the separator fabric underneath riprap in a significant depth of water has not yet been developed, the DT assumed that fabric would be installed under the riprap only in the areas of in-the-dry construction. The team is currently conducting slope stability analyses to delineate those areas that can be constructed in-the-dry.

11.5.3 Failure Modes

The DT assessed the revetment requirements to protect the Conveyance Channel against three potential failure modes: 1) Shear stress, 2) Seepage, and 3) Wave action. Shear stress failure is the movement of revetment material due to the hydraulic forces acting over a range of channel flows during Normal Operating Conditions. Seepage failure is the loss of fines from the underlying soil due to water movement through the revetment. Wave action failure is damage to the revetment and subsequent erosion of channel material due to wind-driven impacts from major Storm/Hurricane Conditions.

11.5.4 Revetment Sizing

The DT calculated the required sizes of riprap revetment and ACBs under both the Normal Operating Conditions and the Storm/Hurricane Conditions. The team modeled the water depth and velocity for cross-sections in each reach of the Conveyance Channel during Normal Operating Conditions with a flow of 75,000 cfs. Based on the resulting water depths and depth-averaged velocities, the DT sized riprap revetment for the various locations within the cross-section using the EM 1110-2-1601 protocol. The results showed that (under the LADOTD classification system) 10-lb stone riprap with layer thicknesses of 1-foot and 1.5-feet would be sufficient for the areas constructed in-the-dry and in-the-wet, respectively. Using the same data, the team calculated the Factor of Safety for various ACBs based on the National Concrete Masonry Association design procedure. The calculations showed that a 4-inch thick ACB would be sufficient to withstand the Normal Operating Conditions with a Factor of Safety of 3.3.

The DT is currently modeling the effects of Storm/Hurricane events on the conditions within the Conveyance Channel. Since that work is not yet complete, the results of earlier modeling performed by HDR were used to estimate the revetment requirements. HDR obtained the following results for the maximum wave heights from two 1-D cases run using the ACES software for a 50-Year return period:

Table 11-1: HDR Model Results from Storm\Hurricane Conditions

Case	WSE Exceedance Probability	WSE (ft NAVD88)	Maximum Wave Height (ft)	Wave Period (sec)
1	10%	1.85	2.2	3.7
2	2%	10.4	6.0	4.7

The maximum wave heights occur for a very short distance along the Conveyance Channel starting at the Barataria Bay end, in the center of the channel. HDR's graphical depiction of the data shows that the wave heights never exceed 0.33-ft along the channel slopes and rapidly attenuate along the length of the channel to just over 1.3-ft in the center of the channel. Based on the industry standard Hudson and van der Meer formulas the riprap sizes to resist the wave forces at the entrance to the channel were determined by HDR to be: 30-lb stone for the 10% probability event and 250-lb stone for the 2% probability event. The DT will investigate the armoring requirements for the intake and outfall ends of the channel in the next phase; the subject work addresses only the main reach of the channel itself.

For the main reach of the channel, based on the relatively small 0.33-ft waves on the channel slopes the 10-lb stone is proposed as sufficient protection. The 10-lb stone is also recommended for the channel bottom because the 30-foot water depth prevents the forces from the 1.3-ft waves at the surface from reaching the bottom. For the ACBs, the DT assumed that a 4-inch ACB block will be sufficient across the entire cross-section for both the Normal Operating Conditions as well as for both Storm\Hurricane events. All of these calculations will be recomputed once the DT completes the modeling of the Storm/Hurricane conditions.

11.5.5 Revetment Friction Coefficient

The frictional coefficient, n, in Manning's equation for open channel flow is inversely proportional to the volumetric flowrate through the channel. E.g., a 10% reduction in n results in a 10% increase in flow. The DT researched n values for various revetment materials from numerous sources. For a finished concrete surface, n = 0.013 is commonly used, while for 6-inch riprap, n = 0.035 is typical. The size of the riprap affects the n value, e.g., for "nominal conditions", 1-inch gravel n = 0.030 while for 12-inch stone n = 0.040. The concrete ACBs do not butt perfectly together, creating a checkboard of gaps across the mattress which raises the n value. The typical value quoted by ACB manufacturers is n = 0.020.

The n value is also a function of the depth of flow; the shallower the water, the larger the n. Thus, the n value for the same material would be less on the channel bottom, under 30-feet of water, than near the surface in only a couple of feet of water. Another factor affecting the n value in the Conveyance Channel is the potential sediment accumulation, filling the gaps and voids, and perhaps even covering the entire surface of the revetment. The DT will continue to research the appropriate n value to use, based on additional literature searches, calculations, modeling of the sedimentation process, and possibly physical modeling of revetment with and without accumulated sediment.

11.5.6 Construction Considerations and Costs

Review of construction considerations and estimated costs of each system can be found in **Appendix N.**

11.5.7 Revetment Configuration

The depth of the channel bottom isolates it from the effects of significant storm events. Therefore, the Normal Operating Conditions govern its design. Since the 10-lb riprap is much less expensive that ACBs and because of the difficulty of aligning the ACBs on the bottom, 10-lb riprap is recommended for the channel bottom. The channel slope, stability berm, and levee will experience the wind-driven wave forces during major storm events. However, since the DT has not modeled the effect of the Storm\Hurricane Conditions on the Conveyance Channel and the HDR graphic shows rapid attenuation of the wave heights, the current design recommendation for the majority of the Conveyance Channel is the use of 10-lb riprap throughout the entire cross-section. The riprap was chosen over the ACB system since proof of performance of an ACB system under such conditions has not been documented yet.

All of the riprap will be installed over a dual filter layer comprised of 6 inches of No. 57 stone plus 2 feet of sand. All areas constructed in-the-dry will have geotextile separator fabric installed. The DT will continue to investigate feasible ways of installing fabric in the wet. The recommended revetment protection system for the in-the-wet construction conditions is thus:

Table 11-2: Recommended Revetment Configuration In-the-Wet Construction

Cross-Section Location	Protective Revetment		Filter Layer[1]	
	Material	**Thickness**	**Material**	**Thickness**
Channel Bottom	10-lb Riprap	1.5-ft	No. 57 Stone	0.5-ft
			Sand	2-ft
Channel Slope	10-lb Riprap	1.5-ft	No. 57 Stone	0.5-ft
			Sand	2-ft
Stability Berm	10-lb Riprap	1.5-ft	No. 57 Stone	0.5-ft
			Sand	2-ft
Bottom ½ Levee	10-lb Riprap	1.5-ft	No. 57 Stone	0.5-ft
			Sand	2-ft
Top ½ Levee	HPTRM	N/A	N/A	N/A

1. If a feasible method of installing the fabric in-the-wet is developed a layer of geotextile filter fabric will be installed above and below each of the sand layers. Where construction can be performed in-the-dry, the geotextile layers will be installed.

11.5.8 Path Forward

The following activities will be performed to progress the design:

- The DT will collect additional geotechnical data and perform laboratory testing and analyses. This will enable refinement of the required filter layers and will inform the selection of the optimum revetment materials, sizes, and layer thicknesses.
- The DT will perform geotechnical stability analyses to delineate areas that can be constructed in-the-dry and define the sequence of construction events required to keep a stable excavated slope (in-the-dry) with an adjacent levee section.
- The DT will perform hydrodynamic modeling of Storm/Hurricane Conditions and define the effects on the Conveyance Channel. The team will select a design storm and design a revetment system to withstand such conditions.
- The DT will perform sediment transport modeling within the Conveyance Channel. The results will be used to select an appropriate n value and model the performance of various revetment configurations.

- The DT may perform physical modeling to measure the n value of riprap and/or ACB with various amounts of sediment deposition. The results will be used along with the sediment transport modeling to select the most cost-effective revetment system.
- The DT will continue to investigate potential methods for the installation of geotextiles and other filter components under water.
- The DT will refine the revetment configuration using multiple riprap and/or ACB sizes across the channel cross-section.
- The DT will estimate the maintenance costs for the various revetment systems and calculate life-cycle costs for comparison of alternatives.

11.5.9 Recommendations

11.5.9.1 Revetment

The recommended armoring system is described in **Section 11.5.7**. The Manning's "n" value will be verified in physical modeling performed in the next design phase. The modeling will include the recommended riprap and the effects of a sustained silt layer covering the riprap. Numerical models will be revised as needed. Armoring enhancements to sustain surge effects at the basin outlet will be developed in the next design phase.

11.5.9.2 Channel Geometry

The recommended channel geometry is described in **Section 8.6**. The 300 ft bottom width at EL-25 and side slopes at 4H on1V is recommended for current boundary conditions and also future conditions. The excavated channel provides sufficient suitable material for levee construction.

11.5.9.3 Levee Design Grade

The DT recommends a Design Grade EL 15.6 for the hurricane levees that extend from the Diversion Gate Structure to the USACE NOV 5a tie in. The levee segment to the basin side of the NOV 5a tie-in, will be constructed for conveyance requirements only; a design Grade of EL 9.5 is recommended. The conveyance water surface elevation, considering future conditions, at the basin end is EL 6.2. A freeboard of 3 feet was added to the conveyance stage, and the design grade was rounded to EL 9.5. The recommendation for the hurricane grade of EL 15.6 considered risk reduction (protection levels), cost, available material, available Right of Way, and the 100-year level of protection proposed by the Alliance Refinery. The main concern is flood protection for Plaquemines Parish. The USACE NOV projects constructed post Katrina used a 50 year future design grade. All major structures built along the NOV levee system south of Oakville, LA were built to the 50 year future design grade. USACE levees would be increased to the same grade over time if funding was sufficient. In building to EL15.6, the MBSD project would be matching the highest level of protection. With the elimination of the back structure, the recommended EL 15.6 grade exceeds the current NOV 5a levee project by 6 feet, an apparent risk reduction measure. The channel excavation will provide a sufficient amount of suitable material to construct Levees to EL15.6 considering all settlement and related overbuild. The unit cost of levee material will not need to be increased to include borrow pits or imported material. The proposed Right of Way of 800 feet each side of Channel C/L is adequate to cover the footprint for the levee section at EL 15.6. If the NOV Levee on the upriver side of the diversion later were to be raised to a 100-year level of protection, that grade is EL 15.1, not accounting for future SLR or regional subsidence. The recommended 50-year future design grade slightly exceeds that grade. The cost increase to construct to the recommended EL 15.6 compared to EL 12.1 is approximately $17.9 million. For the reasons included herein, the DT considers the added cost worth the value added. On a percentage basis, the total cost of the hurricane protection is $276.0 million; the increase represents a 6.5% increase. In comparison, the back structure's construction cost is estimated at $276.6 million, not including the cost to construct the parallel guide levees, so it is still the more economical alternative.

12. OUTFALL TRANSITION FEATURE

12.1 General Design Approach

The outfall transition feature (or outfall channel or outfall ramp) is considered the area on the basin side of the existing NOV Levee that transitions the Conveyance Channel to the natural ground within the basin. The design of the outfall channel considers two primary features. 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 is the scour protection near the NOV Levees and the transition channel.

A hydraulic and cost analysis of the outfall ramp configuration has been conducted to guide the selection of the final ramp design. The primary function of the outfall ramp is to provide a gradual transition from the Conveyance Channel to the basin. The invert of the Conveyance Channel is approximately EL -25 and the basin elevation near the outfall is approximately EL -4. The ramp is intended to be a temporary feature of the design. It is expected that the diversion discharge will eventual erode a channel into the basin based on the results of the TWIG's Basin Wide Model and Outfall Management Models. Thus, the role of the outfall ramp is to provide an initial transition during the first few years of operation or until a channel is eroded. The ramp configurations were evaluated based on two metrics, the head loss and the capital dredging costs.

12.2 Hydraulic Design

The hydraulic analysis of the outfall transition feature was described in **Section 8.8**.

The analysis indicated that the head loss due to the transition feature did not depend significantly on the half-flare angle (the angle that the ramp widened as it extended into the basin). The head loss was dependent on the length of the ramp, with decreasing changes as the ramp approached 4,000 to 5,000 feet. A summary of the head losses for each ramp assuming a 10 degree half-flare angle, are summarized in Table 12-1 and shown graphically in Figure 12-1.

Table 12-1: Summary of Stage Impacts

Ramp Length (feet)	Upstream Stage (ft, NAVD88)	Tailwater Stage (ft, NAVD88)	Head Loss* (ft, NAVD88)	Relative Difference** (feet)
500 ft	6.13	2.84	3.30	1.06
1000 ft	5.63	2.84	2.79	0.55
1500 ft	5.42	2.84	2.58	0.34
2000 ft	5.31	2.84	2.48	0.23
3000 ft	5.19	2.84	2.36	0.11
4000 ft	5.08	2.84	2.24	0.00
5000 ft	5.08	2.84	2.24	0.00

*Head Loss does not include velocity (difference in stage only)

**Compared to Head Loss for the 5000-foot ramp length

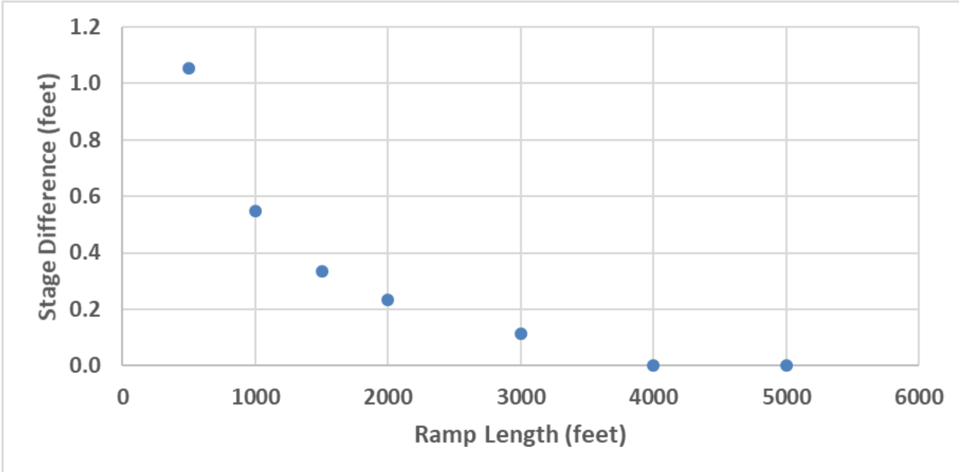


Figure 12-1: Difference in Head Loss (compared to 5,000-foot ramp length)

The footprint of each ramp alternative and the dredge volume required to construct each ramp configuration are provided in Table 12-2.

Table 12-2: Summary of Head Loss and Dredging Requirements

Length (ft)	Flare Half-Angle (deg)	Relative Difference (feet)	Footprint Area (ft^2)	Dredge Volume (cy)
500	10	1.06	321,000	89,900
1000	10	0.55	679,000	195,300
1500	10	0.34	1,163,000	335,100
2000	10	0.23	1,693,000	483,300
3000	10	0.11	3,096,000	866,900
4000	10	0.00	4,826,000	1,329,900
5000	10	0.00	6,891,000	1,874,700

12.3 Armoring and Toe Sheeting

With high velocities within the Conveyance Channel, scour protection will be required near the NOV Levees of the outfall. The revetment sizing within the Outfall Transition Feature is similar to the Conveyance Channel and is selected based on velocities. Scour protection based on wave energy was not considered as water depth in the channel prevents the wave forces from reaching the bottom. The effects of storm/hurricane will be further modeled and incorporated during the next phase of design.

Depth averaged velocities were analyzed within the outfall channel and considered comparable or reduced to the Conveyance Channel. See Figure 12-2 for depth averaged velocities.

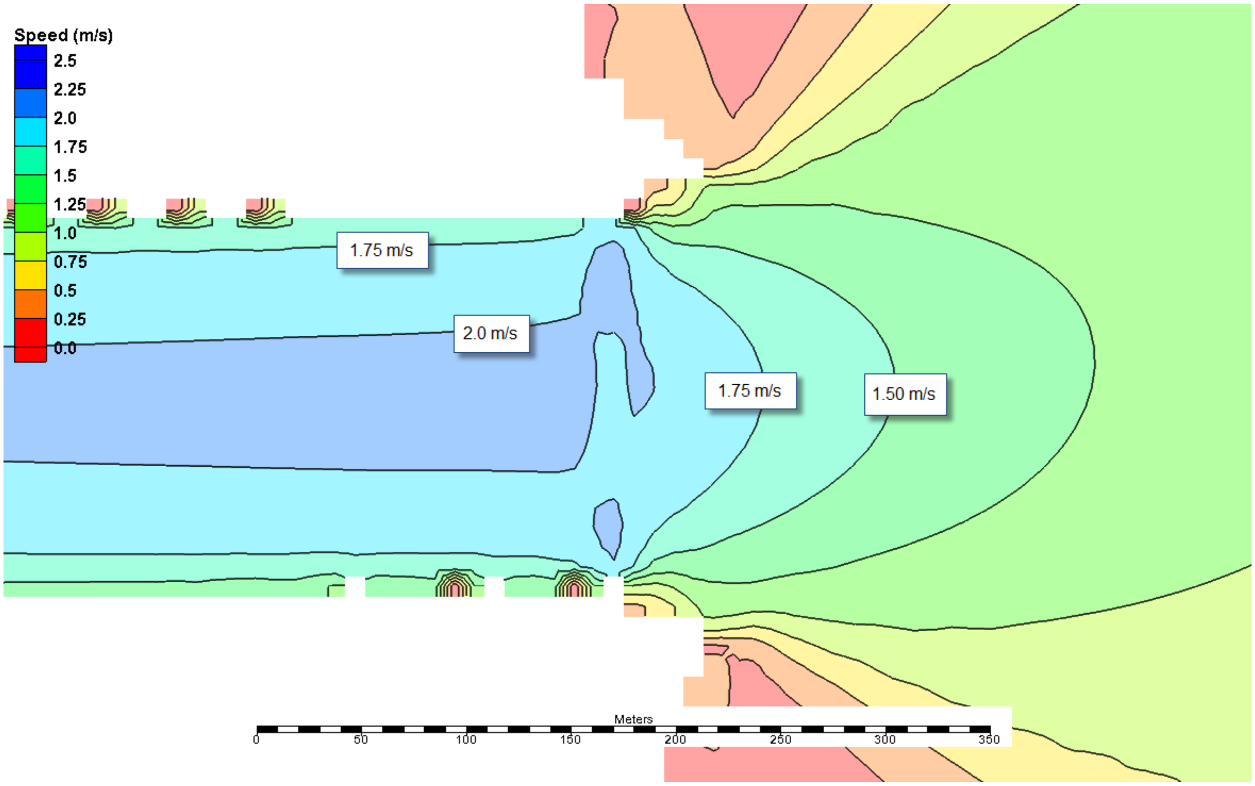


Figure 12-2: Depth Averaged Velocities within Outfall

As the flow within the channel transitions into the basin, velocities are reduced. Therefore, 10lb riprap is proposed for the Outfall Transition Feature similar to the invert of the Conveyance Channel. The armoring will be installed from the existing NOV Levee to EL -20. This armoring is to protect the integrity of the existing NOV Levee and the Shell pipeline. The existing Shell pipeline is located on the flood side of the NOV Levee at a shallow depth. The pipeline will be relocated to below the Outfall Transition Feature prior to construction and operation of the diversion. The pipeline relocation should be sufficient for scour protection of the pipeline, although the armoring proposed for the NOV Levee will provide an extra level of protection. As the flow extends past the Outfall Transition Feature, the intent of the diversion is to build its new channel to deliver sediments into the basin. Therefore, scour protection along the ramp between the EL -20 and the natural ground is not proposed.

The protection measures along the existing NOV Levee is proposed at 250-pound riprap. This is based on the 50-Year storm event and sized as a part of the Conveyance Channel and levees armoring design. The armoring section along the NOV is proposed for a distance of 100 linear feet past the tie in point of the NOV Levee and the Conveyance Channel levee.

In addition to the riprap armoring, toe sheeting at the transition point will be installed near Station 140+00. Sheets will extend across the Conveyance Channel invert to the crown of the Conveyance Channel levee. Sheeting is proposed at PZ-27 and will have a top EL -27 with a tip EL -57. Sheet pile will be capped with a riprap protection that includes 6 inches of bedding stone and 18 inches of 10-pound riprap. Sheets will be stair stepped up the slopes of the Conveyance Channel at 5-foot increments. Figure 12-3 shows the toe sheeting detail. A cross section of the toe sheeting is shown in the BOD Plans.

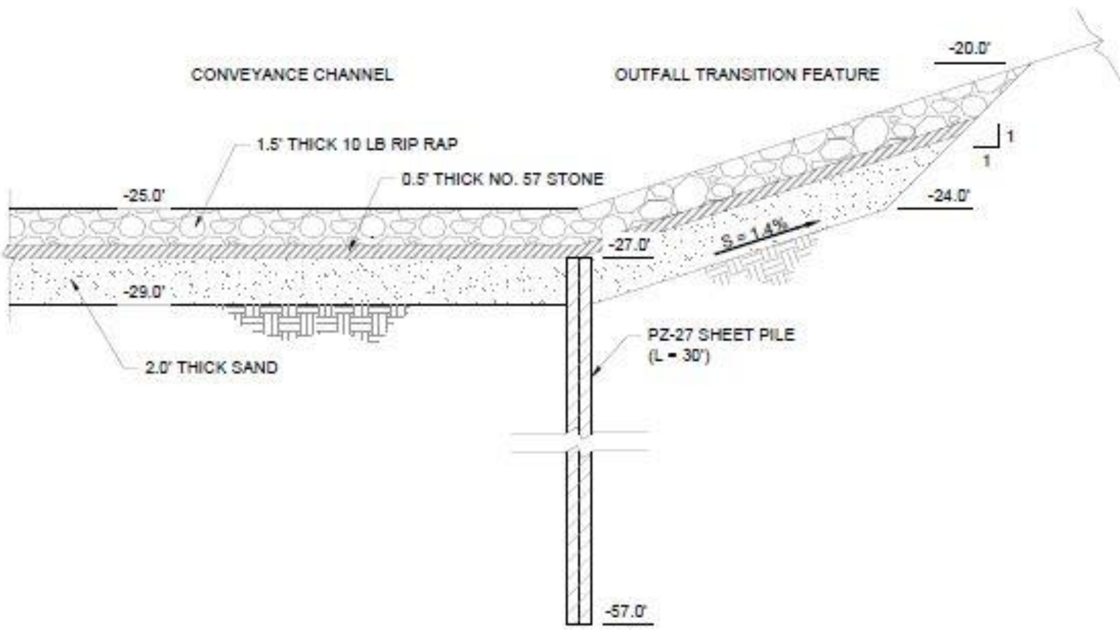


Figure 12-3: Toe Sheeting Detail

13. HWY 23 ROADWAY AND BRIDGE

13.1 General

Hwy 23 is a north-to-south state highway 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. Hwy 23 connects Gretna and Venice. 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. Hwy 23 is approximately 74 miles long. Within the area of the project, the roadway is a four-lane rural arterial asphalt composite roadway with 4 feet wide inside and 10 feet wide outside shoulders and a 42 feet wide depressed grass median. The existing typical section is shown in **Appendix D**.

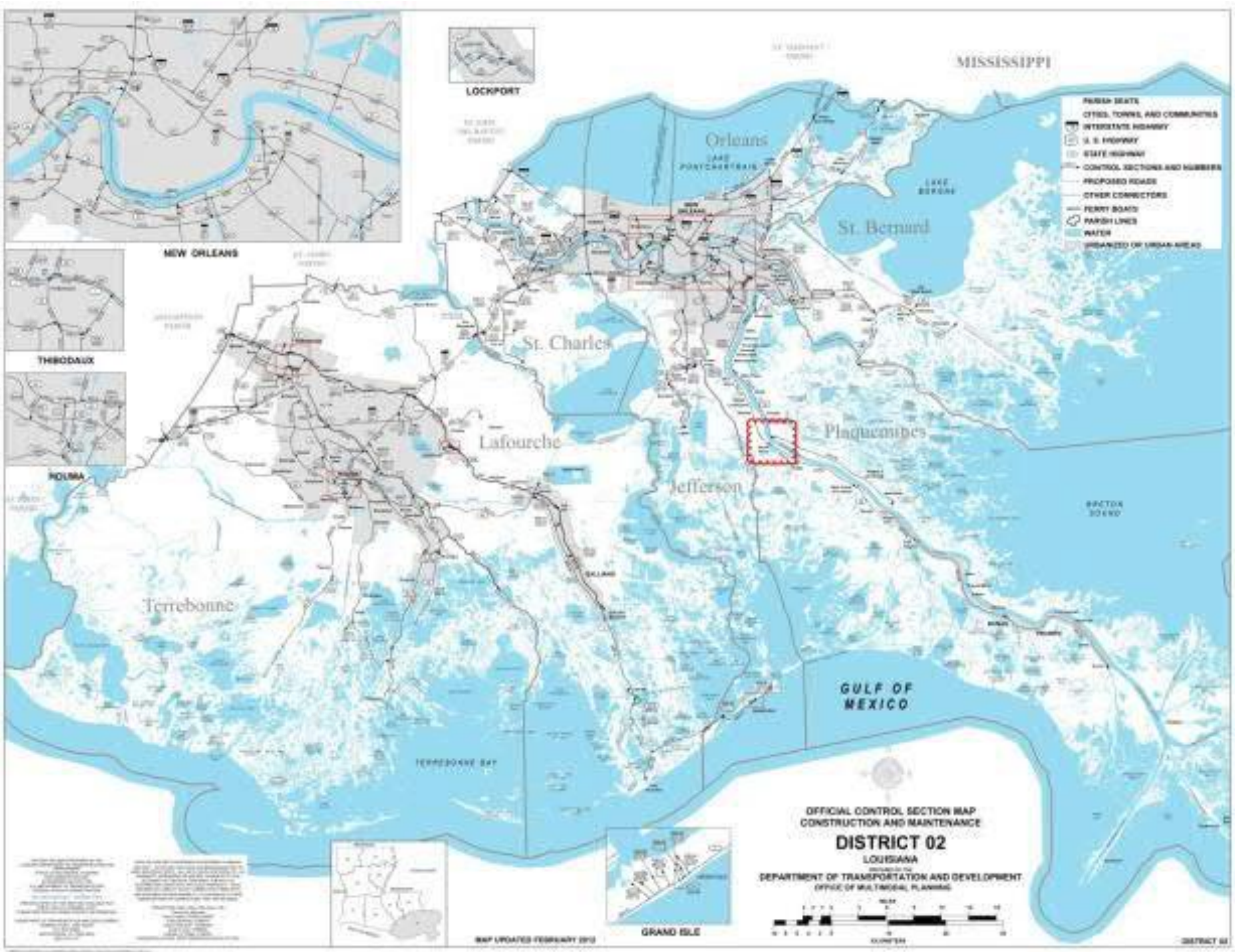


Figure 13-1: Location Map

Source: LADOTD (2012)

The area outlined in red is the location of the MBSD, which is south of the ConocoPhillips Alliance Refinery and north of the town of Ironton in Plaquemines Parish. The portion of Hwy 23 in this area would be affected by the project.

13.2 Design Approach

In BOD Project Phase, the DT was tasked to perform a traffic study and review highway alignment alternatives that update the proposed highway and bridge work to current LADOTD standards. Due to the design changes in the channel, the roadway geometrics will require further refinement in future design phases. Three alternatives were developed and analyzed utilizing the Conveyance Channel geometry established prior to Design Workshop Nos. 1 and 2. These alternatives considered right of way acquisition, maintenance of traffic, constructability, and cost.

13.3 Roadway and Bridge Design Criteria

13.3.1 List of References

The roadway and bridge would be designed in accordance with LADOTD standards and specifications. The following published design standards and manuals are to be used during the design of the Hwy 23 reconstruction:

- LADOTD Roadway Design Procedures and Details (often referred to as the Roadway Design Manual), latest edition
- LADOTD Minimum Design Guidelines dated March 6, 2017
- AASHTO A Policy on Geometric Design of Highways and Streets, 2011 Edition
- AASHTO Roadside Design Guide, 4^{th} Edition
- AASHTO Highway Safety Manual, 2012 Edition
- Engineering Directives and Standards Manual (EDSMs)
- LADOTD Guidelines for Conducting a Safety Analysis for Transportation Management Plans and Other Work Zone Activities
- LADOTD Traffic Management Plan
- LADOTD Construction Plans Quality Control/Quality Assurance Manual v2013
- LADOTD Hydraulics Manual
- LADOTD Erosion Control Guidelines
- LADOTD Bridge Design and Evaluation Manual
- Highway Capacity Manual, 2010 Edition
- Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD), 2009 Edition with revisions 1 and 2 in 2012
- LADOTD Highway Specifications Workbook
- LADOTD Louisiana Standard Specifications for Roads and Bridges, 2016 Edition
- LADOTD Standard Plans and Details
- Current Federal Regulations (CFRs)
- LADOTD Location and Survey Manual

13.3.2 Design Criteria

13.3.2.1 Roadway Design Criteria

Hwy 23 is classified as a Rural Minor Arterial. Table 13.1 presents the selected roadway design criteria selected from the Minimum Design Guidelines, last updated on March 6, 2017.

Table 13-1: Selected Design Criteria for Hwy 23

Item no.	Item	Rural Minor Arterial (LA 23)	Local Roads	Ramps
1	Design speed (miles per hour)	65	30	50
2	Number of lanes	4	2	1
3	Travel lane width (feet)	12	11	15
4	Shoulders			
	Two-lane facility	N/A	2	N/A
	Divided facility inside shoulder	4	N/A	5 (paved)
	Divided facility outside shoulder	10	N/A	6 (paved)
5	Median			
	Depressed	< 64 ft with median barrier	N/A	N/A
	Raised	Not applicable	N/A	N/A
	Two-way left-turn lane	Not applicable	12 feet min.	N/A
6	Fore slope (vertical-horizontal)	1:6	1:4	1:6
7	Back slope (vertical-horizontal)	1:4	1:3	1:4
8	Pavement Cross Slope (%)			
	Cross Slope in Tangent	2.5	2.5	2.5
	Max Cross over Crown (Travel Lanes)	5	5	5
	Max Cross over Crown (Shoulder)	7	7	7
9	Stopping sight distance (feet) (AASHTO Green Book)	645	200	425
10	Maximum superelevation (%)	8	8	8
11	Minimum radius (feet)	12900 (NC)	3240 (NC)	8150 (NC)
		7553 (RC)	1876 (RC)	4770 (RC)
		1,480 (Full Super)	214 (Full Super)	758 (Full Super)
12	Lateral Offset	1.5	1.5	1.5
13	Maximum grade (%)	3	5	5
14	Minimum vertical clearance (feet)	16'-6"	16'-6"	16'-6"
15	Minimum horizontal clearance (feet) (from edge of travel lane)	30	7	10
16	Bridge design live load	LADOTD BDEM	LADOTD BDEM	LADOTD BDEM
17	Width of bridges (min.) (face to face of bridge rail at gutter line) (feet)	Approach Travel Lanes+Full Shoulder Width	Approach Travel Lanes+4 feet	Approach Travel Lanes+Full Shoulder Width

13.3.2.2 Bridge Design Criteria

In addition to the minimum vertical clearance to a roadway surface stated in Section 3.1, the following additional minimum vertical clearances will apply:

- Minimum vertical clearance to the top of a levee floodwall shall be 5 feet from low chord
- Clearances for Navigation in channel
  - Minimum Vertical Clearance shall be 25 feet from Max. Water Surface of EL 2 to low chord.
  - Minimum Horizontal Clearance between pier bents shall be 120 feet.

The highway and bridge will be designed and constructed to current LADOTD standards.

13.4 Roadway and Bridge Geometrics

Three horizontal alignment alternatives were developed and analyzed utilizing the Conveyance Channel geometry established prior to Design Workshop Nos. 1 and 2. All alternatives generally have the same profile which provides for 3.0% maximum approach grades, a 1,280 foot long crest curve at the midpoint of the structure, and 480 foot long sag curves that transition the grade back to the existing roadway elevations on either side. These alternatives considered right of way acquisition, maintenance of traffic, constructability, and cost. Plans are included in **Appendix D**.

The three alternatives can be summarized as follows:

- Alternative 1 provides for one bridge structure centered on the existing right-of-way centerline that carries two lanes of traffic in each direction separated by a median barrier. It transitions to two bridge structures on the south approach in order to transition back to the existing roadway typical section at the end of the bridge. Access ramps provide access to the levee roads and maintain access to properties on each side of the right of way and also would be used for the maintenance of traffic during construction.
- Alternative 2 provides for one bridge structure centered on the existing right-of-way centerline that carries two lanes of traffic in each direction separated by a median barrier. The transitions from the proposed bridge typical section and existing roadway occur outside the bridge approaches on the at grade roadway. Access ramps provide access to the levee roads and maintain access to properties on each side of the right of way and also would be used for the maintenance of traffic during construction.
- Alternative 3 provides for one bridge positioned east of the southbound lanes. Access ramps provide access to the levee roads and maintain access to properties on each side of the right of way. Maintenance of traffic would occur with both directions of traffic on the existing southbound lanes.

13.4.1 Preferred Alternative

The preferred alternative, Alternative 3, begins at Station 388+00 as a 4-lane asphalt rural arterial highway. Utilizing the 2,800-foot radius curve beginning at Station 391+19.71, the roadway transitions through the length of the curve (856.23 feet) from its existing typical roadway section with a 44-foot wide depressed median to the proposed typical section for the bridge which includes two 12-foot wide lanes in both directions, 10 foot outside shoulders, and 4 foot inside shoulders separated by a 2-foot wide median barrier. This transition exceeds the required transition length of 585 feet. A concrete median barrier that starts along the northbound lanes will be used to protect between oncoming traffic.

The ramps to provide access to the adjacent properties on the north side of the Conveyance Channel begin at Station 402+00. The bridge centerline shifts to the east side of the right of way to reduce impact to transmission lines and construction along the west ROW line and maintain two way traffic on the southbound lanes during bridge construction. The mainline highway crosses over the Conveyance Channel. An extension of 5 feet of deck will be on the outside of the existing cross section to anchor a relocated water line. Further investigation will occur to confirm whether the waterline can be supported by the girders underneath the deck. If possible, the deck width will be reduced. On the south approach of the bridge there is a curve beginning at Station 422+56.68 with a radius of 7,668.44 feet at the bridge centerline. This allows for a superelevation of 2.5% in the southbound lane. The deck maintains the cross slope across the northbound lanes. The transition of the south bound lanes occurs over a distance of 310 feet with the transition beginning 257 feet prior to the PC. The transition out the curve is 230 feet long with 184 feet after the PT. The additional length on the bridge is to prevent a ponding area caused by the combination of the longitudinal grade and the cross slope transition. Thus, only the southbound lanes will be surperelevated. The main highway will transition the median width back to the original section between the two curves at Station 428+93.12 and Station 449+92.63. The access ramps for the south side of the Conveyance Channel ties into the mainline roadway near Station 439+50. The project ends at Station 450+00.

Alongside both ends of the bridge, there will be levee access ramps in both directions, each of which will have a 4-foot wide inside shoulder, a 15-foot wide lane, and a 6-foot wide outside shoulder. Starting at Station 400+99.67, the southbound off-ramp will depart from the roadway at a 3.5 degree angle and extend 1,235.7 feet until it reaches the levee of the canal. The southbound on ramp will extend 1,371.45 ft. and enter back on to the roadway at Station 439+50 at a 3.5 degree angle. Both southbound ramps will utilize the existing southbound lane pavement and only divert from the existing road when merging into the relocated southbound roadway. Thus, the project will only require ROW on the east side in order to fit the northbound access ramps. The northbound on ramp will exit the roadway at a 3.5 degree angle from Station 402+00 and extend 1,394.87 feet until it reaches the northern levee road. The northbound off ramp will extend 1,231.9 feet and enter the roadway at Station 439+50 at a 3.5 degree angle. Alternative 3 allows for construction of the bridge to occur east of the detoured traffic which will be maintained in the southbound lanes. Northbound traffic would be detoured to the existing southbound lanes and the southbound traffic would be reduced to one lane.

Ramps that are not on the existing developed roadway will require surcharged fill and muck excavation in order to provide a stable embankment and base for the roadway. The pavement sections for the highway and ramps and the necessary specifications for the subgrade material for the new roadways are as stated in the geotechnical recommendations in Section 9 of this report. Preliminary roadway typical sections and geometrics can be found in the drawings in **Appendix D**.

13.5 Traffic Study Summary

13.5.1 Scope

The DT conducted a traffic analysis report for the MBSD Project Area which included the highway and all of the intersections, commercial driveways, and median openings along Hwy 23 from Ravenna Road to the Plaquemines Parish Access Road. The study includes traffic counts, peak hours, and a safety study to ensure that the proposed bridge project over the MBSD will meet the capacity of future road demand. A copy of its current progress is included in **Appendix M**.

13.5.2 Field Visit

The DT conducted a field visit on June 1, 2018 to visually inspect the corridor. Within the project area, road and pavement markings including outside lane edge rumble strips were in good condition. During the visit, the DT reviewed the two main intersections along the corridor. A visual review at the intersection of Hwy 23 and Ravenna Road did not indicate any issues with line of sight or signs and pavement markings. W. Ravenna Road is a gravel road with no surface markings or signs while E. Ravenna Road is an asphalt paved roadway with signs and pavement markings. During the visit, East Ravenna Road was observed to have several commercial trucks making a left turn from Hwy 23. A visual review at the Intersection of Hwy 23 and Ironton Road also did not indicate any issues with line of sight, signage and marking, or artificial lighting. No queueing was observed at any of the intersections. The documentation of this visit can be found in the Traffic Study in **Appendix M**.

13.5.3 Analysis Summary

13.5.3.1 Peak Hours

As a part of the traffic analysis report, 7 day 24 hour and approach counts were taken in order to determine peak periods and peak hours for the corridor. A 7 day, 24 hour count was taken on Hwy 23 at the approximate location of the proposed bridge. Additionally, several approach counts were taken along Hwy 23; including four at the intersection of Hwy 23 and Ravenna Road, three at the intersection of Hwy 23 and Ironton Road, and three at the intersection of Hwy 23 and Plaquemines Parish Access Road. Using the traffic counts, the peak periods were determined to be 6:00 AM – 9:00 AM and 3:00 PM – 6:00 PM. The peak hours of the corridor were analyzed using the Tuesday, Wednesday, and Thursday counts and resulted in corridor peak hours of 6:45 AM – 7:45 AM and 4:00 PM – 5:00 PM.

13.5.3.2 Network Analysis Existing Conditions

An existing network analysis was conducted for the corridor. The network includes intersections and median turn-arounds from Ravenna Road to the Plaquemines Parish Access Road. A VISTRO model was created to analyze each intersection within the corridor and the Highway Capacity Manual (HCM) 6th Edition methodology was used for analysis and reporting. The analysis used the existing corridor geometry and the traffic counts that were collected in May 2018. The findings of the analysis are presented as delay values that are expressed by a grade based upon level of service (LOS) ranging from LOS A, the best, to LOS F, the worst. Generally, LOS D or better is acceptable. Hwy 23 at Ravenna Road and Ironton Road are the two main intersections along the study corridor. Hwy 23 at Ravenna Road is a four legged unsignalized intersection located within the northern limits of the project. The intersection resulted in an overall LOS B with the overall delay of 11.3 seconds. Hwy 23 at Ironton Road is a three-legged unsignalized intersection located within the southern limits of the project. The intersection resulted in an overall LOS B with a delay of 10.2 seconds. The analysis shows that throughout the corridor there is no LOS below B, thus the corridor operates at an acceptable LOS and there is no heavy queuing.

13.5.3.3 Safety Analysis

In order to identify trends and locations of past accidents along the corridor that can be used to propose countermeasures as part of alternatives that will improve the safety of the corridor, the DT performed a safety crash analysis. The DT reviewed the crash data from 2012 to 2016 within the LADOTD Crash 1 Database. The data was further analyzed using collision diagrams. There were 19 total crashes during the four year period. The most frequent crash type within the study area was non-collision crashes. Non-collision crashes accounted for 9 of the 18 recorded crashes. The non-collision crashes have a rate of 52.6% of the total crashes which is much higher than the state average of 18.8% for similar type roadways. The over represented crashes could potentially be due to the presence of wildlife crossing and the increased potential for vehicle-wildlife crashes. Of the nine non-collision crashes, five of them were animal related. In addition, there were two (2) crashes associated with a construction detour that was present during the time of the accident. The construction has since been completed. The second most frequent crashes were rear-end crashes which did not exceed the state average percentage of 38.5%. The three (3) other types of crashes were minimal in number in comparison to state averages. The addition of a bridge will provide some access control within the project limits that should reduce animals on the roadway. Construction sequencing will be reviewed with the CMAR contractor to determine opportunities to minimize accidents within the construction work zone and detour.

13.6 Detour and 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 Typical Plan and Section in the vicinity of the bridge construction is shown with the set of Drawings in **Appendix D**. A full definition of the maintenance of traffic will occur in coordination with the CMAR contractor in the next design phase.

The preliminary sequence of construction is as follows:

Phase I

1. Construct the construction detour crossovers.
2. Relocate utilities from the east side of right-of-way.
3. Reduce southbound traffic to one lane and shift southbound traffic to shoulder.
4. Shift northbound traffic to the southbound lanes.
5. Place surcharge fill for northbound ramps and levee road crossings on both sides of the Conveyance Channel

Phase II

1. Remove Hwy 23 northbound lanes and place fill for relocated Hwy 23 Roadway
2. Construct floodwalls on LADOTD right-of-way
3. Construct Hwy 23 Bridge, 24-inch waterline relocation on bridge, and relocated highway with median barrier from Station 397+00 to Station 409+03 and Station 430+79 to Station 438+00.
4. Construct northbound ramps on both sides on the Conveyance Channel.
5. Construct remaining segments of median barrier north and south of the Conveyance Channel
6. Shift Hwy 23 traffic to the bridge.

Phase III

1. Remove southbound Hwy 23 pavement from Station 393+00 to Station 405+00, Station 414+00 to Station 426+00, and Station 435+00 to Station 445+00.
2. Construct remaining floodwall across LADOTD right-of way.
3. Complete southbound roadway tie-ins and southbound ramp connections and tie-ins to the haul roads.
4. Place southbound roadway wearing course.

13.7 Bridge Structure

13.7.1 Structural Engineering

The proposed bridge structure begins at Station 409+03 and ends at Station 430+79, and overall length of 2,176 feet. It will consist of 17 spans that are 128 feet long each. The bridge clearance over the levee roads will be at least 16 feet, 6 inches at the high point of the roadway. The controlling clearance is 25 feet above a Water Elevation of 2 (NGVD) which is centered at Station 420+00. There is at least 7 feet of clearance above top of the Conveyance Channel floodwalls of 15.6 feet NGVD. A preliminary set of Type, Size, and Location Drawings can be found in **Appendix D**.

The superstructure would consist of an 8-inch concrete bridge deck with a 4-inch haunch and ten lines of 63-inch tall LG prestressed concrete girders. Concrete barrier rails will be the current standard 36-inch MASH compliant straight sloped barriers.

The substructure would consist of two controlling bent types. Outside the levee sections of the Conveyance Channel, the bents would have 42-inch diameter columns with a 48-inch by 48-inch bent cap. Inside the levee sections, the bents would consist of 60 inch diameter columns with a 72-inch by 72-inch cap.

Foundations are anticipated to consist of pile cap footings under each column with 5 steel H-piles. Outside the levees, the foundations will consist of strip footings with 4 H-piles. Further discussion of the pile foundations can be found in Chapter 9 and will be further refined upon completion of the borings during the design phase.

At grade abutments and approach slabs would be constructed to LADOTD standard details. The north abutment PGL is located at Station 409+03 and at EL 10.675. The south abutment PGL is located at Station 430+79 and at EL 11.244. Bearing seat elevations were also evaluated along with the haunch/girder interaction. The north abutment has the lowest bearing seat elevation of 3.016. LADOTD requires a minimum 4-inch concrete bearing seat. Assuming that the abutment cap would be around 30 inches tall that would mean that the top of pile elevations at the north abutment would be approximately EL 0.183. At the south abutment, the lowest bearing seat elevation is EL 3.840. Assuming that the abutment cap would be around 30 inches tall that would mean that the top of pile elevations at abutment 18 would be approximately EL 1.

13.7.2 Scour Analyses

Scour Analyses will be performed in Phase 2.

14. NOGC RAILROAD BRIDGE CROSSING

14.1 Introduction

The purpose of this chapter is to document the alternatives analysis of various proposed railroad alignments and railroad bridge options that were considered during the BOD phase of work.

The New Orleans Gulf Coast Railroad (NOGC), a subsidiary of the Rio Grande Pacific Corporation, is a 32-mile-long railroad that serves Jefferson and Plaquemines Parishes and interchanges with the Union Pacific Railroad (UPRR) in Westwego, Louisiana (1.5 miles east of the Avondale Yard). It is the only railroad operating on the Westbank of the metro New Orleans area. NOGC currently serves more than 20 switching and industrial customers including the Port of Plaquemines. Predominant shipments include a variety of food products, oils, grains, petroleum products, chemicals, and steel products. Major shippers on NOGC include Delta Terminal (Kinder Morgan), Chevron Oronite Division, and CHS Terminal Grain Elevator. For a substantial portion of its route, NOGC parallels and is immediately adjacent to Hwy 23. Currently, the railroad track terminates approximately 1,500 feet south of the centerline of the proposed Conveyance Channel. NOGC plans to extend the rail south upon agreements of future development that requires railroad service. Regardless, the current length of track needs to be maintained during construction of the Conveyance Channel in order to accommodate switching operations at the Alliance Refinery just north of the MBSD Project.

The 2014 Base Design realigned the railroad track to parallel Hwy 23. The Railroad Bridge would cross the MBSD Conveyance Channel immediately to the river side (east) of the Hwy 23 Bridge alignment. Both highway and rail would require bridges with similar span lengths to traverse the channel.

14.2 Summary of Conceptual Layouts

14.2.1 Design Criteria and Background

Several alternative alignments were considered in this phase of design. Drawings depicting the alternative alignments are **Appendix D**. The DT and CPRA proposed maintaining the rail on its current alignment. In a meeting attended by both NOGC and Rio Grande representatives, held on 15 Feb 2018, railroad personnel indicated that maintaining the current MRL alignment would also be their preference.

The vertical and horizontal alignments are designed in accordance with AREMA and UPRR design criteria. Alternatives were designed for a train speed of 25 MPH. The rail would span the Conveyance Channel supported on the walls of the Intake U-Frame structure. Of particular importance are the hydraulic criteria for bridges. In accordance with UPRR guidelines, the low chord shall be at or above the 50-Year flood event and the subgrade shall be placed at the 100-Year flood event. The subgrade is defined as being 2’-3” below the Top of Rail. The MBSD Intake Structure experiences both riverine and hurricane flood events. The greater of the riverine flowline and hurricane 50-Year Stillwater elevation is EL 14.6. The greater of the riverine design grade and hurricane 100-Year Still Water elevation is EL 17.6. The noted elevations include additional height for sea level rise. Alternatively, the bridge crossing could be placed within a flood proof bridge that would include floodwalls constructed to an elevation above the noted flood stages.

14.2.2 Alternative 1

Alternative 1 is taken from the 2014 BOD. The alignment turns out towards Hwy 23 and parallels the proposed Hwy 23 Bridge crossing. The top of rail over the Conveyance Channel is at EL 24.9, the low chord is at approx. EL 14. The low chord was based on providing 2 feet of clearance over the proposed floodwall. The top of floodwall was set at EL 10. The total length of the rail relocation is 8,520 linear feet, the total raised approach length is 4,330 linear feet, and the length of the main span Conveyance Channel crossing is 1,010 linear feet. The design grade was set at 1.5%. The alignment includes a reverse curve on the north side of the MBSD channel. The reason for turning the alignment out to Hwy 23 was not stated in the 2014 BOD Phase.

14.2.3 Alternative 2

Alternative 2 maintains the current MRL alignment. The low chord of EL 8 was set slightly above the water stage with the Mississippi River flowing at the Project design grade of 1,000,000 cfs. The top of rail over the diversion structure was set at EL 12.5. The low chord elevation was made possible by passing the rail through a flood proof bridge. The flood proof bridge walls will be built to the authorized riverine flood stage EL 16.4 or potentially to the higher hurricane grade at EL 20.1. The bridge spans will be built into and supported by the diversion Intake Structure. The lower rail elevation is preferred to minimize the rail relocation extending beyond the proposed MBSD ROW lines. There is consideration for making one of the spans removable. This would allow access for work barges as needed for future MBSD maintenance which would be an infrequent event. AREMA tunnel criteria and UPRR horizontal and vertical clearance was used to set the flood proof bridge geometry. The width of the bridge was increased to allow for a maintenance road. The maximum Grade was set at 1.0%. The total length of the relocated line is 2,980 linear feet, 1,200 linear feet will be a pile founded raised approach. The advantage of this alternative is its minimal impact to adjacent properties.

14.2.4 Alternatives 2b and 2c

Similar to Alternative 2, Alternatives 2b and 2c maintain the current MRL Alignment. Alternative 2b has a low chord at EL 20.1 which is at the highest Hurricane Design Grade under consideration. The top of rail is at EL 25.1. Alternative 2c has a low chord at EL 16.4 which is at the current, authorized Mississippi River Design Grade. The top of rail is at EL 21.4. Note that the current reach of the MR levees is only federally authorized as riverine protection and are no higher than EL 16.4. The level of flood protection acceptable to the USACE will dictate the selection. A floodproof bridge would not be required. The bridge would be supported by the Diversion Intake Structure piers. Several spans of approach ramps would be required before an earthen embankment could be used. The maximum Grade was set at 1.25% for both alternatives. The total length of Alternative 2b is 5,030 linear feet, the length of the pile founded raised approach is 3,100 linear feet. The total length of Alternative 2c is 4,400 linear feet, the length of the pile founded raised approach is 2,500 linear feet. Alternatives 2b and 2c do protrude further out into the adjacent property at 3,000 linear feet and 2,500 linear feet from channel centerline respectively. The benefit is that the flood proofing of the bridge would not be required as each would be above the selected Design Grade.

14.2.5 Alternative 3

Alternative 3 maintains the current MRL Alignment. The railroad track would be removed for construction of submerged culverts that would be the Intake Structure for the MBSD project. Upon construction, the track would be reconstructed along its existing alignment and grade. Since the submerged culverts were not selected for further design, this alternative is no longer under consideration.

14.2.6 Alternative 4

Alternative 4 consists of 2,200 feet of railroad track along the north side parallel to the MBSD Conveyance Channel. It was considered as an alternative to maintain the railroad switching operations at the Alliance refinery but would not provide an opportunity for NOGC to extend the track without extensive rework. It will be necessary to maintain railroad operations during construction of the Conveyance Channel.

14.3 Sequence of Construction

A preliminary sequence of construction would be as follows:

1. Prior to construction of the Intake Structure and the Conveyance Channel and levees, construct the temporary marshalling track along the north Conveyance Channel levee.
2. Cut and remove a 160-foot segment of the existing track at the intersection of the existing track and the temporary marshalling track.
3. Install the No. 10 turnout at the intersection location. Provide a lockout mechanism so that the existing track cannot be used.
4. Remove the remainder of the track in conflict with the Conveyance Channel.
5. Place embankment approaches on each side of the canal.
6. Upon completion of the U-channel Intake Structure, construct bridge spans and set on top of the U-Frame structure. Make sure the structure is watertight.
7. Construct subballast up to bridge structure.
8. Construction the approach slabs.
9. Lay first lift of ballast throughout the length of the new track.
10. Install ties and second lift of ballast.
11. Install rails by welding rail strings on site and install atop the ties up to the middle span. Install track from north to south on the north approach and south to north on the south approach.
12. Install the rails and the mitered joints for the removable span.
13. Install bumping post or hill.
14. Upon completion of the new track, remove the turnout and replace segment with straight track.
15. Remove the temporary marshalling track once new bridge track is in operation.

14.4 Recommendation

The DT recommended the floodproof bridge mainly because the overall length of the elevated bridge extended only slightly beyond the proposed Right of Way (ROW) boundary. The increase in rail height at the ROW boundary was only 8"above the existing grade. The lower floodproof bridge was also less expensive and allowed an at-grade crossing to the river within the proposed ROW. Along with the shorter approach lengths, fewer piles would be driven near the USACE Levee toe. The USACE has restrictions on piles near the levee toe and prohibits piles within the levee footprint. CPRA preferred the Alternative 2c which has the low chord on the wall top of the U-Frame structure at EL 16.4. At EL 16.4, the bridge would not need to include flood protection. The DT does not oppose the CPRA selection but notes that the cost is $23,210,500 greater than the floodproof bridge alternative and extends approximately 800 ft further past the proposed ROW. Note that the cost of the alternatives have increased significantly since Workshop No. 2. This was due to an increase in approach length as needed to comply with the latest railroad vertical curve criteria, and a second track was added at the request the NOGC Railroad. The increased costs do not exceed the cost of the 2014 Base Design alternative and do not alter the relative ranking of alternatives.

15. MECHANICAL ENGINEERING

15.1 Description of Gated Structure Mechanical Systems

There are two common types of gate lifting systems; electric wire rope hoists and hydraulic cylinders hoists. With either system, there is a hoist on either side of the gate which must be synchronized when lifting the gate. The DT is recommending the wire rope hoist system. The electric wire rope hoist can be done either mechanically with a common shaft or electrically. Mechanically, the hoist drums are synchronized by means of a line shaft connecting the machinery on each side. Electrically, they are synchronized by an electronic system with sensors, on the motor or gear drive which counts rotations and compares and adjusts the hoist motors speed required for synchronization. With the electric synchronizing system, two identical hoists, each with their own variable speed electric motor, brake and gear dive are required. Mechanically, synchronizing the wire rope drums using a line shaft reduces the number of hoist components such as gear drives, motors and brakes, since most of drive is located on one side of the gate and the torque needed to drive the opposite drum is transmitted by the synchronizing shaft. This arrangement reduces the overall capacity requirements of the hoists and construction and maintenance costs. The reliability of the synchronizing shaft is much higher than the electrical synchronizing system since it is not prone to failure from lighting strikes, energy surges, or environmental causes as is the electronic system. The drawback is the need for an overhead walkway bridge or girder to support the shaft. The gate will have infrequent operation, potentially not operating for 4-6 months per year during low water season. The low use, mechanical synchronization, ability to stop and lock the gate in any position without the reliance on a continuously operating drive unit, overwhelmingly favors use of an electric wire rope hoist with a synchronizing shaft. The DT is recommending the more reliable synchronizing shaft.

The drive system will be powered by a 15 HP electric motor located in each control house. The motors primary power source is commercial electric. Remote operation controls are located in the Safe House. A 50 KW back up diesel generator is included on the Safe House platform, dedicated to gate operations.

16. ELECTRICAL ENGINEERING

16.1 Description of Electrical Systems

The electrical power demands for the site will be provided primarily by the commercial utility provider, with standby power provided by multiple on-site generator sets (as described in Section 16.4 below).

Provisions for connecting a trailer-mounted or roll-up generator will also be provided.

Anticipated systems to be included in the design are power distribution (normal and standby), interior and exterior lighting, grounding, and lightning protection systems.

16.2 Electrical Site Distribution

While final loads are yet to be determined, a preliminary load tally has been developed, and it is envisioned that a 200 kVA, 3-phase service will be brought to the site for power distribution. Given the relatively small total power requirement, it is proposed that service be taken at 208-volts, 3-phase, 600-amps, so that intermediate transformation from 480-volts to 208Y/120-volts will not be required, thus saving floor space and equipment costs. This decision will be revisited as the design progresses.

Main power distribution equipment will be installed at a minimum elevation of 6 inches above the base flood elevation, or 3 feet above the highest existing adjacent grade (HEAG), whichever is higher. The equipment will either be located in the Admin Building, the Shops Building, or in a separate Utility / Generator Building. Final decision on equipment location will be based on final site / building layouts, building and floor slab elevations, and building construction types. If a separate utility building is selected, the building will be rated to withstand 150 MPH sustained winds, minimum.

Based on the total preliminary electrical loads, it is anticipated that service will originate from pole-mounted transformers. Service to the main distribution equipment will be routed underground. Electrical service will be in accordance with Entergy requirements.

From the main distribution equipment, power will be distributed underground to the gate structure, support buildings, Safe House, and the emergency crane, should one be required. Distribution equipment for the gate structure will be located within the Diversion Gate Control House.

16.3 Lighting

16.3.1 Gate Structure

Marine Grade, LED floodlights mounted to and / or near the gate structure will provide illumination of the gates for night observation. The gate structure access walkways will be illuminated by marine grade, stanchion-mounted LED fixtures. Floodlights and access walkway lights will be controlled manually and independently from one another via local on/off switches.

Design will include provisions for manually controlling gate structure flood lighting (on/off) from the SCADA system.

16.3.2 Control House and Gate Structure Approach

Walkways to the control house(s) and gate structure will be illuminated by a combination of exterior, marine grade, wall-mounted LED "wall packs" and pole-mounted, marine grade, LED area lighting fixtures. Fixtures will be designed to illuminate approaches to an average of 1 FC along the approach path and will be automatically controlled by a photocell.

16.3.3 Control House

Industrial, surface-mount, IP67-rated LED fixtures will be specified for interior lighting of the control house. Lighting controls will be manual-only. For operator safety, UL924, battery-powered emergency lighting will be specified for the interior of the Control House to provide up to 90-minutes of illumination in the absence of utility or generator power.

An exterior, marine-grade, photocell-controlled LED wall pack located over the entrance door will provide entry/exit lighting. In the absence of utility or generator power, a wall pack will provide up to 90 minutes of emergency egress lighting. Power for egress lighting will originate from an internal battery or separate inverter.

16.3.4 Administration Building

Interior lighting will consist mainly of recessed 2 feet by 4 feet LED fixtures controlled by a combination of manual toggle switches and occupancy sensors. Additional lighting over the conference table may be considered if presentations are expected in the Conference Room.

Exterior, marine-grade, photocell-controlled LED wall packs located over or adjacent to each entrance / exit door will provide entry/exit lighting. In the absence of utility or generator power, wall packs will provide up to 90 minutes of emergency egress lighting. Power for egress lighting will originate from an internal battery or separate inverter.

16.3.5 Shop Building

Interior lighting will consist mainly of surface- or chain-mounted, industrial style LED fixtures in work areas and recessed 2 feet by 4 feet LED fixtures in administrative areas. Lighting will be controlled by a combination of manual toggle switches and occupancy sensors. Additional lighting over the conference table may be considered if presentations are expected in the Conference Room. Boat shed lighting will consist of industrial, surface-mount, IP67-rated LED fixtures. Special lighting requirements for the Soils Lab will determined as the designs progress.

Exterior, marine-grade, photocell-controlled LED wall packs located over or adjacent to each entrance / exit door will provide entry/exit lighting. In the absence of utility or generator power, wall packs will provide up to 90 minutes of emergency egress lighting. Power for egress lighting will originate from an internal battery or separate inverter.

16.3.6 Generator Building

Interior lighting will consist mainly of surface- or chain-mounted, industrial style LED fixtures. Lighting controls will be manual-only.

Exterior, marine-grade, photocell-controlled LED wall packs located over or adjacent to each entrance / exit door will provide entry/exit lighting. In the absence of utility or generator power, wall packs will provide up to 90 minutes of emergency egress lighting. Power for egress lighting will originate from an internal battery or separate inverter.

16.3.7 Obstruction Lighting

The need for obstruction lighting will be evaluated once cofferdam and gate structure drawings progress and the need for a communication tower is determined.

16.4 Power (Identified Electrical Loads)

16.4.1 Gate Structure

Other than the power required to the Gate Structure for the gate motors, general purpose receptacles will be located near each gate motor gear operator for connection of a portable drill.

16.4.2 Control House and Gate Structure Approach

Walkways to the control house(s) and gate structure will include general purpose receptacles.

16.4.3 Control House

Loads for the Control House will include general purpose receptacles for service, power for the Gate Controls and SCADA System UPS, power for ventilation, and power for the Surveillance System associated with remote operation of the diversion structure.

16.4.4 Administration Building

Identified electrical loads include general purpose receptacles, Communication / Ethernet equipment, air-conditioning and heating equipment, Security Systems, site lighting, reproduction equipment (copy machine), and standard Break Room appliances (refrigerator, microwave, coffee maker). Total load is estimated at 20 kW.

16.4.5 Shop Building

Loads within the Shops Building as less defined than in other buildings at this time. We have currently estimated the load of the building to be around 50 kW, which includes general purpose receptacles, ventilation (for the service bays, work areas, and shed), air-conditioning and heating (for the administrative areas and the Soils Lab), a 5 ton hoist, a 5 HP air compressor, a drill press, and an arc-welder.

16.4.6 Generator Building

Other than power required for generator auxiliary systems (heaters and battery chargers), power is limited to general purpose receptacles for service.

16.5 Standby Generators

16.5.1 Standby Power System Overview

At this time, it is anticipated that two generators will be included in the design: one dedicated to the Diversion Gate Structure motors, and the other for the remaining critical loads in the Administration Building, Shop Building, and Safe House.

Generators will be housed within a building having a slab elevation equal to that of the Gate Structure Control House or Safe House, whichever is higher. The building will be rated to withstand a minimum of 150 MPH sustained winds.

Any and all required generators will be configured as separately derived systems, and associated transfer equipment will be 4-pole (neutral-switching).

Design will include provisions for connection of a portable (roll-up) generator.

A generator building will be provided. If a diesel engine generator is selected, the fuel tank will be designed to UL 2085 (ballistic-rated). A tank sized to provide a minimum of 72 hours of run time at full load will be required, but a larger tank may be desired, based on tank accessibility post-storm. If a diesel engine generator is selected, a fuel polishing system will also be specified to keep fuel fresh.

16.5.2 Standby Generator Set for Gate Motors

At present, it is anticipated that the gate structure will consist of 3 gates, each driven by a single 15 HP electric motor through a cable assembly. Operating mode during loss of utility power will be on a gate-by-gate basis. That is to say, only one gate motor will be operated at a time when operating on backup generator power.

In order to start and operate a single gate motor, a 35 kW generator set is required. Generator set size could potentially be reduced, depending on the motor starting method selected. However, for the purposes of this preliminary sizing exercise, across-the-line motor starting was assumed.

Generator set controls will be configured such that the generator will automatically start only when gate operation is necessary. For extended utility power outages (greater than 12 hours), a generator exerciser circuit will automatically start (exercise) the generator once each day for 30 minutes (time and duration programmable) so that generator set batteries can remain charged and the generator set controls can remain functional.

The generator set will be diesel-fueled and have a sub-base tank sized for 12 hours of run time at full load. Anticipating a gate travel time of no more than 2 hours each, a tank sized for 12 hours at full load would allow for two complete operations of each gate. Locating the tank below the generator set will eliminate the need for additional fuel distribution equipment and controls from a separate tank.

To keep the diesel fuel fresh, a Fuel Polishing system will be specified.

16.5.3 Standby Generator Set for Safe House and Other Critical Loads

The following loads have been identified for connection to this standby power source:

- All Safe House Loads: Required load is estimated to be around 25 kW, with roughly 15 of the 25 kW coming from an instantaneous water heater. If a standard tank heater is used, then the load requirement can be reduced.
- Gate Structure Ancillary Loads: Flood Lighting and Access Walkway lighting; current estimate of load is 2 kW.
- Control House: Required load is estimated to be 2.5 kW and consists of the Gate Controls (including the SCADA System), surveillance cameras for remote gate operation, Control House ventilation, and Control House lighting.

- Administration Building: Selected receptacles for PCs, telephones, and other network equipment, in addition to emergency lighting circuits and power for security systems (access control and CCTV) make up the loads identified in this building for generator backup. The total estimate of these loads is 6.5 kW.
- Shop Building: Interior emergency lighting in the building, estimated at 3 kW, is the only load in the building currently identified for backup power.

Based on the identified loads, a minimum generator size of 40 kW is required. However, based on expected minimum loads, a 50 kW generator is recommended. This will provide some additional capacity and, with selected loads temporarily disabled or not used (such as the instantaneous water heater), can also act as a secondary backup source for the gate structure.

Generator set controls will be standard, configured to start the generator set whenever a utility power loss is sensed, and keep the generator set operational until utility power is restored and cool-down cycles are complete.

The generator set will be diesel-fueled and have a sub-base tank sized for 72 hours of run time at full load. Locating the tank below the generator set will eliminate the need for additional fuel distribution equipment and controls from a separate tank. An access platform, the top of which will be set to the same elevation as the top of the fuel tank, will be specified to facilitate service and maintenance of the generator set.

To keep the diesel fuel fresh, a Fuel Polishing system will be specified.

16.6 Gate Drive System

Gate drive system type will be electric. Power and control of the gate motors will originate from a motor control center located in the Control House for the gate structure. A transfer switch will be located within or adjacent to the motor control center for automatic starting and transfer of power to the dedicated standby source whenever gate operation is required.

16.7 Grounding and Lightning Protection

A lightning protection system will be specified for all buildings and enclosures that house electrical distribution equipment, including the gate control house(s). Ground rings will be specified around buildings housing distribution equipment, electrical services, and outdoor generating equipment.

All electrical equipment will be grounded. The fuel tank, if required, will be bonded to the grounding system, and a ground ring will be specified around it as well.

Distribution equipment will be specified with integral surge suppression to mitigate damage from voltage transients.

17. INSTRUMENTATION AND CONTROLS

17.1 Gate Structure Instrumentation and Controls

Control power will be either 120-volts AC or 24-volts DC (voltage to be determined). In either case, standby generator power, and redundant sources of power for the controls, will be specified. Redundant sources will consist of redundant power supplies and UPS backup.

17.2 Diversion Gate Structure Instrumentation and Controls and Back Gate Actuation

Controls will be PLC-based with a manual, hard-wired backup system, should the PLC fail. Design will attempt to limit any single point of failure. Gate position will be monitored by limit switches; type of limit switch to be specified will depend in large part upon the gate geometry, construction, and machinery, and thus has not yet been selected. Gates will be able to be controlled locally from either the Control Room or the Safe House. A requirement for remote (off-site) control is not anticipated at this time.

17.3 Control Room(s)

Gate control equipment and local operator interfaces will be housed within the adjacent control room. Basic operator interfaces (pushbuttons and indicating lamps) are anticipated, since control will be limited to opening and closing of each gate. LED indicators will be specified for each gate position and each monitored alarm condition.

17.4 SCADA and Communication System

It is anticipated that a SCADA system will be specified for the ability to remotely monitor alarms and various river and basin conditions; however, a finalized list of conditions to be monitored has not yet been developed. We further anticipate that the SCADA system will connect to the PLC gate controls for monitoring of gate positions, and that off-site communication will be achieved via Ethernet communication modules for connection to a utility-provided “Metro Ethernet”.

It is still undetermined at this time if the SCADA System will be part of another, existing system, or a new, stand-alone system. In either case, design will specify coordination of the SCADA System for this structure, particularly the user interfaces and HMI, with the one installed at the Mid-Breton Diversion Structure to provide a single, consistent user interface. Furthermore, the details of this system will likely not be fully addressed until the 60% submittal phase.

17.5 Surveillance System

IP-based, pan-tilt-zoom surveillance cameras will be included in the design. The cameras will serve the purpose of securing the reservation with the option of providing internet-based, live visual images of the reservation for remote viewing.

17.6 Access Control Room System

The need for an access control system is yet to be determined. If required, the system will provide dry contact outputs to the SCADA System for remote alarm monitoring.

17.7 Alarm Systems Emergency Power

The SCADA system will be used to transmit alarms offsite. It is anticipated that the following alarms will be monitored:

- Gate controls not in PLC Mode.
- Gate open/close timeout (if a gate does not fully open or close, as indicated by limit switch, within a set time period).
- Loss of utility power.
- Loss of control power (on battery backup).
- Generator low fuel.
- Generator fuel leak detection.
- Generator engine alarms and pre-alarms, including those for low oil pressure, high oil pressure, low coolant level, high coolant temperature, over-speed, and over-crank.
- Generator starting system alarms, including low battery voltage and battery charger failure.
- Generator controls not in auto.

18. ARCHITECTURE

This work will be performed in the 30% design phase.

19. UTILITY RELOCATIONS

19.1 General

Utility relocations are often required during the construction of new civil works projects, and relocations can either be permanent or temporary based on the construction proposed. DT will work with the CMAR and the PMT to identify all utility conflicts within the proposed construction limits and any conflicts within the temporary workspace, which includes both access routes and temporary laydown areas. Below is a list of steps that should be performed to initiate a relocation.

- Identify all the utilities within the project area by:
  - Performing a desktop survey utilizing GIS and in-house data. Websites to be used for MBSD include:
    - Sonris GIS Database
    - DT in-house data
    - 2014 Baseline Report
  - Develop list of utility owners within the proposed MBSD Right-of-Way and the temporary workspace for the project
  - Obtain existing Right-of-Way plats, documents, and as-built data on all utilities identified in the MBSD construction limits.
  - Perform a site visit to confirm utility location per the desktop study
  - Develop a survey plan to obtain/confirm detailed information on utility location and depth.
    - Survey plan shall follow Subsurface Utility Engineering (SUE) standards of practice per ASCE 38-02
- Once MBSD project alternatives are selected, develop a Base Plan showing topographic data and the MBSD Project features for Project Owner to use for initial contact with utility owner
  - A request should be made to utility owner to provide a mark-up of their utility location.
- Develop a MBSD Project Fact Sheet providing utility owners with general project information.
- Develop priority list based on critical path relocations. This list shall be further developed in conjunction with CMAR during the 30% Design Phase

19.2 Coordination w/ Owners

Initial contact with utility owners was made during the 15% BOD phase to determine their point of contact and if their utility is within the MBSD project area. Contact only involved emails and/or phone. No face to face meetings have been performed. Once the primary alternatives for the intake configuration and invert and the channel geometry are determined, a kickoff meeting with utility owners will be scheduled.

The plan and schedule for the utility coordination is to be refined during the 30% Design Phase. Initial plan for contact should include:

- Either DT, on behalf of CPRA, or the CPRA Team shall submit the Base Plan and Project Fact Sheet developed during the 15% BOD Phase to the utility owner.
  - In this submittal, a face-to-face kickoff meeting should be requested.
- Develop conceptual mitigation plan to be presented to utility owners during kickoff meeting.
  - Relocation plan (permanent and/or temporary)
  - Pipeline protection plans (i.e. air bridge, casing, structural fill, etc.)

- Develop schedule for MBSD project features and requested schedule for relocation
    - Determine path forward by DT, CPRA Team, and utility owners to achieve schedule.
- Document all utility relocation meetings (external and internal)

The plan for additional meetings will be planned during the 30% Design Phase based on initial kickoff meeting.

19.3 Dispositions

Below is a list of potential utility companies located in the project area. These utilities will be assessed during the site visit and developed into a prioritized list.

Table 19-1: Utility List

Utility Type	Owner	Description
Electric	Entergy	A distribution line on each side of Hwy 23
Electric	Entergy	A transmission line on the west side with steel poles
Water	Plaquemines Parish	20" PVC beginning south of W. Ravenna Road & running west side of Hwy 23
Water	Plaquemines Parish	16" AC running on west side of Hwy 23
Water	Plaquemines Parish	A windmill / water well to be capped
Water	Inframark Services	16" AC line
Pipeline	Shell Pipeline Co.	20" Nairn to Norco Pipeline - Crude
Pipeline	High Point Gas Transmission	12" Natural Gas
Pipeline	Harvest Midstream	Line north of project site, near the Alliance Refinery
Pipeline	Chalmette La Liquids/ Sulphur River Exploration	16” Propylene Line
Pipeline	American Midstream Assets	12” Gas Pipeline
Communications	AT&T Communication	Fiber optic and copper telephone cables
Communication	CMA Communications	Fiber optic and coaxial cables

Table 19-2: Utility Contact List

Utility Company	Contact	Phone Number	E-mail
AT&T Communication	Barry Barrillaux	504-364-6807	bb0533@att.com
ATMOS Energy	Brian Blum	504-425-4799 504-214-6356 (c)	brian.blum@atmosenergy.com
CMA Communications	Darren Guillot	504-669-9623 (c)	darren.guillot@cableone.biz
CMA Communications	David Herring		david.herring@cableone.biz
Inframark Services	Troy Phillips	504-392-4177 504-912-2673 (c)	troy.phillips@inframark.com
Entergy Distribution	Mike Kenny	504-365-2984	mkenny@entergy.com
Entergy Overhead Transmission	Jimmy Sholar	504-219-4204	JSHOLAR@entergy.com
Shell Pipeline Co.,LP	Tammy Pimley	504-425-4799	
American Midstream Assets	Dan Fayard	504-849-2217 985-807-8272 (c)	dan.fayard@jacobs.com
Chalmette La Liquids/ Sulphur River Exploration	Greg Vujovich	214-373-1091 214-505-4849 (c)	gvujovich@sulphurriver.com
Harvest Midstream	Tony Arellano	504-912-4426	aarellano@harvestmidstream.com
Rio Grande Pacific/ New Orleans & Gulf Coast Railway	Johnny Hydes	504-458-1075	
Rio Grande Pacific/ New Orleans & Gulf Coast Railway	Matthew Mattiza	817-737-5885 ext 3122	mmattiza@rgpc.com

20. SECONDARY SITE FEATURES

20.1 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, access roadways, levee access (roadways) and a boat launch etc. 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 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 slab total dimensions will be approximately 274’ x 160’ and will require approximately 6 feet of fill embankment to bring the final parking/drive grade from existing (EL 4.0 +/-) to approximate EL 10.5 around the buildings and to BFE of EL 10 at perimeter (low point). The entire reservation fill area will be considered for surcharging or wick drained to be determined by geotechnical analysis.

Also, included will be subsurface drainage structures (catch basins, drop inlets, RCP culverts approximately 15 inches to 48 inches diameter) through the concrete area to a drainage ditch outfall 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 on 12-inch stone aggregate and 12-inch compacted sand subbase with swale drainage from Hwy 23.

20.2 Buildings

The reservation site will include several buildings on pile supported slab on grade at assumed EL 11.5 (BFE=10), including a safe house structure with fuel tank platform and control house structures described as such:

1. Diversion Gated Structure (Control House) located on the conveyance structure walls above the gate apparatus; Access to the Control houses and structure decks are not ADA compliant. Control house dimensions are being determined and include gate machinery and control panel. Given the infrequent operation, controls are not to be extended to the adjacent admin building.

SCADA monitoring will be connected to the Western Closure Complex. Neither potable water nor restrooms are going to be present at the Control House units.

2. Administration Building. We recommend accommodations for staffing the entire year; the admin building will include two offices, admin office/reception area, a small conference area, kitchenette and restrooms. The admin building is separate building but directly adjacent to and connected to the shops building with a common firewall. ADA compliant. The structure is to be brick veneer with standing seam metal hip roof approximately 1,500 square feet included on reservation at EL 11.5.

3. Shops (Operation & Maintenance) Building. The shops building will be operational all year, with an increase in staff during the 6-month operation period. Structure will be metal building with R panel exterior and will include a service bay, work area, conference area with kitchen, and restrooms. The shops building shall also include shed area for lawn equipment and a boat shed and room for a soils lab of 12 feet by 20 feet. Two stories, approximately 70 feet by 70 feet. There will be an overhead 5-ton crane in shop. No provisions are included for sleeping quarters. It is assumed that a second shift can be added to man the structure during periods of peak operation.

4. Safe House- The safe house will be built at the riverside of the shop building not at the gate and at the MRL Design Grade EL 16.4; will be sized to contain 3 beds, a small work area, restrooms and the remote gate control panel. The safe house shall be approximately 600 square feet in area; the 2 diesel generators for backup power (20KW-safe house, 60KW-gate back up) located on the safe house platform and feed off the same 7-day fuel tank also on the safe house elevated platform. Additional fuel supply will be placed at the BFE (EL 10) and vented above EL 16.4. Safe house will be located as to have a clear site vision in both directions and impact resistant windows rated for hurricane and bullet proof protection.

20.3 Ancillary Site Features

1. Back Structure. The back structure has been removed from project (for now). There will be a platform for a soils lab required for the sediment flume and soils lab located between the parking area and dock. A parking area and a boat ramp are located on the north side in line with the existing back levee. The Platform and parking area will be built up to EL 10. The boat ramp and access will be designed to accommodate a 35-foot boat and turn around area same.

2. Site layout. The access roads from Hwy 23 will be designed as 2-inch asphalt wearing on 12-inch thick aggregate surface 24-foot wide with geogrid and geotextile fabric with 12-inch minimum sand subbase on the south side of levee crown on south side of the conveyance. The access road will tie into the local road that parallels Hwy 23 approximately 3,000 feet to the west. Layouts for facility utilities, fire protection, and security fencing will be included on the site work drawings. West of Hwy 23 access for operation and maintenance shall be along the levee crown. The levee crown shall be asphalt.

3. Boat Dock. The ramp shall be 20 feet wide and constructed of 8-inch concrete slab on 18-inch noncompacted aggregate and 6-inch concrete precast panels submerged. The ramp will extend to EL -8 (+/-) in the river. Access will be from a 15-foot wide access 12-inch thick limestone surface path that extends from the Boat Ramp and traverse the MRL at a 10% vertical grade. The access ramp extends to the access bridge.

4. Access Bridge over the conveyance structure will be a 24-foot wide prestressed concrete bridge that extends over, and is supported on, the gated structure. The low chord rests on the top of wall at EL 16.4. The center span shall be designed as removable. The access bridge shall be designed to support a 300 Ton crane at an operating loading. The access bridge ends at the BFE = EL 10.0.

21. ANTICIPATED CONSTRUCTION METHODS

The major diversion component alternatives selected during BOD Phase are: Open Channel Intake with the invert EL -40, a Diversion Gate Structure with tainter gates, an earthen trapezoidal Conveyance Channel with a constant invert EL -25 and with guide levees dual-purposed to also serve as Hurricane Protection, and a 1,500-foot long Outfall Transition Feature. The DT's opinion is that the Open Channel Intake's training walls that extend into the Mississippi River will be constructed inside of localized, braced retaining structures by casting in place the reinforced concrete walls. Certain segments of these walls may also be constructed by lifting precast concrete components into place and connecting them in the wet. The specifics will be determined with the CMAR, after the CMAR joins the project. Floating concrete components into position was a potential construction technique for the U-Frame alternatives, which were eliminated during BOD Phase. While the Open Channel Alternative also has a U-Frame segment, it starts at the MRL and extends to the Diversion Gate Structure, which is located landward of the MRL. These HW components and the Transition Segment to the Conveyance Channel will be constructed inside an open, dewatered excavation and behind a structural cofferdam near the MRL. The DT anticipates that these components will be constructed using traditional cast-in-place techniques.

The DT has designed conceptually the Conveyance Channel excavation to be constructed either fully in the wet using a combination of drag lines and bucket-dredging, or by excavating some portion by using draglines to remove the upper organic layers, and then progressively dewatering while further excavating with mechanical excavators, and completing the excavation by bucket-dredging. The DT will tailor the detailed design to accommodate the CMAR's means and methods during Phase 2. Channel armoring likely will be placed in the wet. T-Wall segments within the Conveyance Channel limits are expected to be constructed using traditional cast-in-place techniques with either open cutting localized excavations or by installing structural shoring systems. The closures of the Timber Canal and the Back Levee Canal are anticipated to be constructed by the placement of a combination of granular core and a concrete blanket cap placed in the wet. The Timber Canal closures also can be constructed by placement of temporary dikes and dewatering. In the dry construction of the Back Levee Canal closures likely will destabilize the existing NOV Levee.

Certain section of the Hurricane/Guide Levees will be constructed using a subsurface wick drain system with overlying drainage layer and staged construction to accelerate consolidation of the in-situ subsurface soils and construction with a planned overbuild to account for the majority of predicted future settlement. Other sections may be constructed without the need for wick drains, but preloading will be required. The construction technique selection will be schedule-driven.

The DT anticipates that the inverted siphon will be constructed with a combination of open cut excavations and structural shoring systems. The siphon inlet and outlet will be constructed with traditional cast-in-place techniques.

The Outfall Transition Feature will be constructed by dredging and placement of riprap armoring in the wet.

The other major project features are the two bridges. The Hwy 23 Bridge will be constructed using standard bridge construction techniques. The pilings supporting intermediate piers/bents within the Conveyance Channel may be installed using a follower prior to excavation, or may be installed after excavating the Conveyance Channel at this location. This can be done either way, but will influence the layout of the Hwy 23 detour. The design of the piers/bents will be driven by the Contractor's intended methods of their construction. Designs can be developed that allow their construction without having to dewater the Channel excavation, but must not produce unacceptable head losses to diversion flows. Girders will be standard AASHTO type precast girders and the deck will be cast in place. The railroad bridge superstructure, assuming it is located at or above the MRL authorized crown will likely be steel but the deck on which the rails will be installed potentially being concrete. The concrete would be cast in place. Pile-supported ridge approach segments will likely be precast concrete or steel in accordance with UP standards.

22. EARLY CONSTRUCTION OPPORTUNITIES

In an effort to reduce the overall construction schedule, the DT investigated several early construction start opportunities that could potentially begin prior to or in parallel with major project construction operations. Early construction opportunities may benefit the project by allowing the CMAR team to increase efficiency of the review and construction processes for non-critical path items; by taking advantage of good weather to avoid slipping the overall project schedule; and by saving 4% of the construction cost annually, considering deflation. The process of identifying these opportunities included consideration for each opportunity's estimated early design start date and duration, early construction start date and duration, prerequisites required, advantages, disadvantages, potential cost savings, overall schedule reduction and risk reduction to the project. CPRA provided information regarding each opportunity's EIS impacts and right-of-way status. The DT estimated rough order-of-magnitude construction costs for each opportunity.

At the beginning of the BOD Phase, the DT identified sixteen early construction opportunities, which are described in the Early Construction Opportunities memo and table presented in **Appendix S**. During review, the project management team determined that most of these opportunities were subject to Section 10/404 permits and Section 408 permissions, which eliminated them from early construction consideration. The only viable early construction opportunities include pile load testing, partial purchasing of pile materials and pre-purchasing of electrical and mechanical equipment/materials.

23. ENGINEER'S CONSTRUCTION COST ESTIMATES

23.1 Cost Estimating Methodology and Assumptions

The construction cost estimate for the MBSD Project was developed using Microsoft Excel and generally uses the standard approaches for estimate structure regarding labor, equipment, materials, crews, unit prices, quotes, sub- and prime contractor markups. Developed costs were supplemented with quotes bid data, and AE estimates. The costs for project features not conceptually designed during the BOD Phase were estimated using the 2014 Basis of Design's cost estimate, escalated to the projected mid-point of construction. The intent is to provide or convey a "fair and reasonable" estimate that depicts the local market conditions.

During the BOD Phase, first, Class 5 comparative construction cost estimates were developed to facilitate alternatives analyses of the individual major project features for use in populating the decision matrices developed for Alternatives Workshop No. 2. The overall project cost was estimated by combining the costs of the selected alternatives for the purpose of evaluating whether overall cost was increasing, and if so whether sufficient construction funds are available. After selecting the preferred alternatives for each of the major diversion components, a Class 3 cost estimate was developed for the entire project. A description of the methodology and assumptions of the cost estimating effort is contained herein.

The construction site is located in Southern Louisiana and is accessible from either land, water, or both depending on the project feature. For land access, the region is accessible from Hwy 23, as appropriate for each project location. From water, access is available via the Mississippi River and the Barataria Basin.

All anticipated construction work is common to south Louisiana. In addition, all major construction materials - including structural steel and concrete, steel sheet piling and pipe, and steel and concrete piling are readily available. All earthen fill is obtained from local borrow (either truck-hauled or adjacent borrow). The riprap and bedding material can be barged to the site and placed directly or off-loaded, and truck-hauled for placement. Material cost quotes are used on major construction items when available. Recent quotes include concrete, steel and concrete piling, rock, gravel and sand.

Local and state taxes are applied to materials. The work will be performed in Plaquemines Parish which has a tax rate of 8.95%.

It is assumed that there will not be an economically saturated market. It is known at this time that the contract acquisition strategy is a CMAR contract.

In regards to labor shortages, it is assumed there will be a normal labor market and there will be no issues finding the required labor to complete the job. The local labor market wages are above the local Davis-Bacon Wage Determination. Labor rates used are based upon local information and payroll data available to estimators with experience in this type of construction in the local area.

Major crew and productivity rates are developed by estimators familiar with this type of work. When appropriate, R.S. Means was also referenced. All of the work is typical to South Louisiana. Major crews include clearing and grubbing, hauling, earthwork, piling, and concrete. Most crew work hours are assumed to be 10 hours/day 6 days/week which is typical to large scale civil works-type projects in the area.

Equipment rates used are based from the latest U.S. Army Corps of Engineers Equipment Manual, EP-1110-1-8, Region III. Adjustments are made for fuel and facility capital cost of money (FCCM). Reasonable use of owned versus rental rates was considered based on typical contractor usage and local equipment availability. Fuel costs (gasoline, on and off-road diesel) were based on local market averages for on-road and off-road use in Plaquemines Parish, Louisiana.

Mobilization and demobilization costs are based on the assumption that most of the contractors will be coming from within South Louisiana or the Gulf Coast region. For the cost estimates 3% of the construction cost was used to account for mobilization and demobilization.

Bond is estimated to be 1% of the construction costs.

The estimate uses a field office overhead rate of 10%. This number is based on historical studies and experience for similar civil works type of construction. Field office overhead includes: superintendent, office manager, pickups, periodic travel, costs, communications, temporary offices (contractor and government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp and kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project signs, traffic control, surveys, temp fuel tank station, generators, compressors, lighting, and minor miscellaneous.

For the Class 5 cost estimate, profit was estimated to be 8% which was considered reasonable when taking into account the degree of risk, difficulty of the work, the Contractor's investment, the size of the job, and the period of performance.

For the Class 5 cost estimate, home office overhead was estimated to be 7%. This number was based upon estimating and negotiating experience, and consultation with local construction representatives.

For the Class 3 cost estimate, additional information was available from the CMAR which resulted in using a combined profit and home office overhead rate of 10%.

For the Class 3 cost estimate, guidance was provided from the CMAR on what work would be subcontracted out. The estimate includes a 2% support markup on items being subcontracted out based on input from the CMAR.

The estimate includes no costs for any potential Hazardous, Toxic, and Radioactive Waste (HTRW) concerns.

Relocation costs are defined as the relocation of utilities required for project purposes. In cases where potential significant impacts were known, costs were included within the cost estimate.

Cost associated with EDC as well as supervision and administration during construction (S&A) is not included in the Class 5 cost estimate because it does not affect the alternatives analysis. EDC and S&A are not included in the Class 3 estimate of the selected alternative.

BOD Phase estimated construction costs do not address potential upsizing of the River Intake, the necessity of which has not yet been determined.

23.1.1 Cost Estimates for Alternatives Workshop No. 2

The cost estimates for the MBSD Project's Alternatives Workshop No. 2 are considered to Class 5 type estimates that were prepared utilizing Microsoft Excel. These cost estimates are considered to be rough-order-of-magnitude estimates, although some of the alternatives were further along in the design process than others, and some previous cost estimates were available which improve estimate reliability. Class 5 estimated costs used for the alternatives screening process are relative comparisons on a component-specific basis. Only cost differentiators associated with the individual alternatives being compared where included.

According to the American Association of Cost Engineers as referenced in the CPRA Mississippi River Mid-Basin Sediment Diversion Program: Cost Estimating Plan, the requested Class 5 level of estimate is defined as follows:

> "Class 5: These estimates are prepared based on limited information. Class 5 estimates generally use stochastic estimating methods such as cost/capacity curves and factors, scale of operations factors, and other parametric and modeling techniques. The typical expected accuracy range for this class estimate is –20 to –50 percent on the low side and +30 to +100 percent on the high side."

23.1.2 Cost Estimate for Selected Alternative

Class 3 type cost estimates were prepared for the selected alternative in the BOD Phase using Microsoft Excel which will address specific construction procedures for the various line items in the estimate to as much detail as could reasonably be developed within the task order's schedule. The estimated costs are based upon an analysis of each line item evaluating quantity, production rate, time, equipment, labor, materials and supplies. The cost estimates reflect current and applicable pricing. These estimates, as well as a summary of the O&M costs for the selected alternative, are provided in **Appendix F.**

According to the American Association of Cost Engineers, as referenced in the CPRA Mississippi River Mid-Basin Sediment Diversion Program: Cost Estimating Plan, the requested Class 3 level of estimate is defined as follows:

> "Class 3: These estimates are generally prepared to form the basis for budget authorization, appropriation, and/or funding. As such, they typically form the initial control estimate against which all actual costs and resources will be monitored. Class 3 estimates generally involve more deterministic estimating methods than stochastic methods. They usually involve predominant use of unit cost line items, although these may be at an assembly level of detail rather than individual components. Factoring and other stochastic methods may be used to estimate less-significant areas of the project. The typical expected accuracy range for this class estimate is -10 to -20 percent on the low side and +10 to +30 percent on the high side."

23.1.3 Cost Comparison for MRL and Guide Levee Design Grades

In evaluating design grade alternatives for the Conveyance Channel Levees and the MRL, the DT compared the estimated construction costs for each. The design grades considered for the Conveyance Channel Levees were EL 15.6 and EL 12.1; the EL 15.6 alternative is approximately $20 million more expensive than the EL 12.1 alternative. For the MRL, design grades of EL 20.1 and EL 16.4 were evaluated; the EL 20.1 alternative is approximately $3.7 million more expensive than the EL 16.4 alternative. See **Appendix F** for details of the cost comparisons.

23.1.4 Cost Escalation in the Mid-Barataria Sediment Diversion Cost Estimates

Early in the BOD Phase, the DT submitted a Technical Memo entitled “Cost Escalation Factors” to the Project Management Team. This memo explained the process of determining escalation factors and included a recommendation of escalation factors specific to the MBSD Project. A copy of this memo is included in **Appendix F**.

The escalation factor used to escalate the HDR unit prices from March 2014 to June 2018 using the CWCCIS is 1.07 or 7%. The calculation and indices used are included in the table below.

Table 23-1: Escalation Factors for Estimate Update

Account	HDR Estimate Date	Current Estimate Date	Escalation Factor	Escalation Percent Increase
	Mar-14	Jun-18		
	Index	Index		
Composite All Accts	802.53	854.52	1.065	6.5%
Recommended for Use:			1.07	7%

The Class 5 type cost estimates for the Mid-Barataria Sediment Diversion Project’s Alternatives Workshop No. 2 did not include programmatic costs, such as E&D, construction monitoring and administration, and engineering support during construction. In addition, the estimates for Workshop No. 2 do not include the programmatic costs to be provided by the CPRA for mitigation and real estate. The reason for not including them is that these costs did not impact the results of the alternatives analyses occurring at Workshop No. 2. These costs are also not included in the Class 3 estimates prepared following Workshop No. 2.

23.1.5 Other Costs in the Mid-Barataria Sediment Diversion Cost Estimate

The Class 5 type cost estimates for the Mid-Barataria Sediment Diversion Project’s Alternatives Workshop No. 2 did not include costs for E&D, construction supervision and administration, and engineering during construction. In addition, the estimates for Workshop No. 2 do not include the costs to be provided by the CPRA for mitigation and real estate. The reason for not including them is that these costs did not impact the results of the alternatives analyses occurring at Workshop No. 2. These costs are also not included in the Class 3 estimates prepared following Workshop No. 2.

23.2 Cost Data Sources

Specifically for the Class 5 costs estimates developed for Workshop No. 2, there are three different estimating approaches used. They are as follows:

1. Independently verify and escalate the unit prices from an existing MBSD estimate from HDR dated March 2014 when appropriate.
2. Reference RS Means to determine unit prices for certain line items.
3. Reference publicly available bid data for pricing.

The Class 3 cost estimate reflects current and applicable pricing and addresses specific construction procedures for the various line items in the estimate to as much detail as is available at the time of the estimate. The estimated costs are based upon an analysis of each line item evaluating quantity, production rate, time, equipment, labor, materials and in-house knowledge and experience of design and cost engineers who either personally designed or estimated similar projects.

23.3 Comparative Estimates for Alternative Workshops

The cost estimates prepared for Workshop No. 2 are included in **Appendix F**.

23.4 Contingencies

In an attempt to identify and quantify the project cost risks at each level of design the cost estimating team in coordination with the DT has developed the following contingency structure.

For the Class 5 type cost estimates:

50% contingency: Higher than normal level of uncertainty
40% contingency: Normal level of uncertainty
30% contingency: Lower than normal level of uncertainty

For the Class 3 type cost estimates:

50% contingency: Higher than normal level of uncertainty
40% contingency: Normal level of uncertainty
30% contingency: Lower than normal level of uncertainty

Generally, design completion, design complexity, and construction difficulty and complexity were considered when assigning contingencies for the project cost features. When determining contingencies, the designers provided the cost estimating team with levels of certainty surrounding the design information at the time of the cost estimate. The cost estimators used professional judgment regarding quantifying the cost risk and corresponding contingencies associated with the uncertainty surrounding the design details.

23.5 Life Cycle Construction Cost Estimate of Selected Components

The Class 5 type cost estimates for the MBSD Project's Alternatives Workshop No. 2 include rough order of magnitude estimates of Life Cycle Costs over 50 years including operations and maintenance. The scope of the life cycle costs to be included for the project features was determined by the design engineers and resident experts based on historical knowledge of similar structures. Costs were determined from sources available on the internet and local and state governments that operate and maintain similar structures and/or project features. A summary of the life cycle costs for the selected alternatives is included in **Appendix F.**

23.6 Construction Schedule

It is estimated that the construction project will occur over a 5 year period of time and begin in June 2021 and end in June 2026. It is assumed that there will be concurrent construction of various project features in order to meet this construction schedule.

24. BENEFICIAL USE OF EXCESS MATERIALS

24.1 General

As an ancillary component to the construction of MBSD, the project will excavate millions of cubic yards of earthen material. Material that is considered suitable for levee construction will be used for construction of the Conveyance Channel levees and the temporary reroute of the MRL levee system. Material deemed unsuitable for use in levees will be used to provide benefits in the form of marsh creation or restoration. The DT will analyze two (2) alternative sites and prepare a detailed design approach for the preferred alternative.

24.2 References and Publications

- CPRA Marsh Creation Design Guidelines
- (BA-0164) Bayou Dupont Marsh Creation & Terracing
  - Geotechnical Report – Geo Engineers (10-14-14)
  - Draft Bid Set – Moffatt Nichol (2-25-16)
- (BA-0043-EB) Long Distance Sediment Pipeline Project
  - Geotechnical Report – Fugro Consultants (11-29-11)
- (BA-0039) Bayou Dupont Sediment Delivery System
  - Bid Plans – CPRA (8-11-2008)

24.3 Material Allocation

Material excavated from the Conveyance Channel and the Outfall Transition Feature will be used as earthen fill associated with the MBSD structure and conveyance levee or beneficially used for marsh creation. For the purposes of this section of the 15% BODR, material deemed unsuitable for levee construction is to be used for beneficial use of material (BUM). This is based on geotechnical data collected during the 2014 study. The below table quantifies the available, unsuitable material to be used for marsh creation.

Table 24-1: Material Allocation

MBSD Feature	Unsuitable Material (CY)
Conveyance Channel	515,472
Outfall	764,063
Total	**1,279,535**

24.4 Alternative Analysis

Two alternatives were evaluated for disposal of the Beneficial Use of Excess Material (BUM). Both sites are within an approximated three (3) miles from the Outfall Transition Feature. The first alternative considered is the Bayou Dupont BUM Alternative. The Wilkinson Canal Marsh Creation Alternative is the other consideration for placement of BUM. See **Appendix D** for a map showing the two alternatives in relation to the MBSD Project site.

24.4.1 15% Design Assumptions

During the 15% BOD Phase, minimal data was collected within the two alternative sites. The assumptions described within this section, provides the information currently being used in assessing these alternatives.

1. Survey data collected by TBS during the (BA-0043-EB) Long Distance Sediment Pipeline Project was used for assessing potential elevations within the Bayou Dupont BUM Alternative. It should be noted that this data is approximately 7 years old and since then, (BA-0164) Bayou Dupont Marsh Creation & Terracing – Phase III has been constructed within the neighboring area.
2. TBS performed a one (1) day exploratory survey to collect three (3) transects along the Wilkinson Canal Marsh Creation Alternative. No other existing elevation data was found within this area to be used for the 15% BOD Phase.
3. Geotechnical data from (BA-0039) Bayou Dupont Sediment Delivery System, (BA-0164) Bayou Dupont Marsh Creation & Terracing – Phase III, and (BA-0043-EB) Long Distance Sediment Pipeline. The geotechnical design showed variations in fill height between EL 2.0 (BA-0039), 2.5 (BA-0164), and 3.0 (BA-0043-EB).
4. Geotechnical investigations were not performed as a part of the 15% BOD Phase for BUM. Therefore, an average of the existing investigation performed within the Bayou Dupont area was used for the conceptual design and comparison of alternatives. Construction Marsh Fill Elevations are currently estimated at EL 2.5.
5. It is estimated that the Conveyance Channel and the Outfall Transition Feature will be dredged with an approximate 18" portable cutter head dredge. This will be further defined and evaluated during the 30% Design Phase with the CMAR contractor.
6. Estimated cut to fill ratio of 1.5 to 1 was used for the loss of dredge material from the borrow area to the fill site for the 15% BOD Phase. This is the same cut to fill ratio proposed for the BA-0043-EB LDSP Project. The LDSP Project used a 30" dredge and had 100% of the borrow area located within the Mississippi River. The cut to fill ration will be reevaluated with the geotechnical analysis once the CMAR is involved during the 30% Phase. A 1.5 to 1 cut to fill ration is considered conservative and may be reduced during design.
7. Estimated cut to fill ration for the construction of the containment dikes is estimated at 2.0 to 1.0. This is to account for shrinkage and settlement losses during construction. This will be further evaluated during the 30% design phase.

24.4.2 Bayou Dupont BUM Alternative

The Bayou Dupont BUM (BDBUM) Alternative is located in Jefferson Parish, LA and is bounded on the north by BA-0039 Increment II along the Chenier Traverse Bayou, the east by an unnamed canal and the BA-0043-EB Pipeline Corridor Extension, open water to the south, and a combination of broken marsh and open water to the west. The fill site falls within the River Rest, LLC property boundary. The BDBUM Alternative is located approximately 2.0 miles from the Outfall Transition Feature. This is the closest distance the dredge will be to the fill site. An additional 2 miles of Conveyance Channel will put the maximum distance of the borrow area to the fill site at approximately 4.0 miles. Booster pumps will be required to reach the fill site from the max distance. Temporary workspace will be required for a booster pump and will be included as a part of the permitted workspace. Locations of the booster pumps will be determined during the 30% design phase with the CMAR contractor with consideration given to access and water depth.

The Construction Marsh Fill Elevation is estimated at EL 2.5 based on recent geotechnical assessments within the area. Existing survey data (BA-0043-EB) was used to create a 3D surface model within the proposed fill site to evaluate the fill required. Natural ground elevations averaged EL 2.5. The overall acreage proposed for the BDBUM Alternative is approximately 119 acres.

24.4.3 Wilkinson Canal Marsh Creation Alternative

The Wilkinson Canal Marsh Creation (WCMC) Alternative is bounded on the south by the northern bank of Wilkinson Canal, the east by the Shell 20-Inch Delta pipeline and the BUM from the Wilkinson Canal Pump Station, and broken marsh and open water on the north and west. This alternative proposes to rebuild the northern spoil bank of Wilkinson Canal and construct a marsh platform that potentially could reduce the sediments discharged from the MBSD Project from entering Wilkinson Canal. The BUM area stretches approximately 1.6 miles from the boundary of Myrtle Grove Marina to south where Wilkinson Canal begins to angle slightly southeast toward Bayou McCutchen and Lake Laurier.

The dredge pipeline corridor is approximately 2.7 miles from the Outfall Transition Feature to the northern area of the WCMC fill site. There is also an additional 2.0 miles of conveyance corridor channel to reach the borrow material near the Intake Structure and 1.6 miles to reach the southern side of the WCMC fill site. Therefore the dredge pipeline corridor may vary from 2.7 miles to 6.3 miles pending construction operations. Although it is estimated that the dredge pipeline will not exceed more than 4.3 miles, which is the distance from the Outfall Transition Feature to the southern end of the WCMC, fill site. At least one booster pump will be required. Locations will be determined during the 30% design phase.

The Construction Marsh Fill Elevation is estimated at an EL 2.5 based on the same geotechnical data used for the BDBUM Alternative since geotechnical data within the Wilkinson Canal area was not available. This will also allow for similar comparison to BDBUM Alternative. Survey data was collected along 3 transects that traversed the proposed marsh creation area in the upper region, middle region and southern region of the 1.6 mile fill site. The most conservative cross section (i.e. deepest cross section) was used in evaluating the fill area for the 15% BOD Phase. Due to shallow water depths in comparison to the BDBUM Alternative, the overall acreage proposed for the WCMC Alternative is approximately 156 acres.

24.5 Design Approach

The design of the marsh creation area will follow CPRA's Marsh Creation Design Guidelines released in April 2018. During the 15% BOD Phase, a desktop site analysis was performed, conceptual layout, existing data gap analysis and data collection plan. The design approach described within this report will provide the framework for data collection task, design calculations, and construction plans.

24.5.1 Data Collection Services

Upon approval of the Data Collection Plan, acquisition of the data required for design will commence. Data within the MBSD project features being used for BUM has been collected within the 15% BOD Phase and is not included in this approach. The surveys required for marsh creation design will include the following field investigations:

1. Marsh Creation Area Survey
   a. Topographic and bathymetric surveys of transects

b. Magnetometer surveys of transects
c. Healthy marsh elevation surveys
d. Marsh shoreline surveys
e. Hazard Investigation (pipelines)
f. Containment dike / Perimeter Surveys

2. Dredge Pipeline Corridor Surveys – topographic, bathymetric, and magnetometer
3. Geotechnical Investigation of Marsh Creation Area – combination of borings and CPTs within the fill area and containment dike alignment.
4. Environmental surveys and cultural resource investigations will be performed by the EIS consultant and coordinated with the DT.
5. Deliverables will include Survey Methodology Report, field notes, point files, survey maps, and preliminary geotechnical investigation data report (GIDR). The data collected and presented within this report will be the foundation for building the BODR.

24.5.2 Preliminary Design Phase

24.5.2.1 Geotechnical Design

The geotechnical engineering report (GER) will utilize the boring logs and CPT logs presented in the GIDR. The soil samples will undergo a laboratory testing program to determine soil strength and material characteristics for the project features. This includes the standard test for marsh creation as well as specialized testing on dredged material such as settling column test and low-stress consolidation tests for composite samples prepared from the borrow area. Engineering for marsh creation design will focus on the following features:

1. Earthen Containment Dikes
   - Perform stability analysis to evaluate the geometry required for stable dike configuration (construction elevation, side slopes, and crown width);
   - Provide settlement curves, including immediate and consolidation settlement due to self-weight compaction and subsurface soils;
   - Provide recommendations related to setup time required for the newly placed material before dredged material slurry is placed in containment area;
   - Provide construction sequencing recommendations; and
   - Provide bearing capacity recommendations.
2. Marsh Creation Sites
   - Perform settlement evaluations using Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) for the dredged material slurry and settlement programs for the foundation soils starting with initial placement, then on selected intervals after placement;
   - Provide settlement curves for selected marsh fill elevations scenarios projecting settlement over the 20-Year project life for subsurface soils and self-weight consolidation of the dredged material. S&ME will evaluate the maximum, and minimum design elevation, then interpolate between these values to get the remaining curves; and
   - Provide dewatering recommendations for fill materials, as required.

24.5.2.2 Marsh Inundation Assessment

The two primary goals in this assessment are to determine the Construction Marsh Fill Elevation (CMFE) and the Target Marsh Elevation (TME). The percent inundation will be established based on local MHHW/MLLW elevations and will be adjusted over the course of the project life with calculating the Eustatic Sea Level Rise (ESLR). The inundation graph will be overlaid with the settling curve to determine the optimum CMFE for the project. This calculation will be compared with Healthy Marsh Elevation surveys conducted as a part of the quality control.

i. <u>Preliminary Design Report</u>
The results from engineering tasks will be combined into a single comprehensive decision document. The document will contain major design features and volumes. Project cross-sections and platform areas will be included. We will summarize the science and engineering calculations in support of the project design within the document. 30% construction plans for the marsh creation will accompany the design report.

ii. <u>Final Design Phase</u>
This submittal will use the information submitted and approved in the BODR. Deliverables will include construction plans, technical specifications, material take-offs, and a final design report. Throughout the various design phases of the project, the DT and the construction team will be working together to minimize the risk associated with predicted performance versus actual performance.

Appendix C

Pallid Sturgeon Entrainment and Population-Level Risk

Computation of pallid sturgeon entrainment and population-level risk

Nicholas Friedenberg and Jack Siegrist, Applied Biomathematics, Setauket, NY

November 11, 2019

I. Volumetric entrainment rate for shovelnose, pallid, and intermediate sturgeon

Summary

We considered two estimates of a volumetric entrainment rate of combined sturgeon species from the lower Mississippi River (LMR).

1. The volumetric entrainment rate presented in the 2018 Biological Opinion on Bonnet Carré emergency operations (USFWS 2018). This estimate made use of data from research at Davis Pond conducted by FWS personnel and a team from Nicholls State University (NSU) from 2009 to 2011. We refer to it as the FWS rate.
2. A rate based on a mark-recapture estimation using the same data. The mark-recapture approach incorporated information on the timing of captures and recaptures that was not used in the FWS estimate. We refer to it as the mark-recapture rate.

The FWS rate for combined sturgeon species was 1 sturgeon per 2.368 x 10^9 cubic feet diverted.
The mark-recapture rate was 1 sturgeon per 3.947 x 10^9 cubic feet diverted, indicating 40% fewer sturgeon per volume. Derivation of both rates is detailed below.

Data

Data on sturgeon captures at Davis Pond by FWS and NSU teams were provided by FWS personnel. The data detail the date and source of each sturgeon that was tagged. The majority of FWS-tagged fish were sourced from the Mississippi River and introduced into the Davis Pond channel.

Table 1: FWS tagged sturgeon released at Davis Pond

Date	Source	Number	Recaptured	Recapture_date
2009-07-09	channel	1	0	NA
2009-10-20	river	1	0	NA
2009-10-22	river	1	0	NA
2009-11-03	channel	1	0	NA
2009-11-17	channel	1	1	2009-11-19
2009-11-19	river	3	0	NA
2009-11-23	river	5	0	NA
2009-12-14	river	3	1	2009-05-27

Table 2: NSU sturgeon captures at Davis Pond

Date	Number	River.tagged	Channel.tagged
2009-07-09	1	0	0
2009-11-19	2	0	1
2010-03-11	1	0	0
2010-05-27	2	1	0

FWS entrainment estimate

The Service's approach to estimating the number of sturgeon entrained at Davis Pond leveraged a detection probability. The logic of their calculation follows.

- 20 unmarked sturgeon were tagged and released within Davis Pond by FWS (16) and NSU (4).
- Of these, NSU recaptured 2 during their sampling, suggesting their survey effort only detected 10% of the sturgeon present.
- The Service applied the 10% detection rate to the 4 unmarked sturgeon that NSU captured to conclude that a total of 40 sturgeon had been entrained.

Mark-recapture estimate of entrainment

Another approach to finding entrainment at Davis Pond is to use a mark-recapture estimate. This approach has the advantage of accounting for precisely when marked fish were available to be detected by the NSU team.

We used the unmodified Schnabel estimator (Schnabel 1938) for repeated mark-recapture data. This estimator is biased toward overestimation, which in our case is a conservative error. The typical bias correction is ineffective for such small samples (Chapman 1952) and would reduce estimated entrainment by half. We restricted our analysis to the three sturgeon that were caught and tagged in Davis Pond by FWS or NSU. The five river-sourced fish were excluded, since they were 1) not naturally entrained and 2) caught in a different population that lacked study-tagged fish. Table 3 provides the data used in our calculation.

The Schnabel estimator assumes that population size does not change. We therefore assumed that sturgeon leave the Davis Pond population at about the same rate they enter it, for instance by moving into the mid-Breton basin or dying. This assumption is reasonable in that there would otherwise be a large accumulation of entrained sturgeon. To maximize the estimate of entrainment, we assumed all of the individuals marked during the study remained in the local population, retained their tags, and were available for recapture.

Table 3: Captures and recaptures of FWS and NSU sturgeon, excluding fish sourced from river

Date	Marked	Captured	Recaptures
2009-07-09	0	2	0
2009-11-03	2	1	0
2009-11-17	3	1	0
2009-11-19	4	2	1
2010-03-11	5	1	0
2010-05-27	6	1	0

Referring to the column headings in Table 3, the estimate of entrainment is

$$\frac{\sum Marked*Captured}{\sum Recaptures} = 24 \ / \ 1 = 24.$$

This is exactly 60% of the FWS estimate. Hence, all projections of volumetric or total entrainment that follow are 60% of those based on the FWS estimate of entrainment.

The detection rate based on the mark-recapture estimate was found from the number of sturgeon NSU captured divided by the estimate of total entrainment: $\frac{5}{24} = 0.208$.

Volumetric entrainment rate for combined sturgeon species

The Service estimated the total volume of water diverted through the Davis Pond diversion during the NSU study was 9.472 x 10^{10} cubic feet. Volumetric entrainment rates were obtained by dividing the number of sturgeon entrained by this volume. The FWS estimate of volumetric entrainment rate for combined sturgeon species was 1 sturgeon per 2.368 x 10^9 cubic feet of water diverted from the LMR. The mark-recapture entrainment rate for combined sturgeon species was 1 sturgeon per 3.947 x 10^9 cubic feet.

II. Volumetric entrainment rate for pallid sturgeon

Summary

We estimated an entrainment rate specific to pallid sturgeon using an updated estimate of the expected proportion of pallids among entrained sturgeon. The updated proportion, 0.24, was similar to the value of 0.25 used in the 2018 BO.

The entrainment rate for pallids was then increased to account for young age classes that are typically not detected in studies in diversions or in the main stem of the LMR. Previous demographic modeling suggested these juveniles account for 21% of the pallid sturgeon population in the LMR (Friedenberg et al. 2013). The resulting volumetric entrainment rate for age 1+ pallid sturgeon was 1 per 7.795 x 10^9 cubic feet using the Service's numbers and 1 per 1.299 x 10^{10} cubic feet using the mark-recapture estimate. Details of the calculation of these rates are given below.

Proportion pallid sturgeon among entrained sturgeon

To find a volumetric entrainment rate specific to pallid sturgeon, it was necessary to estimate the frequency of pallids among all sturgeon entrained. The Service accomplished this using the frequency of pallids among the four sturgeon captured by the NSU team at Davis Pond, which was 0.25. However, this sample size was small and the frequency was higher than that of pallid captures reported in the adjacent reach of the Mississippi River, which was closer to 0.17 (Killgore et al. 2007a), suggesting the NSU data may have overestimated pallid entrainment.

Table 4: Sturgeon entrainment by species since 2008.

	Pallid	Shovelnose	Intermediate	Proportion pallid
BC 2008	14	41	0	0.25
Davis Pond	2	5	0	0.29
BC 2011	20	78	1	0.20
BC 2016	1	0	0	1.00
BC 2018	4	4	0	0.50
Total	41	128	1	0.24

We combined entrainment data from the FWS and NSU Davis Pond teams as well as all recent openings of the Bonnet Carré diversion (Table 4). Data for 2016 included one pallid sturgeon located by the NSU team. Data from monitoring after the closure of the second Bonnet Carré release in 2019 (17 pallid sturgeon and 208 shovelnose) were available for this analysis but were not included because they were so different from other Bonnet Carré events. There was no statistically significant difference among the included samples in the proportion of pallids using either a Chi-squared test of independence (simulated p-value = 0.25) or Fisher's Exact Test (p = 0.22). Though pallids made up a variable proportion of sturgeon entrainment during each study, the overall frequency of pallids among entrained sturgeon was 0.24, close to the Service's original estimate, with an exact 95% confidence interval of (0.18, 0.31).

Applying this mean frequency to entrainment at Davis pond suggested 9.6 and 5.8 pallid sturgeon entrained for the FWS and mark-recapture estimates, respectively. These numbers are presented without rounding to illustrate that entrainment of pallid sturgeon is slightly lower under the revised frequency.

Volumetric entrainment rate of age 3+ pallid sturgeon

It is important to note that pallid sturgeon found during sampling of the LMR and the Bonnet Carré spillway are almost without exception long enough to be at least 3 years old. This inference of age is based on an age-length relationship developed from morphometric pallid sturgeon sampled from the LMR (Killgore et al. 2007b). The entrainment rate of pallid sturgeon based on Davis Pond should therefore be considered a description of risk to age 3+ individuals.

The volumetric entrainment rate of age 3+ pallid sturgeon was 1 per 9.867 x 10^9 cubic feet for the FWS estimate or 1 per 1.644 x 10^{10} cubic feet for the mark-recapture estimate.

Volumetric entrainment rate for age 1+ pallid sturgeon

We assume that a volumetric rate of entrainment based on data from the Davis Pond study addresses only that part of the pallid sturgeon population that is age 3+. It is conservative to assume that younger fish were entrained without detection. We assumed this cryptic entrainment occurred at the same rate as that for older age classes. A demographic model developed for the LMR pallid sturgeon population (Friedenberg et al. 2013) suggested 21% of individuals are age 1 or 2. This estimate ignores young-of-year fish, which are expected to have a survival rate on the order of 1 in a million.

Incorporating age 1 and 2 fish into the volumetric entrainment rates yielded 1 pallid sturgeon per 7.795 x 10^9 or 1.299 x 10^{10} cubic feet for the FWS or mark-recapture estimate, respectively.

III. Volumetric entrainment scenarios for MBSD

Summary

Three scenarios of volumetric entrainment rates in the vicinity of the MBSD were developed from a set of data-supported assumptions about relative pallid sturgeon population density downstream of New Orleans. These consisted of two scenarios in which all stages are entrained and one scenario in which only juveniles are entrained. All three scenarios were consistent with the rarity of observations of pallid or shovelnose sturgeon downstream of New Orleans but make the conservative assumption that individuals are present in the vicinity of the MBSD. The analysis found it likely that population density was lower in the vicinity of the MBSD than it is upstream of New Orleans where entrainment has previously been measured. Trotline data from the LMR indicated a mean relative population density of 10%. A relative density of 50% was supported as a conservative upper limit. All data were consistent with the alternative hypothesis that only juvenile age classes have appreciable abundance downstream of New Orleans and are entrained in diversions without detection.

Background

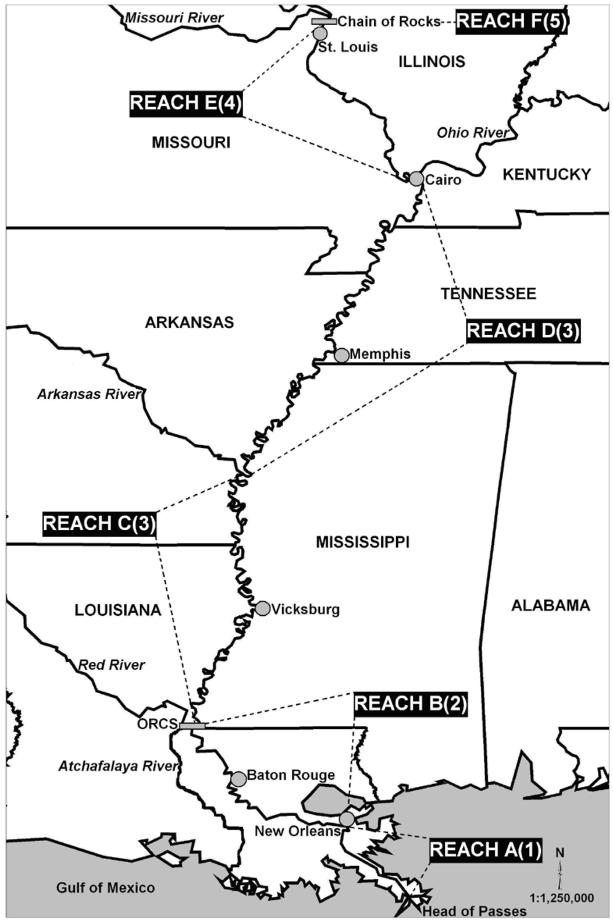


Figure 1: The lower and middle Mississippi River. Entrainment risk is assumed to affect pallid sturgeon downstream of New Orleans. Figure from Killgore et al. (2007a).

Evidence for the abundance and age structure of the pallid sturgeon population downstream of New Orleans is sparse. New Orleans is the farthest downstream a mature individual has been caught, at river mile (RM) 95. Two shovelnose sturgeon were caught at RM 85. Direct evidence for the presence of early life stages comes from the capture of two larval *Scaphirhynchus sp.* at RM 33, well below the proposed location of the MBSD (USACE 2017). The Army Corps constructs a temporary weir with dredge material at RM 50 during low water months to manage salinity. Individuals below that point in the river may essentially be lost from the population due to low habitat quality and seasonal inhibition to upstream movement by the sand weir.

We assumed volumetric entrainment was proportional to local population density. Developing a volumetric rate for the vicinity of the MBSD therefore required insight into the pallid sturgeon population density of that part of the LMR relative to the segment near the Davis Pond diversion. We investigated relative abundance of pallid sturgeon using catch per unit effort (CPUE) as a proxy. CPUE was derived from fish sampling data in the LMR provided by the US Army Corps ERDC Environmental Laboratory (database snapshot obtained 30 July 2019). Effort was quantified as the number of 60-m trotlines deployed in the LMR, defined as the Mississippi River downstream of the confluence of the Ohio River at Cairo, IL. The data were divided into those collected upstream (1,300 trotlines) and downstream (62 trotlines) of New Orleans. For simplicity of analysis, each trotline was treated as an independent sampling unit.

Relative abundance hypotheses

We evaluated three potential patterns of relative population density downstream of New Orleans: 1. The same population density as upstream 2. A lower population density than upstream 3. Juveniles only (age 1 and 2)

1. Uniform population density

We evaluated whether it was reasonable to assume that population density upstream and downstream of New Orleans was equal. We used two approaches to determine the plausibility of this pattern.

Approach 1: Data from Caernarvon Small Diversion

The first approach considered a second small diversion, Caernarvon, similar in design to Davis Pond and sampled by NSU as part of the same diversion entrainment study. Sampling at the Caernarvon diversion at RM 81 did not yield any sturgeon, though the total number of fish of all species detected was about 90% that found at Davis Pond. This result might be expected due to lower diversion volume and a lower sampling effort at Caernarvon. However, it could also be the result of lower sturgeon density in the vicinity of Caernarvon.

Caernarvon diverted about 80% as much water as Davis Pond based on comparison of uncorrected USGS gage data over the period of the NSU study. This relative volume appears to be a reasonable estimate. Caernarvon's maximum flow rate is 75% of that at Davis Pond, but neither diversion operates at maximum capacity very often. Greater precision in this number is not necessary for our analysis. All else being equal, the diversion of 80% as much volume would entrain 80% as many sturgeon if population density was equal at both diversions.

We found relative sampling effort at Caernarvon by comparing units of trawl and gillnet effort at the two diversions. Effort was 128 units at Caernarvon and 219 units at Davis Pond, indicating 58% relative effort. We assumed the lower sampling effort reduced the detection rate of sturgeon proportionally.

With these assumptions, the number of sturgeon (pallid and shovelnose) expected to be detected at Caernarvon was calculated as the product of Davis Pond detections, the relative volume diverted, relative sampling effort, and relative sturgeon density. Calculation using the 4 sturgeon included in the FWS analysis yielded

$$4 \times 0.8 \times 0.58 \times 1 = 1.87$$

For the mark-recapture estimate (one additional sturgeon at Davis Pond), expected detections were

$$5 \times 0.8 \times 0.58 \times 1 = 2.34$$

In both cases, about 2 detections were expected. Assuming uniform population density without detections at Caernarvon implies that the expected 2 sturgeon were missed by chance. The Poisson probability of no detections when 2 are expected is 0.14.

Approach 2: Bayesian estimate of CPUE using informed prior

Between Cairo and New Orleans, ERDC caught 183 pallid sturgeon on 1,300 trotlines. A Bayesian estimate of the resulting CPUE was obtained by updating a gamma(1, 0) prior with the catch and effort information, yielding a gamma(1 + catch, 0 + effort) posterior distribution with mean (1 + catch) / (0 + effort). The estimate of CPUE upstream of New Orleans was therefore gamma(184, 1,300) with a mean of 0.142.

Using the posterior estimate of upstream CPUE could be used as an informed prior for downstream, the downstream mean CPUE was $(184 + 0)/(1,300 + 62) = 0.135$, suggesting little difference in population density.

This result is unconvincing for two reasons. First, there was not enough downstream effort to meaningfully modify the informed prior. Second, it produces an answer that seems unlikely. With a downstream CPUE of

0.135, the probability that a single trotline fails to capture any pallid sturgeon is $p = 0.874$. The probability of no pallids among 62 trotlines is $p^{62} = 0.00023$, suggesting that the absence of captures downstream would be improbable if population density was uniform over the LMR.

2. Lower population density downstream of New Orleans

There is general agreement that pallid sturgeon population density decreases in the lowermost reach of the LMR. This inference is supported by the rarity of observations of either pallid or shovelnose downstream of New Orleans. However, the few observations of *Scaphyrhynchus spp.* in that reach suggest the population density is not zero.

We used two approaches to examine what population density, relative to that upstream of New Orleans, would be reasonable to assume if all life stages were present.

Approach 1. CPUE downstream of New Orleans

The most direct approach to estimating downstream relative density is to look at the ERDC survey data for that part of the river in isolation. Using a prior distribution for CPUE of gamma(1, 0) as above, the posterior distribution was gamma(1,62), indicating a downstream mean CPUE of 0.016. The posterior distribution indicated a 99% chance that the true CPUE is less than 0.07. Compared with the mean upstream CPUE (0.14), the downstream CPUE therefore had a mean relative density of about 10% and a 99% probability that relative density is less than about 50%.

Approach 2. Data from Caernarvon Small Diversion

Following the data and logic for expected detections at the Caernarvon small diversion above, we sought a population density that would make zero detections more probable. Assuming 50% relative population density, the expected number of detections decreased from about 2 to about 1, increasing the probability of zero detections from 0.14 to 0.37.

3. Juveniles only (age 1 and 2)

Finally, it was consistent with all data to assume that Caernarvon did entrain sturgeon over the period of the NSU study but that those individuals escaped detection due to small size. In this scenario, only juveniles age 1 and 2 were abundant enough downstream of New Orleans to constitute a consistent proportion of potential entrainment at the MBSD. We further assumed that the pallid sturgeon population in the 262 river miles of the LMR between RM 50 (the sand weir) and RM 312 (the Atchafalaya River) was collectively at the stable age distribution predicted by the demographic model of Friedenberg et al. (2013), such that 21% of individuals were age 1-2. However, we assumed age 3+ individuals concentrate in the 217 river miles above New Orleans (RM 95), corresponding to the distributional limit reported in Killgore et al. (2007a). We assumed the juvenile population occupies the full length of the reach, as might be expected from the significant distance larvae are capable of drifting from upstream spawning locations (Kynard et al. 2007; Braaten et al. 2007; Braaten 2010; FWS 2018) and evidenced by the detection of larvae at RM 33.

This set of assumptions led to two expectations relevant to the potential impact of the MBSD:
1. A smaller number of total individuals in the lower reach of the LMR than expected from uniform population density (relative abundance = $\frac{217}{262} = 0.83$)
2. Restriction of entrainment risk to a subset of individuals (juvenile relative density = 0.21)

Volumetric entrainment scenarios

Of the above hypotheses about the relative population density of pallid sturgeon downstream of New Orleans, those that posited a lower population density of sturgeon were the most consistent with data. We developed three entrainment scenarios to investigate population-level impacts.

1. 50% relative population density

The assumption that pallid sturgeon population density falls by half downstream of New Orleans was consistent with the lack of detections at Caernarvon and was the one-tailed 99% upper credible limit of relative density based on CPUE in the ERDC trotline data.

With 50% relative population density, the volumetric entrainment rate of the MBSD would be half that of Davis Pond.

2. 10% relative population density

Our estimate of CPUE in the LMR computed separately for ERDC trotlines upstream and downstream of New Orleans suggested a mean relative population density of 10% downstream. Under this scenario, the volumetric entrainment rate of the MBSD would be 90% lower than that of Davis Pond.

3. Juveniles only

If we assumed the pallid sturgeon population in the vicinity of the MBSD only included juveniles, the volumetric entrainment rate was reduced by both the exclusion of older life stages and the assumption that the juvenile population itself was spread more thinly over the reach of the LMR downstream of the Atchafalaya River. These factors combined to lower expected volumetric entrainment by 83% relative to Davis Pond.

Volumetric entrainment rates across scenarios

Table 5 summarizes the expected volume diverted per pallid sturgeon entrained. A larger volume indicates a lower entrainment rate.

Table 5: Cubic feet diverted per pallid sturgeon entrained (x 10^{10})

	Ages entrained	FWS 2018 rate	Mark-recapture rate
50% density	Age 1+	1.5589	2.5982
10% density	Age 1+	7.7947	12.9911
Juveniles only	Age 1-2	4.4828	7.4713

IV. Projected entrainment of pallid sturgeon at the MBSD

Summary

The mean and standard deviation for the number of pallid sturgeon expected to be entrained at the initial population size was developed by applying the volumetric entrainment rates to estimates of annual diversion volumes provided by the CPRA. Volumes had high inter-annual variability (33%), implying that entrainment will be a highly variable phenomenon.

Projected annual volume diverted

CPRA projects the diversion of 9.04 x 10^{11} cubic feet per year on average with a standard deviation of 3.03 x 10^{11} cubic feet. The coefficient of variation of annual volume, 0.33, indicates high variability from one year to the next.

We found the estimated mean and standard deviation of pallid sturgeon entrainment for each of our scenarios by applying volumetric entrainment rates to the CPRA projections of entrainment volume. Variability was found by assuming the coefficient of variation in entrainment matches that of projected annual volume.

Table 6: Mean (SD) annual pallid sturgeon entrainment

	Ages entrained	FWS 2018	Mark-recapture
50% density	Age 1+	58.0 (19.1)	34.8 (11.5)
10% density	Age 1+	11.6 (3.8)	7.0 (2.3)
Juveniles only	Age 1-2	20.2 (6.7)	12.1 (4.0)

Estimates of expected entrainment ranged from a low of 7 age 1+ individuals per year to a high of 58 (Table 6). The juvenile-only scenario projected about 12-20 age 1 and 2 individuals entrained per year. Ignoring potential changes in population size, the mean total entrainment over a 50-year project based on the FWS volumetric rate was 2,900, 580, or 1,010 pallid sturgeon for the 50% density, 10% density, and juvenile-only scenarios, respectively. Using the mark-recapture volumetric rate, the scenarios of expected total entrainment were 1,740, 350, or 605. These projections of total take ignore potential impacts on population density and are therefore like to be overestimates. If entrainment caused a decline in population density over time, the volumetric entrainment rate would also decline. The following section describes the population modeling we conducted to investigate this dynamic.

V. Population-level impact of MBSD entrainment on pallid sturgeon

Summary

We used a stochastic age-based demographic model to assess the potential population-level impact of pallid sturgeon entrainment through the MBSD over 50 years. This population viability analysis (PVA) required us to make assumptions about population size, survival, reproduction, and dispersal. These assumptions were taken from a previous pallid sturgeon PVA model developed for the LMR (Friedenberg et al. 2013) and a published estimate of the lower bound on possible population density in the LMR (Friedenberg et al. 2018).

The PVA generally indicated low population-level risk associated with the MBSD under the 10% and juvenile-only entrainment scenarios. Median declines in abundance over 50 years were less than 5% and the chance of falling below 5,000 individuals in the LMR was less than 10%. The notably more conservative 50% population density scenario indicated median declines of 12-20% over 50 years associated with the take of 1,500 - 2,400 pallid sturgeon across all age classes, and a 16-32% chance of abundance less than 5,000 individuals in the LMR.

Background

Conversion of projected numbers of pallid sturgeon entrained to a population-level response required conversion of entrainment into a per capita rate (a mortality rate). This conversion required an estimate of the number of pallid sturgeon in the population.

Definition of the pallid sturgeon population

Pallid sturgeon occur over a range that includes the Atchafalaya River, the lower and middle Mississippi River, portions of the lower and upper Missouri river, and portions of the major tributaries of these rivers. We defined the population impacted by the MBSD as individuals occupying the LMR.

We considered the impact of MBSD entrainment at two spatial scales. The local scale focused on entrainment as a proportion of the population occupying the LMR from the location of the sand weir at RM 50 to the Atchafalaya River at RM 312. The larger spatial scale encompassed the entire LMR up to RM 953 at the confluence of the Ohio River. Our analysis addressed most of the Coastal Plain Management Unit defined in the updated pallid sturgeon recovery plan (FWS 2014), excluding the Atchafalaya River and the lower 50 RM of the LMR.

Pallid sturgeon abundance

The abundance of pallid sturgeon in the Mississippi River is not precisely known. A long-term effort to estimate population size through mark-recapture methods failed to recapture any individuals (Killgore et al. 2007a). Friedenberg et al. (2018) estimated there is a 95% probability that the population has more than 4 age 3+ pallid sturgeon per river kilometer (6.44 per RM). We used this population density to estimate population size (Table 7).

It is important to note that an upward revision of total pallid sturgeon population size would decrease projected population-level impacts of entrainment through the MBSD. This is because volumetric entrainment rates in our analysis were relative to the number of individuals thought to be entrained at Davis Pond and are independent of the estimate of total population size.

Table 7: Abundance of age 1+ pallid sturgeon used to calculate entrainment mortality at the scale of the local population and the lower Mississippi River (LMR)

	Local	LMR
50% density	1,952	7,177
10% density	1,806	7,031
Juveniles only	1,769	6,994

50% population density

This scenario assumed there were 3.22 age 3+ pallid sturgeon/RM in the 45 RM between the sand weir and New Orleans. For the 217 RM upstream to the Atchafalaya River, there were 6.44 age 3+ pallid sturgeon/RM.

10% population density

This scenario assumed a population density of 0.644 age 3+ pallid sturgeon/RM downstream of New Orleans.

Juveniles only

The scenario of juvenile-only entrainment based local abundance on an age 3+ density of 6.44 pallid sturgeon per RM in the 217 RM LMR from New Orleans up to the Atchafalaya River. Juveniles, though distributed over a larger area including the MBSD, were assumed to be 21% of the population downstream of the Atchafalaya River.

Mortality of pallid sturgeon due to MBSD entrainment

The number of individuals entrained was expected to increase or decrease with population size, reflecting a constant mean per capita risk of entrainment. We assumed this risk was uniform over age classes except in the scenario restricting MBSD entrainment to juveniles. We further assumed that all entrained individuals were lost from the population permanently and therefore treated entrainment as a source of mortality.

Table 8: Mean (SD) pallid sturgeon mortality rate due to MBSD entrainment, calculated at the local and lower Mississippi River scales using volumetric entrainment rates estimated by FWS or mark-recapture

	FWS 2018		Mark-recapture	
	Local	LMR	Local	LMR
50% density	0.030 (0.010)	0.008 (0.003)	0.018 (0.006)	0.005 (0.002)
10% density	0.006 (0.002)	0.002 (0.001)	0.004 (0.001)	0.001 (0.000)
Juveniles only	0.011 (0.004)	0.003 (0.001)	0.007 (0.002)	0.002 (0.001)

We estimated MBSD-associated mortality on two spatial scales. The local scale acknowledged that only individuals in the vicinity of the MBSD are at risk. The LMR scale acknowledged that the impact of the MBSD occurs in the context of a larger population extending beyond the range at which most intra-annual movements of pallid sturgeon likely put individuals at risk of entrainment. Table 8 provides mortality estimates at the two scales for each of the three scenarios based on the FWS and mark-recapture estimates of entrainment. Over the whole of the estimated LMR population, MBSD entrainment was projected to decrease abundance by 0.2-0.8% annually.

Cumulative impact over 50 years

We estimated cumulative take over 50 years as well as the increase in decline risk using a stochastic metapopulation population model. The baseline model structure and vital rates were taken from Friedenberg et al. (2013), a study that used LMR-specific information to parameterize an age-structured demographic model for pallid sturgeon and assessed the potential impacts of water diversions. The model was implemented in RAMAS Metapop Version 6, a widely accepted tool for PVA using stochastic, matrix-based population projection (EPA OSA 2009; Akcakaya and Root 2013; Morrison et al. 2017) using 10,000 replicate simulations of each scenario.

Survival and recruitment

Survival of age 1 individuals was initially set to 0.69, as reported for stocked pallid sturgeon in the Missouri River (Steffenson et al. 2010), then adjusted down to 0.65 to help balance births and deaths in the process described in the next paragraph. Age 2 survival was set to 0.75 (Hadley and Rotella 2009). Survival of age classes 3-24 was 0.93, as measured by the catch curve in the LMR (Killgore et al. 2007a). Age class 25 was a compounding terminal class in which individuals remained until dead. The mortality rate was doubled in this class to represent senescence, resulting in survival of 0.86.

The rate of recruitment to age 1 is not known. The model used an age-based function to estimate egg production, with maturity beginning at age 9 and all individuals mature by age 15. Survival from egg to age 1 was set to balance annual births and deaths. Survival of age 1 fish was also reduced from reported values in this process. With mean birth and death rates equalized, the model was run in stochastic mode and age-0 survival adjusted further until median abundance remained constant over 50 years. Population growth was assumed to be independent of population density. This balanced approach is conservative (Ginzburg et al. 1990). There is no compensation for the effects of entrainment through increased recruitment or survival, yet an equilibrium abundance is implied by the expectation that abundance remains nearly constant *on average* across replicate simulations. In individual replicates, abundance will drift above or below its initial value due to random annual variation in recruitment and survival. As a result, there is a risk of decline even under baseline conditions.

A complete table of the single-population demographic functions and parameters is presented in Appendix 1.

Annual variability was modeled by drawing lognormal random deviates around mean survival and recruitment rates. We assumed 10% variation in mortality and 50% variation in recruitment.

Dispersal

Table 9: Upstream dispersal rate from the lower reach and the relative fecundity and larval drift from the upper reach of the LMR

	Upstream dispersal	Relative fecundity	Larval drift
50% density	0.061	1.374	0.272
10% density	0.066	1.346	0.257
Juveniles only	0.068	1.339	0.253

The population model included two reach-level populations connected by dispersal. While the middle Mississippi River (MMR) is not in the model, we used a reported emigration rate from a telemetry study in the MMR (Koch et al. 2012) to calculate inter-reach movement rates for the LMR. MMR emigration was as much as 15% per year. If there are 6.44 age 3+ pallid sturgeon per RM and 21% of the population is age 1 and 2, then approximately 240 pallid sturgeon emigrate from the MMR annually, assuming all ages have the same dispersal rate. If half of these individuals, 120, move downstream to the upper reach of the LMR (between the MMR and the Old River Control Complex), a reciprocal number must move upstream from the LMR if we assume dispersal does not alter the relative population density of reaches over time. We assume an equal number of individuals also move downstream to the lower reach of the LMR, producing an annual dispersal rate of 0.023. Again assuming that reaches exchange equal numbers of individuals, 120 pallid sturgeon move upstream from the lower reach of the LMR every year, representing a dispersal rate that varies with the differences in population size among entrainment scenarios (Table 10).

Larval drift and relative fecundity

The lower reach of the LMR lacks hard substrates (Baker et al. 1991) that act as natural spawning habitat (Dryer and Sandvol 1993). Hence, we assumed there was no reproduction in the lower reach. Rather, we elevated the recruitment rate in the upper reach of the LMR to produce surplus young of year. We assumed these move to the lower reach via larval drift in numbers that maintain a uniform age distribution.

The proportion of larvae drifting from the upper to the lower reach of the LMR was computed as the proportion of LMR population residing in the lower reach (Table 10). The relative fecundity in the upper reach necessary to support this level of drift was computed as 1 / (1 - drift) (Table 10).

Total entrainment over 50 years

Table 10: Mean (SD) total entrainment of pallid sturgeon over 50 years

	FWS 2018	Mark-recapture
50% density	2,403 (292)	1,561 (186)
10% density	515 (62)	350 (47)
Juvenile only	1,020 (281)	647 (191)

Total entrainment projected by the population model was similar to the levels expected without a change in population size or annual variability in entrainment rates. Realized entrainment exceeded expected levels in the juvenile scenario, in which the survival of mature individuals was not affected by the MBSD and the impacted juvenile classes were sustained by larval drift from upstream of the Atchafalaya River.

Decline risk

Projected entrainment through the MBSD had less than a 5% impact on median abundance over 50 years in the 10% and juvenile-only scenarios relative to baseline models without entrainment. In the 50% relative abundance scenario, which had 5 times the entrainment rate of the 10% scenario, the impact on final median abundance was less than 5 times as high at 12-20% (Table 11).

Table 11: Median terminal abundance under baseline conditions and percent reduction with FWS or mark-recapture estimated entrainment

	Median baseline	% FWS reduction	% Mark-recapture reduction
50% density	7090	19.2	12.2
10% density	6971	4.6	3.9
Juvenile only	6964	4.8	3.5

Translated into an impact on annual population growth rate, median declines in all scenarios represented less than a 0.5% decrease in population growth. In most cases the impact was 0.1% or less (Table 12). For reference, a 0.5% decline compounds to about 5% per decade, while a 0.1% decline compounds to about 1% per decade.

Table 12: Reduction of mean annual growth of the LMR pallid sturgeon population (percent per year) for the FWS and mark-recapture estimates of entrainment

	FWS	Mark-recapture
50% density	0.43	0.26
10% density	0.09	0.08
Juvenile only	0.10	0.07

The probability of abundance falling below a given threshold at any time over the 50-year population projection is shown for each scenario in Figure 2. The horizontal distance between two curves on a plot indicates a difference in median minimum abundance. The vertical distance between two curves on a plot indicates a difference in the probability of falling below a given threshold. Due to environmental variation in reproduction, survival, dispersal, and entrainment, some trajectories that fall below a given threshold will subsequently recover.

We chose a threshold of 5,000 pallid sturgeon in the LMR, including all age classes, to examine the effect of projected MBSD entrainment on the risk of population decline. We examined the response of both minimum abundance (across all years of the model projections) and terminal abundance (Table 13). In the 10% and juvenile-only entrainment scenarios, the risk of decline below 5,000 roughly doubled with entrainment, though in all cases remained under 10%. In the 50% relative abundance scenario, decline risk increasedabout 10-fold under the FWS entrainment rate and 5-fold under the mark-recapture entrainment rate.

Table 13: The percent chance of abundance less than 5,000 pallid sturgeon in the LMR, measured over all 50 years of the population projections or in the terminal year alone.

	In any year			In 50th year		
	Baseline	FWS 2018	Mark-recapture	Baseline	FWS 2018	Mark-recapture
50% density	3.2	32.0	15.7	1.5	19.9	8.6
10% density	4.4	7.7	7.0	1.8	3.6	3.1
Juvenile only	4.8	8.7	7.1	2.1	4.0	3.1

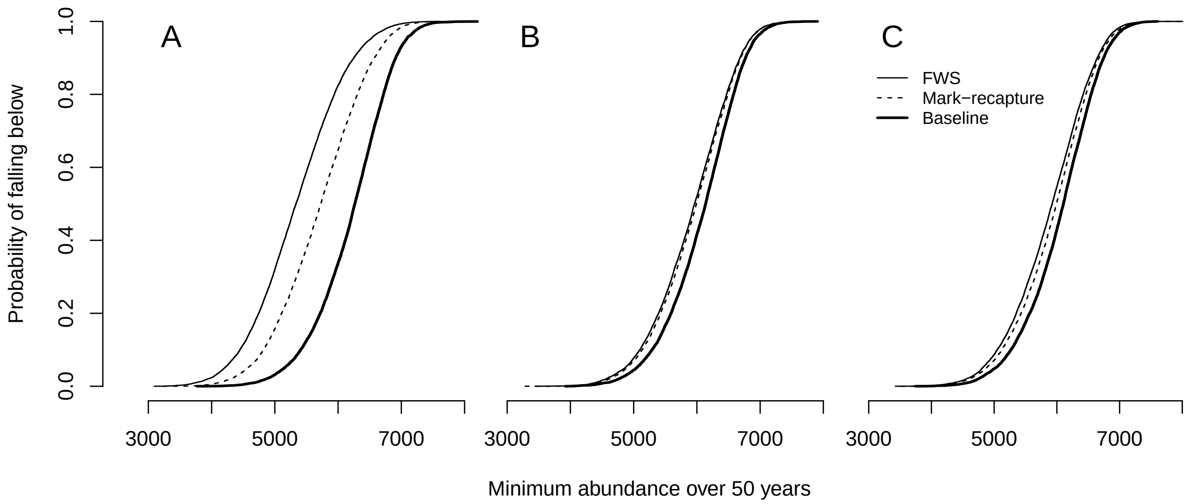


Figure 2: The distribution of minimum abundance over 50 years. A: 50% population density below New Orleans. B: 10% population density below New Orleans. C: Only juveniles entrained.

VI. Potential effect of water temperature on entrainment

Summary

Pallid sturgeon may be at lower risk of entrainment at low water temperatures found during winter. This hypothesis is supported by the observation that pallid sturgeon are caught in deeper water during winter months, when water temperature in the LMR falls below 12 C (DeVries et al. 2015). Also, few entrained pallids were found in monitoring after a January opening of the Bonnet Carré in 2016 (FWS 2018). Studies have noted reduced growth and survival of juvenile *Schaphyrynchus spp.* at 10 and 12 C (Kappenman et al. 2009) and reduced sustained swimming speed below 12 C (Adams et al. 2003), suggesting metabolic stress and the possibility that individuals may seek energetic refugia and reduce activity during winter. The ERDC trotline data, which include water temperature taken when trotlines were retrieved, evidenced a strong relationship between CPUE and temperature with a peak between 10 and 15 C.

We did not incorporate temperature effects into our population viability analysis for two reasons. First, low catch at low and high temperature should not alter our estimate of entrainment. This is because the volumetric entrainment rates we used were developed from data collected over a full year and therefore already incorporate the effect of seasonal variation in temperature on pallid sturgeon entrainment.

The second reason we did not consider temperature in our analysis is that the pattern illustrated above is a preliminary result and has not yet been corrected for possible sources of bias due to non-systematic sampling across latitude and seasons. Upon further investigation, the relationship between CPUE and temperature may prove to be spurious.

The effect of temperature could be leveraged to minimize take through seasonal curtailment of operations or to focus monitoring efforts on times of greatest risk. Consideration of climate change and the continued warming of rivers and streams (Kaushal et al. 2010) would suggest a shift in the date of maximum entrainment risk over time that could affect total expected entrainment by increasing or decreasing risk during times of peak water diversion.

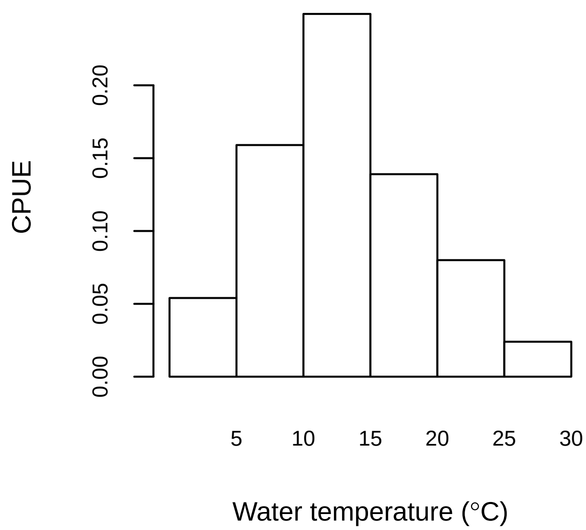


Figure 3: Catch per unit effort (CPUE) by temperature. Bin labels are degrees Celsius and indicate the upper limit of each bin.

VI. Literature cited

Adams, S. R., G. L. Adams, and G. R. Parsons. 2003. Critical swimming speed and behavior of juvenile shovelnose sturgeon and pallid sturgeon. Transactions of the American Fisheries Society 132:392-397.

Akçakaya, H. R., and W. Root. 2013. RAMAS Metapop: viability analysis for stage-structured metapopulations (Version 6). Applied Biomathematics, Setauket, NY.

Braaten, P. J. 2010. Spatial distribution of drifting pallid sturgeon larvae in the Missouri River inferred from two net designs and multiple sampling locations. North American Journal of Fisheries Management 30:1062-1074.

Braaten, P. J., D. B. Fuller, L. D. Holte, R. D. Lott, W. Viste, T. F. Brandt, and R. G. Legare. 2008. Drift dynamics of larval pallid sturgeon and shovelnose sturgeon in a natural side channel of the upper Missouri River, Montana. North American Journal of Fisheries Management 28:808-826.

Chapman, D. G. 1952. Multiple sequential sample censuses. Biometrics 8:286-306.

DeVries, R. J., D. A. Hann, and H. L. Schramm. 2015. Increasing capture efficiency of pallid sturgeon *Scaphirhynchus albus* (Forbes and Richardson, 1905) and the reliability of catch rate estimates. Journal of Applied Ichthyology 31:603-608.

Dryer, M. P., and A. J. Sandvol. 1993. Recovery plan for the pallid sturgeon (*Scaphirhynchus albus*), United States Fish and Wildlife Service. Denver, CO.

Friedenberg, N. A., J. J. Hoover, K. A. Boysen, and K. J. Killgore. 2013. Water diversions and pallid sturgeon population viability in the lower Mississippi River: uncertainties and priorities for ecological risk assessment, Pages 151-177 in ERDC Environmental Laboratory, ed. Entrainment Studies of Pallid Sturgeon Associated with Water Diversions in the Lower Mississippi River, New Orleans District, U.S. Army Corps of Engineers.

—. 2018. Estimating abundance without recaptures of marked pallid sturgeon in the Mississippi River. Conservation Biology 32:457-465.

Ginzburg, L. R., S. Ferson, and H. R. Akçakaya. 1990. Reconstructability of density dependence and the conservative assessment of extinction risks. Conservation Biology 4:63-70.

Hadley, G. L., and J. J. Rotella. 2009. Upper basin pallid sturgeon survival estimation project, Montana State University. Bozeman, Montana.

Kappenman, K. M., M. A. H. Webb, and M. Greenwood. 2013. The effect of temperature on embryo survival and development in pallid sturgeon *Scaphirhynchus albus* (Forbes & Richardson 1905) and shovelnose sturgeon *S. platorynchus* (Rafinesque, 1820). Journal of Applied Ichthyology 29:1193-1203.

Kaushal, S. S., G. E. Likens, N. A. Jaworski, M. L. Pace, A. M. Sides, D. Seekell, K. T. Belt et al. 2010. Rising stream and river temperatures in the United States. Frontiers in Ecology and the Environment 8:461-466.

Killgore, K. J., J. J. Hoover, S. G. George, B. R. Lewis, C. E. Murphy, and W. E. Lancaster. 2007a. Distribution, relative abundance and movements of pallid sturgeon in the free-flowing Mississippi River. Journal of Applied Ichthyology 23:476-483.

Killgore, K. J., J. J. Hoover, J. P. Kirk, S. G. George, B. R. Lewis, and C. E. Murphy. 2007b. Age and growth of pallid sturgeon in the free-flowing Mississippi River. Journal of Applied Ichthyology 23:452-456.

Kynard, B. K., E. Parker, D. Pugh, and T. Parker. 2007. Use of laboratory studies to develop a dispersal model for Missouri River pallid sturgeon early life intervals. Journal of Applied Ichthyology 23:365-374.

Morrison, C., C. Wardle, and J. G. Castley. 2016. Repeatability and reproducibility of population viability analysis (PVA) and the implications for threatened species management. Frontiers in Ecology and Evolution 4:4:10.3389/fevo.2016.00098.

Schnabel, Z. E. 1938. The estimation of total fish population of a lake. The American Mathematical Monthly 45:348-352.

Schultz, D. L. 2013. Fish entrainment by freshwater diversions of the lower Mississippi River in ERDC Environmental Laboratory, ed. Entrainment studies of pallid sturgeon associated with water diversions in the lower Mississippi River, New Orleans District, U.S. Army Corps of Engineers.

Steffensen, K. D., L. A. Powell, and J. D. Koch. 2010. Assessment of hatchery-reared pallid sturgeon survival in the lower Missouri River. North American Journal of Fisheries Management 30:671-678.

USACE. 2017. Biological Assessment: Bonnet Carré Spillway 2011 and 2016 emergency operations, New Orleans District.

USEPA. 2009. Summary report: risk assessment forum technical workshop on population-level ecological risk assessment, Office of the Science Advisor, United States Environmental Protection Agency. Washington, D.C.

USFWS. 2014. Revised recovery plan for the pallid sturgeon (*Scaphiryhynchus albus*). U.S. Fish and Wildlife Service Northern Rockies Fish and Wildlife Conservation Office. Denver, Colorado, USA.

—. 2018. Biological Opinion: Bonnet Carré Spillway 2011 and 2016 emergency operations. FWS Log #: 04EL1000-2018-F-0590, Louisiana Ecological Services Office. Lafayette, LA.

CONFLUENCE
ENVIRONMENTAL COMPANY

O2: Biological Assessment Correspondence
(to be provided in the FEIS)

O3: USFWS Biological Opinion
(to be provided in the FEIS)

O4: NMFS Biological Opinion
(to be provided in the FEIS)